International Water Supply Association Eleventh Congress 13-17 September 1976

Association Internationale des Distributions d'Eau Onzième Congrès 13-17 Septembre 1976

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SUNDAY 12th September, 1976	MONDAY 13th September, 1976		TUESDAY 14th September, 1976		
	"Main Hall" **9.45 a.m.** **Opening Session and Installation of the President**	"Main Hall" **9.15 a.m.** **General Report 1** Long term planning of water supply **Author:** Prof. P. L. Knoppert (Netherlands) **Chairman:** Chester A. Ring III (U.S.A.)	"Glass Hall" **9.15 a.m.** **Subject 5** New developments in water disinfection **Author:** A. Diatchkov (U.S.S.R.) **Chairman:** Prof. F. Chevelev (U.S.S.R.) **11.00 a.m.** **Subject 8** Design, construction and operation of service reservoirs **Author:** Prof. A. Baur (Germany) **Chairman:** Dr. H. Zander (Germany)	"Blue Hall" **9.15 a.m.** **Group Discussion 1** Mathematical modelling and simulation **Convener:** Dr. R. Heck (Germany) **Leading Discussors** P. Yletinen (Finland) B. Hofmann (Germany) H. Manczek (Poland) **11.00 a.m.** **Group Discussion 5** Measurement of turbidity **Convener:** A. Lencastre (Portugal) **Leading Discussors** M. Buffle (France) A. P. Meijers (Netherlands) F. A. Van Duuren (S. Africa)	"Main Hall" **9.15 a.m.** **General Report 2** Control of water supply demand **Author:** J. A. Young (Great Britain) **Chairman:** S. Priha (Finland)
Registration	**2.30 p.m.** **Special Session—Water Quality Management of a River Basin with Particular Reference to the River Rhine.** Ecological aspects of water management in the Rhine catchment area Prof. W. Stumm (Switzerland) The relationship between water supply and the river Prof. H. Sontheimer (Germany) Legal aspects of international river management S. Patijn (Netherlands) **Summing up by the Chairman:** **G. Dejouany (France)**	**2.30 p.m.** **Committee on Pollution and Protection of Water Sources** **Chairman:** C. Gomella (France) 1. Quality criteria for water **Author:** Prof. P. L. Knoppert (Netherlands) 2. Protection of water supplies in the view of the evolution in energy **Author:** J. Dirickx (Belgium)	**2.30 p.m.** **Committee on Corrosion and Protection of Underground Pipe Lines** **Chairman:** M. Chalet (Belgium) 1. Chemistry of natural waters with particular reference to their aggressivity **Authors:** L. Legrand & G. Poirier (France) 2. Practical measures for the cathodic protection of R.C. pipes. Insulation joints for large diameter pipes **Author:** F. Neussner (Spain) 3. Dissolution of materials from service pipes and house installations and its sanitary aspects **Author:** Kate Nielsen (Denmark) 4. Measurement of potential at cut-off points on cathodically protected systems **Authors:** W. G. Von Baeckmann (Germany) R. Petermann (Switzerland)	**2.30 p.m.** **Committee on Water Supplies in Developing Countries** **Chairman:** C. van der Veen (Netherlands) 1. Review of the work of leading international organisations in water supplies in developing countries **Author:** Dr. B. H. Dieterich, WHO, (Switzerland) 2. Water supplies problems in rapidly expanding cities (a) Lagos **Author:** B. A. Olodude (Nigeria) (b) Jakarta **Author:** Irwin Nazir (Indonesia)	**2.30 p.m.** **Committee on Water Quality and Treatment** **Chairman:** Dr. E. Windle Taylor (G.B.) 1. Toxicity of polynuclear aromatic hydrocarbons (PAH) and nitrates in water and measures for their removal **Authors:** Prof. Borneff (Germany) & Dr. P. Fuller (Great Britain) 2. Biological monitoring of water abstracted for a potable water supply **Authors:** G. Leynaud (France) & Dr. Poels (Netherlands) 3. Basic criteria for water purification plants of small capacity **Authors:** A. Ambrosio & M. F. Silva (Portugal)
6.30 p.m. **Reception to welcome delegates, offered by the Netherlands Waterworks Association (Vewin) the Testing and Research Institute of the Netherlands Water Undertakings KIWA Ltd., and the Netherlands Water Engineers Association (V.W.N.)** (Van Gogh Museum)	**8.00 p.m.** **Official Reception by the Minister of Health and Environmental Protection and the Lord Mayor of the City of Amsterdam in the Rijksmuseum, Stadhouderskade 42, Amsterdam**	**Concert by the Amsterdam Philharmonic Orchestra in the Concert Hall, Van Baarlestraat 98**			**Free Evening**

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WEDNESDAY 15th September, 1976			THURSDAY 16th September, 1976		FRIDAY 17th September
"Glass Hall"	"Blue Hall"	"Main Hall"	"Glass Hall"	"Blue Hall"	**Tours and Excursions**
9.15 a.m. **Subject 6** Peak load waterworks **Authors:** L. Ulmgren & M. Westermark (Sweden) **Chairman:** Leo Louis (U.S.A.)	**9.15 a.m.** **Group 2** Foreign organic substances in water—nature, measurement, effects **Convener:** G. Wijnstra (Netherlands) **Leading Discussors:** M. Fielding (G.B. N. Tambo (Japan) W. Van Der Meent (Netherlands)	**9.15 a.m.** **Subject 1** Automatic control of large water systems **Author:** K. Gotoh (Japan) **Chairman:** T. Ishibashi (Japan)	**9.15 a.m.** **Subject 2** Sludge disposal **Author:** G. Van der Cruyce (Belgium) **Chairman:** F. Snel (Belgium)	**9.15 a.m.** **Group Discussion 4** Biological principles in filtration **Convener:** T. Tzachev (Bulgaria) **Leading Discussors:** Mme Rizet (France) Z. Brulinsky (Poland)	
11.00 a.m. **Subject 4** Relationship between the environment and water development schemes **Author:** R. Hazen (U.S.A.) **Chairman:** P. F. Stott (G.B.)	**11.00 a.m.** **Group Discussion 3** Sanitary and operational control by rapid bacteriological methods **Convener:** Dr. E. Windle Taylor (Great Britain) **Leading Discussors:** K. Megay (Austria) G. J. Bonde (Denmark) M. S. Comendador (Spain)	**11.00 a.m.** **Subject 3** General review of waterborne diseases **Author:** D. L. Coin (France) **Chairman:** Prof. Mendia (Italy)	**11.00 a.m.** **Subject 7** Water for fire fighting **Authors:** J. Bernis & J. F. Galan (Spain) **Chairman:** J. Linati Bosch (Spain)	**11.00 a.m.** **Group Discussion 6** Finance in water authorities **Convener:** K. F. Roberts (G.B.) **Leading Discussors:** Z. Janakiev (Bulgaria) P. Holzel (Germany) E. C. Gilliland (G.B.)	
2.30 p.m. **Committee on Water Distribution** **Chairman:** R. J. Laburn (S. Africa) 1. Pipes and pipelines—design criteria and experiences in use of various materials **Author:** R. Y. Bromell (G.B.) 2. Pressure control in distribution systems **Author:** Dr. Chr. Laske (Germany) 3. Advances in the protection of distribution systems against backflow **Author:** W. C. Wijntjes (Netherlands)	**2.30 p.m.** **Committee on Public Relations** **Chairman:** R. J. Clark (G.B.) 1. Planning, organisation and control of an integrated and fully comprehensive public relations department in a water authority **Author:** C. Cotton (Great Britain) 2. Interpretation of statistics and financial matters in public relations terms **Author:** H. Gundermann (Germany) 3. A P.R. exercise—"use soap wisely"—save money and reduce pollution **Author:** Dr. K. Zwintzscher (Germany)	**Committee on Education and Training of Waterworks Personnel** **Chairman:** S. G. Barrett (G.B.) 1. Guidelines for the development of training programmes in water supply—**Working Group on Education and Training** 2. The training of water treatment plant operators **Author:** A. B. Redekopp (Canada) Training for distribution work in a water authority **Author:** C. Dyard (France)	**2.30 p.m.** **Committee on Water Meters and Water Metering** **Chairman:** A. Achten (Belgium) 1. Choice of type and determination of dimensions of water meters **Author:** Mr. Lauterbach (Germany) 2. Test benches for water meters **Author:** M. Sollman (Netherlands) 3. Maintenance of water meters **Author:** R. L. Williams (U.S.A.)	**2.30 p.m.** **Committee on Desalination** **Chairman:** C. Adam (Belgium) 1. Energy consumption of different desalination techniques in view of increasing energy costs **Author:** J. Franquin (France) 2. Reverse osmosis for public drinking water supply and its use for desalination of brackish water and for treating heavily polluted rural waters **Author:** Dr. D. Kuiper (Netherlands)	
		"Dutch Night" Buffet Dinner and Dance Music			

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INTERNATIONAL WATER SUPPLY ASSOCIATION

Amsterdam Congress

Conduct of Sessions - 14-16 September 1976

1. Place

All sessions will be at the RAI Congress Centre Europaplein Amsterdam.

Three Halls namely the Grand Hall, the Blue Hall and the Glass Hall will be in use concurrently on Tuesday 14 to Thursday 16th September inclusive.

2. Timetable

The complete timetable is enclosed.

3. Simultaneous Interpretation

Simultaneous interpretation will be operating in English, French and German in all three Halls.

4. Conduct of Sessions (General Reports)

i) Three hours is allowed for the presentation and discussion of a General Report namely from 9.15 a.m. to 12.30 p.m. with a break of 15 minutes at 10.45 a.m.

ii) The Report will be circulated in advance and be taken as read. The General Rapporteur will be allowed 30 minutes to introduce the report.

iii) The Chairman will then call the national rapporteurs who will be allowed five minutes each.

iv) The Chairmen will then call discussors from the floor calling first those who have notified their intention to speak by completing a discussion slip. No discussor will be entitled to speak for more than five minutes save at the discretion of the Chairman.

v) No later than fifteen minutes before the end of the session the Chairman will close the discussion to give the General Rapporteur opportunity to reply to the discussion.

5. Conduct of Sessions (Special Reports)

i) One and a half hours is allowed for the presentation and discussion of a Special Report.

ii) The Report will be circulated in advance and taken as read.

iii) The Chairman will then call the nominated discussors who will be allowed five minutes each.

iv) The Chairman will then call discussors from the floor calling first those who have notified their intention to speak by completing a discussion slip. No discussor will be allowed to apeak for more than three minutes save at the discretion of the Chairman.

v) No later than ten minutes before the end of the session the Chairman will close the discussion to give the Author opportunity to reply to the discussion.

6. Conduct of Sessions (Standing Committees)

i) Two and a half hours is allowed for each session organised by a Standing Committee.

ii) Reports and papers will be circulated in advance and taken as read.

iii) In calling discussors from the floor the Chairman will call first those who have notified their intention to speak by completing a discussion slip.

iv) The amount of time to be allocated to each of the papers or discussions within a Standing Committee session and the method of conducting the session will be determined by the Chairman of the Standing Committee.

7. Conduct of Sessions (Discussion Groups)

i) One and a half hours is allowed for each discussion group.

ii) The objects of the discussion groups are to promote informal discussion on subjects of current interest, to identify topics for further discussion or investigation and to prepare a report to be included in the conference proceedings.

The Convenors have been responsible for nominating three leading discussors to introduce the discussion.

iii) The convenor will open the session promptly and will take no more than five minutes to introduce the general theme of the discussion group.

iv) The convenor will then call upon the leading discussors in turn to make their points. The convenor will allow a total of 30 minutes for this part of the session.

v) The convenor will then call for contributions from the floor giving priority first to other nominated discussors and second to those who have notified their intention to speak by completing and handing in a discussion slip.

vi) Ten minutes before the end of the session the convenor will draw the discussion to a close and will sum up the points raised by the Nominated discussors and discussion from the floor.

8. Proceedings

No verbatim record will be made of the proceedings at the presentation of General and Special Reports and at the Standing Committee sessions. Chairmen, authors and discussors who wish their remarks to be recorded must send a typescript to the Secretary-General, IWSA, 1 Queen Anne's Gate, London, SW1H 9BT to reach him not later than 30 November, 1976.

Similarly no verbatim record will be made of the Discussion Groups; but convenors and Secretaries will provide a summary of the proceedings for publication.

INTERNATIONAL WATER SUPPLY CONGRESS, AMSTERDAM 1976

DISCUSSION SLIP

Discussion

If you wish to be called by the Chairman to take part in the discussion in any session complete this form and hand it in either before the session to the Association Office in the RAI Congress Centre or at the session to the Chairman or Secretaries.

Name:..

Country: ....

Session title: ....

....

(Further copies of the discussion slip are obtainable at the Association's Office in the RAI Congress Centre.)

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Pipes and pipelines: Design criteria and experiences in the uses of various materials

by R. Y. Bromell

Assistant Director of Operations, Severn-Trent Water Authority

1 Preface

This report has been drawn up following a review of information supplied by members of the International Standing Committee on Water Distribution:

Mr R. J. Weiss	Austria
Mr M. Chalet	Belgium
Mr T. Tchachev	Bulgaria
Mr K. C. Hassabis	Cyprus
Mr P. Kieler Jensen	Denmark
Mr J. Liimatainen	Finland
Mr R. Chappey	France
Dr H. Tessendorff	Germany
Mr E. C. Reed	Great Britain
Mr R. Gurevitz	Israel
Dr Ing. Pierluigi Martini	Italy
Mr W. C. Wijntjes	Netherlands
Mr G. A. Longe	Nigeria
Mr Janczewski	Poland
Mr A. M. Santos	Portugal
Mr R. J. Laburn	South Africa
Mr D. Pedro Grau Berdaguer	Spain
Mr D. Antonio Renedo	Spain
Mr Olle Niste	Sweden
Mr E. Shaw Cole	U.S.A.

In addition the author acknowledges the valuable help received from a large number of water undertakings, companies and individuals, some of whom are listed in Appendix A.

This paper is intended to cover pipes having internal diameters between 100 mm and 1 000 mm. For convenience they are designated between 100 mm and 300 mm as being small, over 300 mm and up to 600 mm as medium, while those pipes greater than 600 mm are defined as large. Pipes having an internal bore of less than 100 mm are not considered.

2 Introduction

Since the days of the Roman Empire aqueducts have been used to convey water from the source of supply to the point where it is to be used. Some of these aqueducts are still in existence and one can but wonder how many pipelines that are being laid today will still exist after the next two thousand years. A large proportion of capital investment in water supply is buried in pipes and careful consideration has to be given to the selection of the most appropriate material.

In order to assess performance it is necessary to examine pipe design, trench bedding requirements, trench loading, impact loading, hydraulic pressure resistance, permissible deflection, factors of safety, types of joints, resistance to corrosion and chemical action, flexibility of installation as well as the storage and handling of materials. After taking into account the cost of making repairs and the economics of making future connections there is little doubt that the final decision will depend upon these factors moderated by a strong element of personal choice.

3 Design of pipelines

3.1 General

Nearly 100 years ago pipes were thick, rigid and subject to little disturbance. Since that time operational requirements have become far more exacting due to higher pressures, tighter control of leakage and greater trench loadings. Over the same period pipe manufacturers have improved their technology, high strength materials have become available and economies have been achieved by a reduction in wall thickness. Thick walled pipes were capable of withstanding all vertical loads likely to be applied in normal installations, but thin walled pipes of today mean that consideration must be given to the supporting effect of the soil. Pioneer work on the application of soil mechanics to the design of pipelines was undertaken by Spangler, Marston and Schlick and showed how the bedding around a pipe can influence structural strength.

3.2 Rigid pipes

Asbestos cement, grey iron and prestressed concrete pipelines may be regarded as being rigid conduits that fail by rupture of the pipe walls at small deflections. The ability of a rigid pipe to carry an external load is directly related to structure ring strength which can be determined by the three edge bearing test. This takes into account the lateral pressure exerted against the sides of the pipe by the backfill material, the degree of compaction and the shape of the trench bottom. There are many conflicting standards for excavated and selected material for pipe embedment and a personal selection from various authorities is shown in Figure 1. Bursting pressures consist of known operating pressures plus surge or water hammer overpressures that require careful calculation and vary with the pipe diameter, pipe material, configuration of the pipeline and the method of operation. Surge suppression devices are frequently used to restrict the magnitude of water hammer.

3.3 Flexible pipes

Ductile iron, steel and pipes manufactured from various plastics materials tend to have a low inherent strength and some more than others deflect under vertical loadings. As the sides move outwards passive soil pressures are induced in the embedment and the development of more flexible pipe materials places a greater reliance on the backfill to withstand the vertical loads. Reference is made to the selected recommended embedments for ductile iron, steel and plastics pipes as shown in Figure 1. Smith discussed this point in greater detail

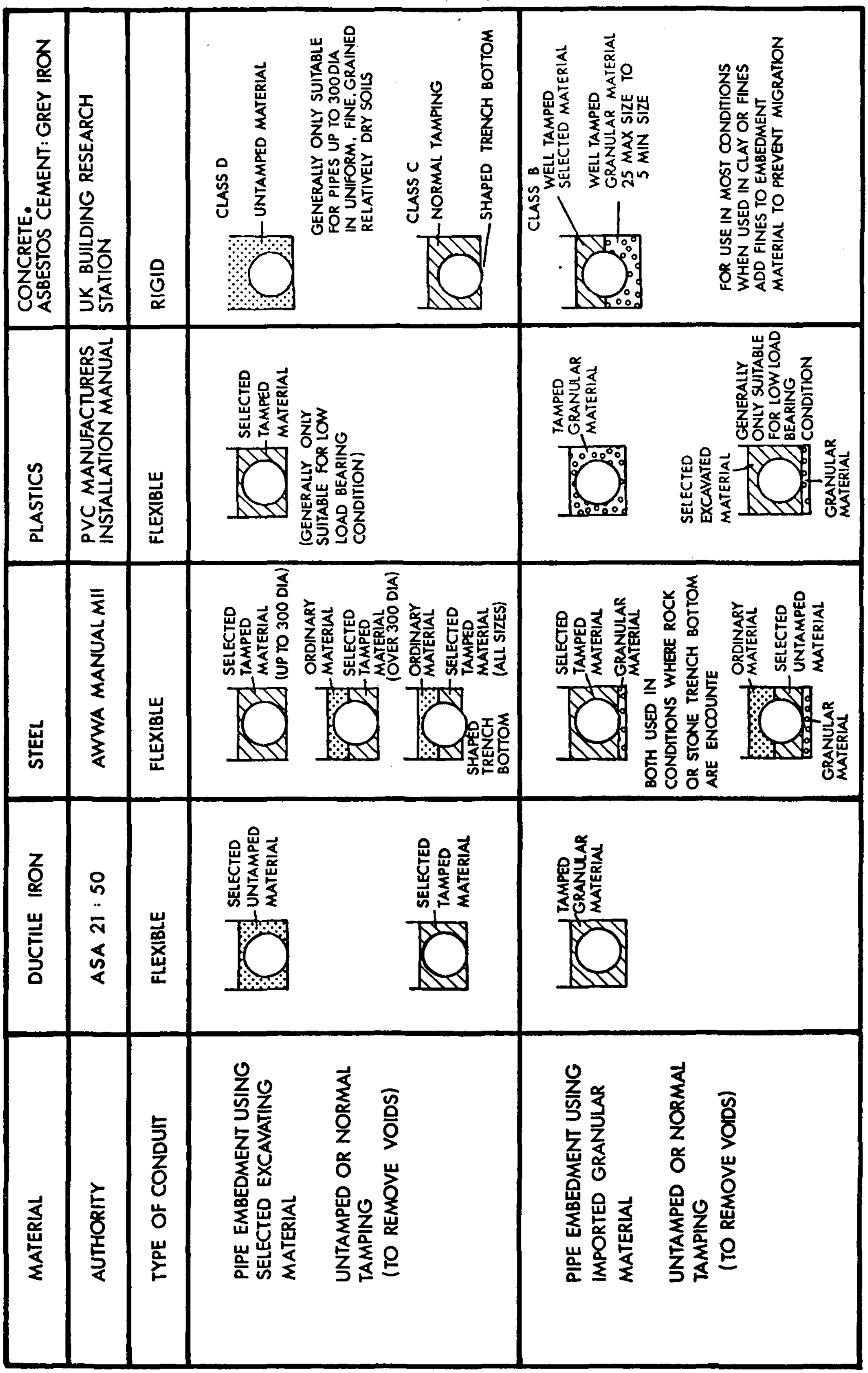

Figure 1—Recommended embedment for pipes of Differing materials.

earlier this year and suggested that the ASA 21.50 recommendations were very conservative. External loadings require deflection, bending, compression and buckling analysis. Uneven bedding, variations in backfill, temperature fluctuations and internal pressures give rise to longitudinal stresses. Checks are made for stress corrosion, cyclic strength and impact properties. It is generally accepted that the effect of transitory loads on flexible pipes is less than on rigid pipes, but some authorities assume the full rigid pipe load until more reliable facts are available.

The U.K. Construction Industry Research and Information Association (CIRIA) is about to publish a report on the design and construction of thin walled buried pipes where the wall to thickness ratio is such that the soil structure interaction is essential for stability. The pipe materials considered cover plain steel, corrugated steel, unreinforced plastics and reinforced plastics. It is understood that from a review of existing design methods, the recommended design procedure includes guidance on the suitability and properties of various soils to be used as backfill. The aim of this report is to show how to gain the full economic advantage from the soil/structure interaction.

3.4 Soil conditions

Where it is possible there are obvious advantages in using excavated spoil instead of having to import material for backfill around a pipeline. Most coarse grained soils are suitable for embedment provided stones larger than 30 mm diameter are not placed in contact with the pipe although larger stones up to 80 mm diameter may not be allowed in the remainder of the embedment. The best material is probably a mixture of soil with fines and graded gravel that will compact down into a dense mass. Examination of a route for a proposed pipeline is by samples taken from trial excavations and submitted to chemical examination together with a soil resistivity survey to ascertain whether the soil may be considered aggressive to certain pipe materials.

If there is a change of direction or internal diameter or a valve or a connection in a pipeline then the dynamic and static unbalanced forces arising from water velocity and internal pressure should be resisted by suitable anchorages. Special consideration should be given to pipes on steep slopes and to pipelines laid in bad ground having poor bearing pressure. Pipelines with fixed joints such as welded steel mains require careful attention as the pipe being continuous carries a diminishing stress away from the bends as the ground resistance to sliding accumulates.

The possibility of ground movement affecting the stability of a pipeline may be significant. In areas where subsidence or earthquakes may be encountered then either short lengths of ductile iron pipes or all welded steel pipes appear to be the most suitable. For pipes laid in poor ground suitable compaction of the embedment may meet the design specification. Where very poor ground is encountered then it may be necessary to excavate a trench up to five times the pipe diameter. Suitable embedment material may then be imported to an adequate level of compaction without support from the original ground. In really unstable conditions pipes should be placed on piles, one for each pipe, situated immediately behind the socket. This enables movement in one pile to be taken up in the joints on either side.

3.5 Pipe joints

3.5.1 Rigid joints

Pipe joints are rigid, semi-rigid or flexible. Rigid flanged joints are popular for exposed pipelines; particularly for pipework in pumping stations and treatment plants where disconnection may be required to be easy and quick. Welded joints on steel mains are used particularly for large diameter pipes where the internal welds can be made efficiently. Solvent joints were used extensively when small diameter plastics pipes were first introduced but have proved unsuccessful.

3.5.2 Semi-rigid joints

Hot run lead caulked joints on grey iron mains were used widely fifty years ago before being replaced by the introduction of flexible joints. The big disadvantage was the use of highly skilled labour necessary to make effective water tight run lead joints. Lead has the mechanical property of being very plastic and under slow progressive ground movement it flows under stress and allows the joints to take up considerable deflection without leaking.

3.5.3 Flexible joints

Flexible joints of a patented proprietary nature were developed to overcome the problems of run lead. Early mechanical joints of the bolted or screwed type incorporated a gland or sealing ring and this lead to the present rubber ring seal push in type of joint which is flexible and easy to make. Basic requirements for making most push in type joints are cleanliness and the avoidance of grit that can affect the efficiency of the seal between the rubber ring and the pipe. Proper location of the component parts is important. Some proprietary joints can tolerate abuse more than others, but the speed and simplicity of making the modern push in type of joint has led to rapid and widespread acceptance.

3.6 Hydraulic friction in pipelines

Much has been written in recent years on the limitations of empirical formulae for the hydraulic friction in pipelines. Hazen-Williams is still used widely for various types of pipe and takes into account the roughness of the bore either actual or predicted. Modern pipes have a coefficient C ranging from 135 to 155 and these figures may be maintained provided that there is no attack or erosion of the inside of the pipes and no internal deposition. Build-up of even a thin layer of slime can lead to a rapid reduction of C value; a few millimetres of deposit can effectively reduce the calculated diameter of a pipe by several centimetres. Blair, Colebrook and Lamont besides others have produced modified formulae for smooth pipes. Judicious choice should be made of the most suitable formula and due allowance must be provided for entry losses, bends, tees, changes in diameter, fittings, valves, meters and exit losses before arriving at the proper hydraulic conditions.

3.7 Standards and codes of practice

National and international standards and codes of practice for pipe materials are numerous and are being revised continuously. U.K. standards are published by the British Standards Institution and many have been metricated in recent years. In the U.S. the American Waterworks Association publishes its own standards. Many national standards are now based on recommendations produced by the International Standards Organisation. Relevant codes and standards are set out in Appendix B.

3.8 Comparative statistics

With a wide range of materials available to pipeline designers the selection of the most suitable can be a lengthy and laborious task. To illustrate the variations in mechanical properties and facilities available in the U.K. Table No. 1 has been drawn up showing the normal range of comparative design statistics for various pipe materials.

3.9 Factors affecting pipelines subject to traffic loadings

Work in the U.K. has been undertaken by the Transport and Road Research Laboratory. Trott and Gaunt recently presented a report on studies made during the construction of a main road when it was found that the most severe loadings on pipes occurred during the construction period when heavy contractors' vehicles traversed the pipes before the road was completed. Subsequent loading by road traffic produced lower strains and deflections.

A decade ago, Page carried out tests on concrete pipes laid under a road to investigate the impact factor produced by several different types of lorry travelling over a severe surface irregularity consisting of a length of timber 45 mm thick by 250 mm wide. With vehicles travelling at speeds varying between 3½ and 55 km/h the impact factor increased with speed. However, the relationship between impact factor and vehicle speed was found to be independent of size of pipe, its depth below the road surface, the type of pipe, the pipe bedding and the material used to backfill the trench.

fittings in the U.K., it is suggested that the same parameters are relevant to pipe materials, linings, jointing compounds and lubricants used in making joints and which can come into contact with water being transported.

Methods cover test procedures for assessing the ability of materials when used in contact with water to produce taste, odour, colour, turbidity or toxicity in the water or to support microbiological growth. Acute toxicity is determined by seeding water, in which the material has been soaked, with monkey kidney cells, applying standard tissue culture techniques and examining for toxic effects. Organoleptic and physical assessments for taste, odour, colour and turbidity use the

TABLE No. 1

COMPARATIVE STATISTICS FOR VARIOUS PIPE MATERIALS

	Ductile Iron	Steel	Prestressed Concrete	Asbestos Cement	Glass Reinforced Plastics	Grey Iron	Polyvinyl Chloride
Tensile Strength (MN/m²)	420	340 to 420	214 (Cyl)	22,5	150–500	18–40	45–60
Young's Modulus (GN/m²)	165	207	30	23,5	10–25	100	4
Elongation	7–10%	According to grade	½%	Nil		< 1%	40% min
Impact (IZOD) Resistance (Joules)							4–4,5
Beam Strength (MN/m²)	500	Depends on grade		24	15	95	92
Compressive Strength (MN/m²)	300	Depends on grade	40	45	70	615	68
Design fact of safety	2,5	2–2,5	1,8–4	4 bursting 2,5 crushing		2,5	1,5 min
Max. Working Temp. (°C)					30		60
Linear Thermal Expansion (°C)	11×10^{-6}	11×10^{-6}	11×10^{-6}	$11{,}8 \times 10^{-6}$	27×10^{-6}	11×10^{-6}	$50–60 \times 10^{-6}$
Thermal Conductivity (Cals cm/sec/cm²/°C)	13×10^{-2}	12×10^{-2}	24×10^{-4}		63×10^{-4}	12×10^{-2}	35×10^{-5}
Density (kg/m³)	70×10^{2}	78×10^{3}	$26–28 \times 10^{3}$	22×10^{2}	25×10^{3}	70×10^{2}	18×10^{3}
Hazen Williams Flow Coefficient (C)	135/150 (lined)	152 (lined)	150	140/155	150	148	150
Water Absorption	Nil	Nil	1,1–2% by weight	Up to 20% by weight	0,1	Nil	0,1
Diameter range (mm)	80–1 200	60–2 140	400–3 000	50–900	300–3 000	80–700	12,5–600
Type of Joints	Mech, push-in, flange, couplings	Weld, push-in, couplings	Push-in, couplings	Push-in, collar, couplings	Push-in, couplings	Lead, push-in, couplings	Push-in, solvent weld
Type of Fittings (material)	Ductile	Steel	Steel C/L/C	Ductile Steel	Steel, Ductile	Grey Iron	PVC iron or steel
Pressure Ratings (working) (bar)	25/40	16/70	4/18	7,5/12,5	6/16	10/16	6/15

4 Suitability of materials used in contact with potable water

Reference is made to the problems of taste and toxicity with early plastic materials. BS 3505 requires that PVC pipes will not have any detrimental effect on the composition of water flowing through them. Carbonic acid is used to extract any metals or other toxic substances and determinations made for lead, dialkyl C_4 and higher homologues and other toxic substances. Advanced methods for testing the suitability of materials for use in contact with water used for domestic purposes have been produced recently. Although such tests were evolved specifically for the testing of materials used in water

standard methods described in "Analysis of raw, potable and waste waters". Taste and odour tests are repeated using tap water with 1 mg/l of chlorine in order to detect chloroderivatives. An assessment for heavy metals should be within the limits specified by the World Health Organisation European Standards for Drinking Water. Microbiological growth tests are carried out by inoculating the soak water with a mixture of micro-organisms which are likely to include those capable of utilising as a source of nutrient a variety of natural and synthetic organic materials in an aquatic environment. Samples are examined quantitatively for coliform organisms, bacteria capable of growth at 37°C and 22°C Pseudomonas aeruginosa, fungi and yeasts. It is essential that these tests are carried out or supervised by qualified chemists and microbiologists who are familiar with the standard methods and media.

5 Life expectancy

In a recent survey water undertakings estimated the life expectancy of modern pipeline materials and these are shown summarised in Table No. 2.

TABLE No. 2

Pipe material	Expected life in years		
	Minimum	Maximum	Average
Grey iron	50	100	80
Prestressed concrete	50	100	73
Asbestos cement	50	100	65
Ductile iron	50	100	65
Steel	30	100	56
Plastics	50	67	*

* *statistically unreliable*

As may have been expected, a traditional material like grey iron had the longest expectation of life but was followed closely by cylinder and non-cylinder prestressed concrete pipes. Reservations may have been expressed about asbestos cement because it is likely to be laid in highly corrosive conditions. Ductile iron as a new material is treated with suspicion, but steel is also regarded with some doubts presumably because of possible failure of protective coverings to prevent corrosion. The statistics were unreliable for plastics materials but none gave a life expectancy greater than 67 years. It would appear from this table that the waterworks engineer's natural reluctance to accept new materials until absolutely convinced of longevity is well illustrated.

6 Trends in the use of different materials in the United Kingdom

Last year in the U.K. the Department of the Environment published a useful Report of a Working Party on Sewers and Water Mains that sets out views on trends in the choice of materials for water mains. Ductile iron is steadily replacing grey iron, particularly in the smaller sizes. This change has been accentuated by manufacturers' policy, nevertheless the use of ductile was generally favoured. Some reservations have been expressed regarding the life of ductile in comparison with grey iron due to the action of corrosion on the thinner pipe wall. Such doubts have been discounted by the research carried out by Collins, Fuller and Harrison and published by the British Cast Iron Research Association.

Steel pipes with bitumen linings and welded joints are predominantly used for large diameter trunk mains and it is thought that this practice would continue. Since ductile iron pipes are now being made in large diameters there might be a change from steel, although the heavier pipe could be a significant factor.

Prestressed concrete pipes are not at present being used extensively possibly due to the extra weight per metre of pipe. Asbestos cement pipes are being used in certain areas particularly where there is corrosive ground. In rural or mixed rural and urban areas either asbestos cement or plastics are used more frequently.

PVC pipes in small diameter sizes are used because of their corrosion resistance and ease of handling, particularly in rural areas. Push-in type joints are regularly used except when laying by means of the mole-ploughing technique when solvent weld joints are necessary.

7 Comparative statistics on pipe failures

7.1 General

Experience varies widely in comparing statistics for different pipe materials. This is not surprising in that ground conditions, laying methods, depth, traffic and temperatures vary from site to site. An interesting example is given in Table No. 3 which shows the rate of failure in street water mains in twelve selected Swedish towns from April 1974 to March 1975.

TABLE No. 3
FAILURES IN SELECTED SWEDISH TOWNS 1974–1975

Pipe material statistics

Pipe material	Number of failures			Pipelines length in km	Failures per km
	Trunk mains	Dist. mains	Total		
PVC	49	55	104	302	0,344
Steel	23	32	55	488	0,113
Grey iron	193	164	357	3 585	0,100
Asbestos cement	1	1	2	46	0,043*
PEL/PEH	3	6	9	314	0,029
Ductile iron	2	3	5	333	0,015
Concrete	1	0	1	296	0,003
Others	2	7	9	—	—
Total	274	268	542	5 364	0,101

* *statistically unreliable*

Major causes of failures

Cause	Grey cast iron	Steel	PVC
Corrosion	11%	81%	—
Uneven settlement	42%	2%	11%
Faulty material	—	—	37%
External influence	6%	2%	4%
Not specified	12%	—	34%
Not known	29%	15%	14%

In comparison Table No. 4 shows similar figures provided for two large cities in Poland from 1971 to 1974.

TABLE No. 4

FAILURES IN TWO LARGE POLISH CITIES 1971–1974

Pipe material statistics			
Pipe material	Failures per km/year		
	Small dia.	Medium dia.	Large dia.
Grey iron	0,2–0,32	0,37	0,45–0,86
Steel	0,09	0,03	
Asbestos cement	0,12–0,21		
Plastic	0,22		

7.2 Grey cast iron

Roberts and Regan have carried out a detailed investigation into the causes of fractures in grey cast iron water mains for the former Metropolitan Water Board, London, England over the period up to 1974. It was reported that the average fracture rate was 0,16/km/a for a total length of all sizes of approximately 15 000 km and it was found that the principal type of fracture was the transverse break, ranging from 90% in non-corrosive soil to 40% in a corrosive area. The rate of fractures decreased as the pipe size increased. Age was not thought to be a primary cause of failure, unless in a corrosive soil. In medium and large diameter pipes the main form of failure was longitudinal splitting due to ground pressure. It was suggested that mains under heavily trafficked major roads did not suffer unduly because highway authorities recognised the heavy-loading and had provided suitably strong road structures. Conversely, mains under the footpaths of major roads suffered, as did mains at road junctions. Mains seldom fractured under lightly trafficked roads, but suffered repeatedly where unusually heavy traffic turned on to a side road leading, for example, to a factory.

A study of meteorological data indicated that a spate of fractures followed a drop in air temperature. Figure 2 shows the relationship between average daily

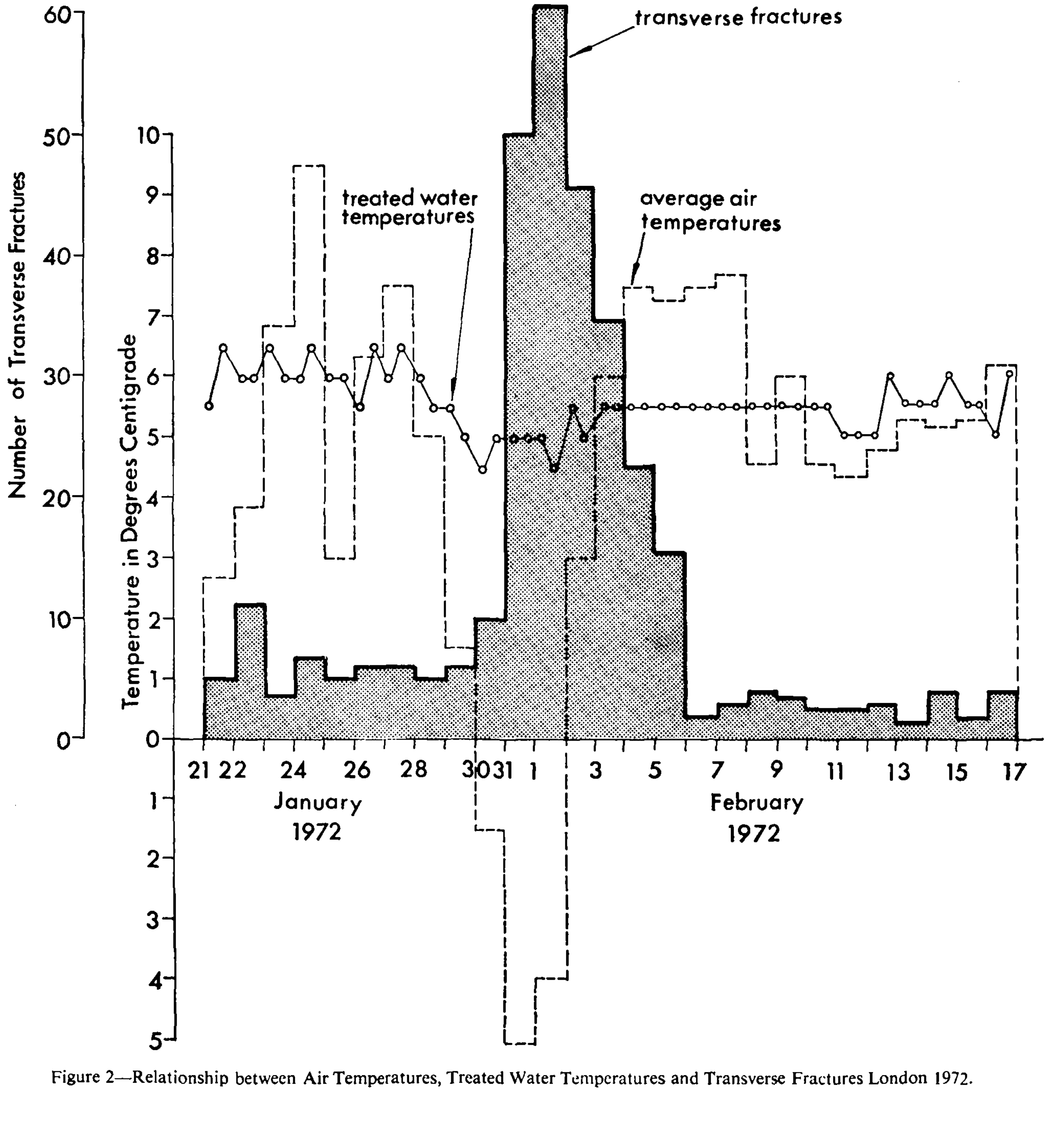

Figure 2—Relationship between Air Temperatures, Treated Water Temperatures and Transverse Fractures London 1972.

air temperatures at London airport, treated water temperatures and the total number of transverse fractures in the Board's area during January and February 1972. As water pipes in the United Kingdom are normally laid about 1 m deep it was not immediately obvious how air temperature could cause fractures of pipes and excite such a rapid response, particularly when the water is only marginally changed in temperature. It has been suggested that changes in air temperature had induced differential ground movement. One of the results of the survey was a recommendation that 100 mm ductile iron or 150 mm grey iron pipes should be the minimum sizes used provided there was adequate protection against corrosion.

7.3 Plastics

Plastics are now used widely for water pipes in many countries throughout the world. Most commonly used are unplasticised Poly Vinyl Chloride (uPVC), Polyethylene (PE), Polypropylene (PP) and Acrylonitrile-Butadiene-Styrene (ABS) with uPVC used in the greater quantity. Although it is appreciated that each thermoplastic material has different characteristics it would be impossible to deal with each separately in this paper and consequently these comments are directed primarily at PVC pipes. Expected design life is calculated to be in excess of 50 years. Impact strength decreases with lowering temperatures and tensile strength decreases with increasing temperatures. As a result careful handling of pipes is required in extremes of temperature. Exposure to sunlight tends to embrittle PVC and ultraviolet light will affect PE pipe. Flow characteristics are good with Hazen-Williams coefficient of 150 and no reduction of flow is expected with age unless slimes are formed on the bore of the pipe.

It has been acknowledged that there has been a continuous improvement of extrusion technology in the manufacture of plastics pipes as a result of the experience that has been built up over the years and failures that were rather more frequent than expected even with a new material have now reached reasonable proportions. The most dramatic type of failure in the past was the so-called spider line failure where as a result of a manufacturing defect pipes split longitudinally, sometimes several together even travelling through solvent joints. Large diameter pipes and integral joints gave trouble and for one reason or another solvent joints leaked, probably due to lack of care from inexperienced jointers.

Interesting figures have been given by a number of Swedish towns in Figure No. 3, illustrating how PVC failures varied with age. Uneven settlement showed up quickly but material faults took a number of years to appear. The relationship between failure of pipelines according to age is shown in Figure No. 4 comparing PVC, grey iron and steel for the same towns.

7.4 Prestressed concrete

Many hundreds of miles of prestressed concrete cylinder and non-cylinder pipes have been laid throughout the world and operate satisfactorily. Failures are few and most of the incidents in recent years have been well investigated and documented so that knowledge of potential problems has increased, leading to improved manufacturing and laying techniques. In Jordan, Regina and Karachi failures were associated with both general and pitting corrosion resulting from action by chloride ions, while at Karachi sulphate ions were also present.

In Australia 250 km of prestressed concrete pipelines have been laid since 1944 and five aqueducts have failed in 11 incidents since 1968, a frequency of 0,006 failures/km/year. The major problem appears to have been the high permeability of the cement mortar coupled with inadequate thickness of cover and the presence of voids next to the steel/concrete interface. In the case of the Geehi River pipeline acidic soil and groundwater aggravated the situation and the corrosion was shown in some cases to result in hydrogen embrittlement of the prestressing cables. Failures on the Morgan-Whyalla No. 2—36 inch (900 km) diameter aqueduct were due to normal wasting of the steel. Chlorides in both ground water and water conveyed of up to 350 mg/l with an average of

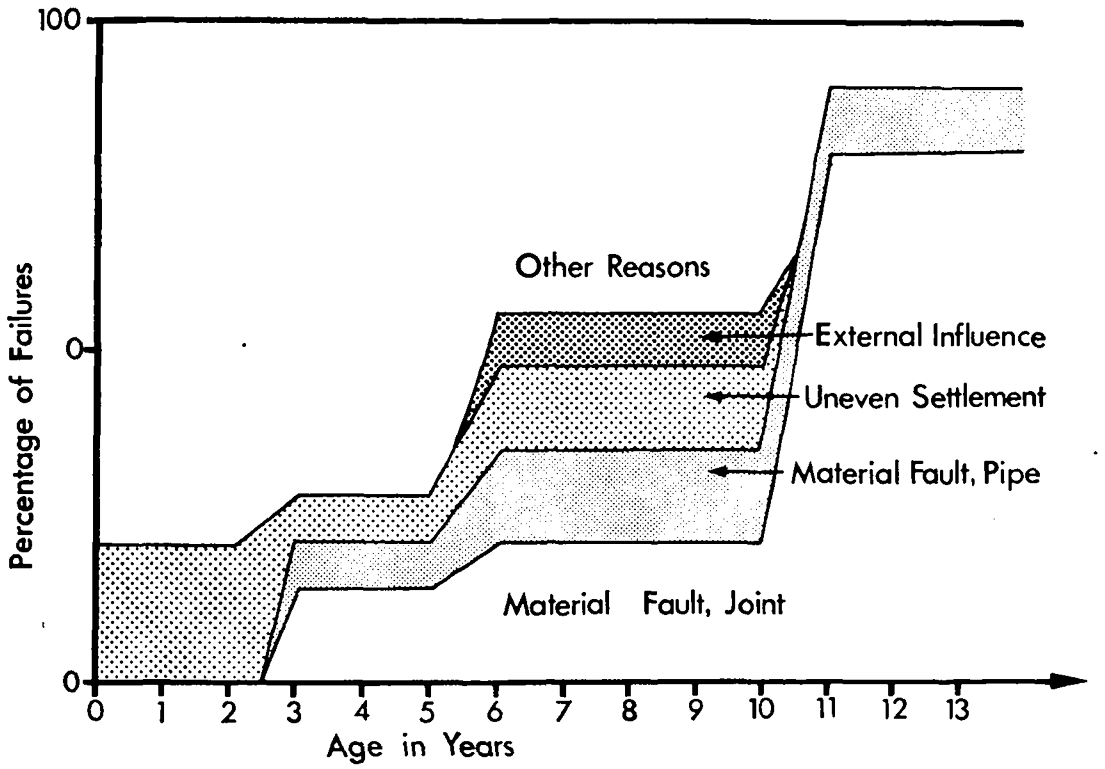

Figure 3—Fault/Age failure in PVC pipes.

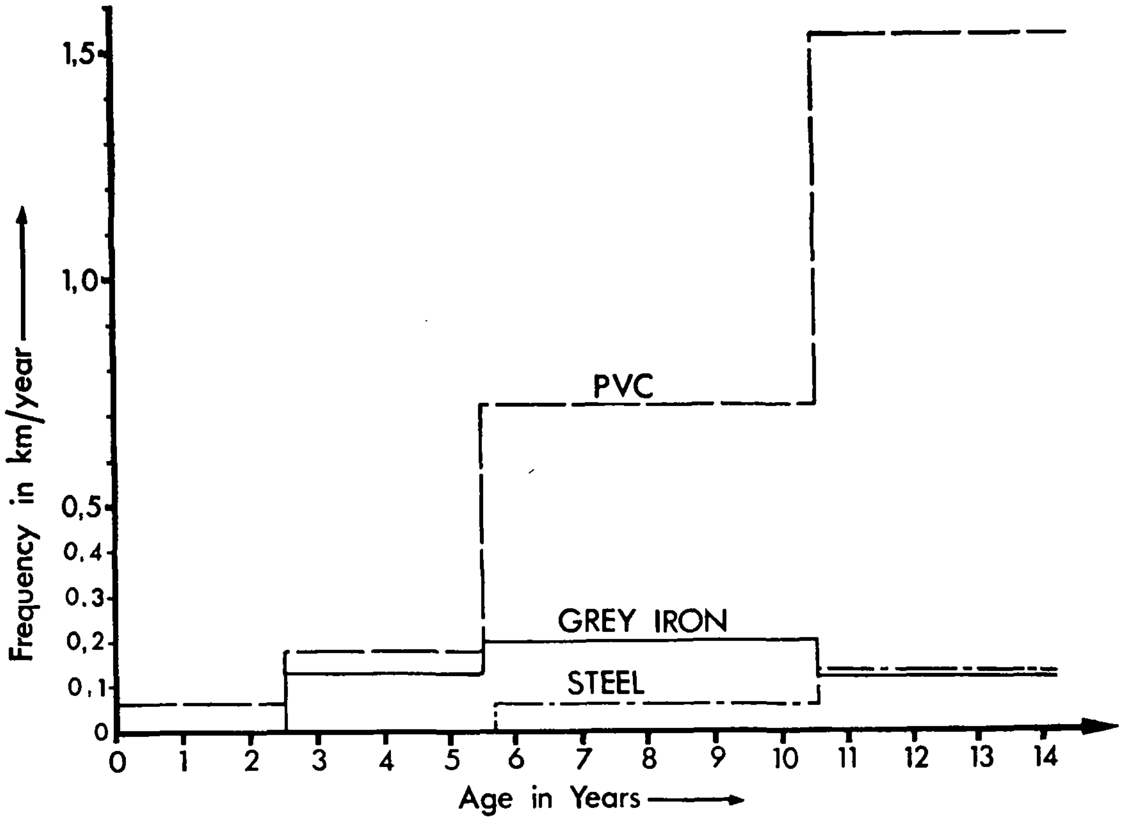

Figure 4—Failure in pipelines according to age.

135 mg/l were contributory factors. Complete replacement of 45 kilometres of Morgan-Whyalla and the repairs and lining of the Geehi pipeline is estimated to cost $ Aust. 15–16 000 000.

Dangers posed by the presence of aggressive ions in soil or groundwater are now fully appreciated and effective methods of cathodic protection or sealing the mortar surface are available, although such safeguards add to the cost of a pipeline. Epoxy tar coatings or similar forms of protection are now applied as standard practice in Australia when the soil conditions are known to be at all aggressive. In the Jordan Valley the mortar cover has been sealed with coal tar epoxy since 1963 and no failures have been reported since that date. Other alternatives worth considering are bitumen paint, coal tar enamel, plastic film wrapping or clay backfill. However, it must be recognised that the mechanics of how a soil can change from non-aggressive to aggressive does not appear to be fully understood and it would be wise to err on the side of caution.

7.5 Asbestos cement

Asbestos cement pipes are prone to beam breakage and care must be taken in preparing the trench. The bedding must be properly prepared and levelled. Hard high spots or stones must not be left projecting. Care must be taken not to drop pipes or to drop heavy objects onto a pipe laid in the trench. According to Anderson the most usual cause of failure is damage by impact sustained at some stage after manufacture, perhaps during off-loading or stringing out. It is recommended that the ends of each pipe are carefully inspected before laying. Correct positioning of the components of the joints are important. With push-in type joints the most common cause of failure is scuffing or tearing of the rubber seal. It has been said that stresses are set up in asbestos cement pipes as they absorb moisture after laying, but this rarely appears to cause failure.

It is important to protect cast iron detachable joints by running solid with bitumen if used in aggressive soils. Many asbestos cement pipelines suffered from this type of attack that did much to influence early users against asbestos cement as a pipe material. The construction and maturing of asbestos cement materials confers a built-in resistance to corrosive effects of soil. As a general guide asbestos cement pipes should be bitumen coated if magnesium sulphate is present greater than 2 000 mg/l as SO_3 in soil water or 0,8 % in soil.

Reports of 80 % of all breakages of asbestos cement pipes due to external factors such as traffic and adjacent excavations are significant. Other cases have been cited of failures due to rapid pressure surge.

7.6 Steel

Perfectly protected steel pipes are claimed to last for ever. This is no doubt true, but no protection can be said to be perfect. Most failures are due to corrosion leading to pinhole leakage or leakage at joints. Some failures have been dramatic. Three major circumferential breaks in a portion of the newly constructed Second Los Angeles Aqueduct have occurred since 1970, in each case due to brittle failure. Faults in design, specification and welding techniques contributed to the breaks.

Steel is used mainly for large diameter pipelines because for distribution systems in towns steel is prone to attack from stray currents and alternative materials have been found more satisfactory. Reports have been received of chemical and electro-chemical attack in aggressive soils due to failure of the other sheathing.

8 External protection of metal pipes

8.1 General

Some protection of iron pipes against possible corrosion in a neutral soil is afforded by dipping or spraying with coal tar or bituminous solution or enamel. Bitumen based sheathing usually reinforced with glass fibre predominates for the protection of steel pipes. For aggressive or very aggressive soils it is becoming more usual to employ inert materials such as asbestos cement, concrete or plastics rather than iron or steel, particularly in the smaller diameters. However, there are occasions when the properties of metal pipes are required in a mildly or sporadically corrosive route, in which case special precautions need to be taken. The cost of external protection must be taken into account when the decision is made to employ metal pipes.

8.2 Cathodic protection

A report in 1973 by the U.K. Institution of Water Engineers Research Panel No. 13 gave details of a comprehensive survey into the extent of cathodic protection. In the U.K. the number of sacrificial anode schemes had grown at a uniform rate for some time, while over the same period impressed current schemes had shown a much greater rate of increase. Difficulties had been experienced with electrical continuity and this highlighted the need for an adequate soil survey to determine the likely corrosive properties of the ground before deciding upon the type of joints to be used. Bonding across push-in type joints when laying a main is not particularly expensive, but the cost of excavation to bond after reinstatement is prohibitive, although for medium and large diameter pipes bonding may be carried out inside the pipe. There was a preference for sacrificial anodes to be used on medium diameter pipelines. It may be that the impressed current method is considered more reliable with a greater degree of control.

More than three quarters of the protected mains were steel, half used flexible joints and these were copper bonded generally using thermalite welding. Some mains with lead joints appeared to achieve satisfactory electrical continuity without bonding. Many ground beds for impressed current schemes used silicon iron as the anode material. For sacrificial anodes magnesium was generally used although zinc had been used in other countries. The spacing of anodes varied from 7 to 500 metres and the expectation of life was between 2 and 100 years with an average of 15 years. It has been calculated that cathodic protection added about 1% to the capital cost of a pipeline. There appeared to be little difference in cost between the two systems; impressed current was cheaper marginally to install but running costs, although small, were significantly variable.

8.3 Polyethylene sleeving

There is now the considerable experience of many hundreds of kilometres of iron pipe that have been protected successfully by the use of loose polyethylene sleeving. It has been suggested that a saving of 50% can be achieved over bitumen sheathing. As the wrapping takes place on site it has the advantage of not being subject to damage in transit. Minor damage does not impair the efficiency of protection but repairs are easy. Fixing may be by either PVC tape or string. Material specification standards exist in the U.S.A. and West Germany. In the U.K. the usual recommendation is for a natural polyethylene film with a single nominal thickness of 1 000 gauge i.e. 250×10^{-3} mm, a melt-flow index of 10 or less to BS 2782 and a density of 915 to 925 mg/ml. In France and the U.S.A. the thickness is specified at 200×10^{-3} mm, while in West Germany the limits are 200 to 250×10^{-3}. In Newcastle, England, there are pockets of highly aggressive soil capable of eating through iron pipes in less than two years. Since 1966 all iron pipes have been wrapped in polyethylene sheaths and to date no failure of any main so protected has been reported. The cost for a 200 mm pipe is £100 per kilometre run. It is interesting to note that in that area the water company uses blue polyethylene sleeving, while the gas board uses yellow as an aid to identification.

8.4 Zinc based coating

Metallised zinc plus a coal-tar varnish has been used on small diameter grey iron pipes in France since 1959 and for ductile since 1962. The protection is different from that given by the galvanic action of zinc in galvanised steel. The layer of zinc is believed to combine gradually with the ground waters in contact with the pipe to form insoluble mineral products of carbonate etc. which then being impermeable protect the pipe from further attack. This chemical reaction raises the pH locally slowing up the action of sulphate reducing bacteria. According to Brooks no failures have been reported over 15 years in a production of 70 000 km of pipes protected by zinc based coating.

9 Internal protection of metal pipes

Internal corrosion of metal pipes is less severe in terms of failure than external attack and is directly related to the quality of the water being carried. In the author's opinion the key to this problem is likely to be the adequate and proper control of the treatment given to the water before transmission whenever that is possible. Although internal corrosion may be less serious from the point of view of failure, it is more important as regards operational costs. Corrosion internally results in tuberculation formed from the products of corrosion and the additional roughness will lead to a reduced C value in the Hazen-Williams pipe friction formula. This means a reduced flow for the same headloss, or, if pumping head can be increased, then increased power consumption for the same flow. In distribution systems this will usually result in complaints of dirty or red water. If the normal protection of either hot coal tar dipping or bituminous paint is not satisfactory then it is usual to apply a centrifugal spun lining of dense cement. In some cases this will be a sulphate resisting cement. Cement linings must be cured under controlled conditions or by the application of a seal coat of bituminous material while still moist.

10 Cleaning pipelines

After laying, a pipeline to carry potable water will require cleaning. Provided that care and adequate supervision during construction has left the inside of the pipes reasonably free from foreign matter, then flushing followed by adequate sterilisation usually 50 mg/l for at least 24 hours will possibly give satisfactory bacteriological results after draining and refilling. More usual practice in the U.K. is to clean the main by passing a relatively soft grade swab made of expanded polyester foam throughout the length using the techniques developed by the former Water Research Association now the Water Research Centre, Medmenham.

Nowadays metal pipelines are protected against internal corrosion but many thousands of kilometres of cast iron pipe have been laid in the past without adequate protection. In time cleaning is required to restore the hydraulic flow rate and to obviate dirty water complaints. Debris and loose material may be removed by the swabbing techniques using hard grade pigs driven through the main by water pressure. Serious encrustation requires boring and flailing, drag scraping or hydraulic pressure cleaning. Recently in Birmingham, England a hydraulic jet blaster working at 20×10^3 kN/m^2 has been successfully used in small diameter mains. Up to 200 metres can be cleaned from a single excavation using a retro-jet nozzle which keeps the head concentric within the main and drives it forward. For larger sizes boring and scraping are more usual. Problems are primarily in negotiating awkward bends, valves, connections and the removal of debris. Scraped mains become encrusted again more rapidly than before and it is essential to provide a new lining or to accept a regular programme of scraping. As a result specialist contractors are usually employed to provide a complete scraping and relining service.

11 On site lining of pipes

Adequate cleaning and removal of debris is essential before lining. A number of patented processes are available including sprayed bitumen coatings and the electrophoretic deposition of bitumen. The widespread practice of applying cement mortar lining after cleaning varies with the diameter of the pipe. For small diameter pipes, particularly where there are connections, the cement is discharged centrifugally with a resultant dense but rough finish which is reputed to be less significant in terms of C value than would appear possible. With medium and large diameter pipes the cement is sprayed centrifugally and then finished to a smooth surface with rotary trowels. It is suggested that the thickness of the lining should be not less than 4 mm in small diameter pipes, 6 mm in medium, but 10 mm thickness for pipes above 450 mm diameter. It has proved difficult to obtain reliably comparable figures between the cost of relining and relaying water mains but a rough guide would be a saving of 50–70%.

In the U.S.A. and Africa the epoxy lining of steel pipelines on site have been reported as being completed successfully. The pipes are cleaned chemically leaving an etched surface which is then neutralised, phosphated, and coated at least twice with epoxy resin to a dry film thickness of at least 150 microns. An instance of damage to amide-cured epoxy linings has been reported where a certain amount of damage has been associated with making good field welds. Apparently it is difficult to avoid damage to thin coatings. A high standard of in situ cleaning is essential and may be the cause of certain failures at field welds.

12 Rubber joint seals

In many modern flexible joints the seal to prevent water leakage is formed by means of compressing a rubber ring. Leeflange reported in 1963 that in the Netherlands out of several hundred rings examined it had been found that over half had deteriorated on the surface in contact with potable water, but only a small percentage showed change on the soil side. Since that date wasting of rubbers from pipes have been reported from Australia, New Zealand and elsewhere. It seems likely that a change in rubber technology in the early 1960's made such rings more prone to attack by bacteria. However, the problem involves much more than biodeterioration; chemical attack and poor joint design may be contributory factors.

It has been suggested that the phenomenon may be more prevalent throughout the world than is generally believed by rubber and pipe manufacturers. The evidence so far suggests that the water supply industry should be on the alert and aware of the problem so that instances may be recorded which, together with current research investigations, will produce the evidence on which a decision may be based as to whether changes are necessary. With the present widespread use of rubber joint rings based on the premise that the life of the rubber will be compatible with the life of the pipe, it cannot be denied that the subject is not important, but how important will depend upon the result of the present investigations.

13 Glass reinforced plastic pipes

During the last few years reinforced plastics have begun to be used for large diameter pipelines. Advantages claimed are high strength to weight ratio, good resistance to corrosion, low resistance to flow and non-tainting of potable waters. There are two basic methods of manufacture, each using resin impregnated glass fibre filaments either wound in a controlled helix or placed in circumferential and longitudinal directions to give the required hoop and beam strength.

GRP pipe has the fundamental advantage over homogeneous materials in that the pipe can be made to suit exactly any particular duty. It is classed as a flexible conduit and care must be taken to ensure proper backfill and vertical deflection of the pipe which should not exceed 5% of the pipe diameter after proper reinstatement. Internal vacuum can introduce buckling and with shallow cover the possibility of vertical heaving. According to Greatorex and Chambers, GRP/RPM pipes are designed on the assumption that the internal pressure induces uniaxial hoop tensile stress which is taken solely by the hoop glass filaments.

In 1971 a pilot batch of twenty-five 750 mm diameter GRP/RPM pipes manufactured at Stanton and designed to operate at 12 bar internal pressure were laid experimentally at Bristol, England with different conditions of bedding or consolidation. Strain gauges were fitted and variations in vertical and horizontal dimensions measured. Some of the sidefill consisted of lumps of new wet clay that would never have been used under normal circumstances and at these points the pipe deflection was 14,7% compared with the more usual figure of up to 5% elsewhere. After completion of the tests the pipeline was connected into the normal distribution system and continues to perform satisfactorily.

Design to a stipulated wall section has tended to limit competitive pipe sizes normally to those over 600 mm diameter with current pressure class limitation of about 4 or 6 bar. Smooth clean internal bores indicate low hydraulic friction and Hazen-Williams coefficients of 145 to 155 are reported. It is claimed that there is no leaching of noxious substances or fostering of yeasts, fungi and bacterial growths that might impair the quality of transported water and the pipes are virtually inert to all except the most acidic soils. In common with most plastic materials GRP pipes are time dependant and long term tests have been undertaken to ensure that adequate strength will be maintained for a service life presently anticipated as being up to 50 years. Reports have been received of mixed success in operation of GRP in the U.S.A., U.K., West Germany and the Middle East.

Appendix A
Acknowledgements

The following water authorities, associations, research organisations and companies have provided a great deal of data and information and their assistance is acknowledged with thanks.

Anglian Water Authority, England
Antwerp Waterworks, Belgium
Atlanta Bureau of Water, Georgia, U.S.A.
Austrian National Committee IWSA, Vienna
Barcelona Water Company, Spain
Bristol Waterworks Company, England

British Steel Corporation, England
Brussels Water Company, Belgium
City University, London, England
Concrete Pipe Association, England
Construction Industry Research and Information Association, England
Copenhagen Water Supply, Denmark
Dallas Water Utilities, Texas, U.S.A.
DVGW, Gas and Waterworks Association, Germany
General Descaling Co. Ltd, England
Gentofte Water Supply, Denmark
Gothenburg Water and Sewage Works, Sweden
Groningen Waterworks, Netherlands
Humes Ltd, Melbourne, Australia
Hydraulic Research Station, England
Indianapolis Water Company, Indiana, U.S.A.
Milwaukee Water Works, Wisconsin, U.S.A.
Newcastle and Gateshead Water Company, England
North West Water Authority, England
Northumbrian Water Authority, England
Pipelines Industries Guild, England
Pitometer Associates, New York, U.S.A.
Rand Water Board, South Africa
Sentab Pressure Pipe Consortium, Sweden
Severn-Trent Water Authority, England
South West Water Authority, England
TAC Constructions Materials Ltd, England
Tate Pipe Lining Processes Ltd, England
Thames Water Authority, England
Transport & Road Research Laboratory, England
Warsaw Polytechnic, Poland
Warsaw Water Supply, Poland
Water Research Centre, England
Welsh National Water Development Authority, Wales
Yorkshire Water Authority, England

Appendix B
Standards and codes of practice

International Standards

ISO/R 13	Cast iron pipes for pressure main lines
ISO/R 160	Asbestos cement pressure pipes
ISO/R 559	Steel pipes for gas, water and sewage
ISO/R 1165	Plastic pipes for transport of fluids—uPVC pipes
ISO/R 2531	Ductile iron pipes for pressure pipelines
ISO/R 2785	Guide to the selection of asbestos cement pipes subject to external loads with or without internal pressure

United Kingdom

BS 486	Asbestos cement pressure pipes
BS 3505	uPVC pipe for cold water services
BS 3601	Steel pipes and tubes for pressure purposes
BS 4622	Grey iron pipes and fittings
BS 4625	Prestressed concrete pressure pipes
BS 4772	Ductile iron pipes and fittings
CP 310	Water supply
CP 312	Plastics pipework (thermoplastics material)
CP 2010	Part 1 Installations of pipelines in land
	Part 2 Design and construction of steel pipelines
	Part 3 Design and construction of iron pipelines
	Part 4 Design and construction of asbestos cement pipelines
	Part 5 Design and construction of prestressed concrete pipelines

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Résumé

Ce rapport a été rédigé grâce à l'aide d'un grand nombre de services d'eau, sociétés et personnes. Il couvre les tuyaux et conduites allant de 100 à 1 000 mm de diamètre interne.

Etude des conduites

Les conduites en amiante-ciment, la fonte grise et le béton précontraint peuvent être considérées comme des cylindres rigides dont l'aptitude à supporter une charge extérieure est en relation directe avec la rigidité annulaire de la structure, en tenant compte du matériau de remblaiement, de sa compaction et de la forme de la tranchée. Un choix personnel de normes pour le matériau excavé et choisi est donné.

Les tuyaux en fonte ductile, acier et plastique ont une faible rigidité inhérente et s'affaissent sous les charges verticales. L'emploi de ces matériaux flexibles signifie que l'on doit prendre en considération l'effet de support du sol. Les charges externes exigent l'analyse de la déflexion, de la courbure, de la compression et du flambage. Il est nécessaire de vérifier les corrosions d'efforts, les forces cycliques et les propriétés d'impact.

La détermination des conditions physiques et chimiques du sol est importante. Le développement des joints flexibles est retracé depuis les joints rigides en passant par les joints semi-rigides. Référence est faite aux choix disponibles dans la sélection des formules pour déterminer la perte de charge hydraulique des conduites.

Expériences d'emploi des divers materiaux

On a récemment exprimé des inquitudes pour les problèmes de goût et de toxicité pouvant résulter de l'emploi de certains matériaux. Il existe maintenant des tests pour déterminer l'aptitude à donner à l'eau goût, odeur, couleur, turbidité, ou toxicité ou à favoriser un développement microbien.

Une enquête récente a montré que la fonte grise avait la plus longue espérance de vie, suivie par le béton précontraint. Des réserves ont été exprimées sur l'amiante ciment et l'acier posés en sols corrosifs, tandis que la fonte ductile et les plastiques, étant des matériaux relativement nouveaux, étaient regardés avec une certaine suspicion.

En Grande-Bretagne, la fonte ductile remplace la fonte grise, particulièrement dans les petites tailles. Pour les conduites principales de grande taille. on utilise des tuyaux d'acier revêtus de bitume, à joints soudés. On utilise des tuyaux en amiante-ciment en sols agressifs malgré les doutes sur leur espérance de vie. Le PVC est populaire en régions rurales.

L'expérience mondiale varie quand on compare les différents matériaux de conduites, ce qui n'est pas surprenant car les conditions du sol, les modes de pose, la profondeur, le trafic et les températures varient d'un site à l'autre.

Dans un rapport de Londres sur les conduites en fonte grise, le principal type de fracture était la rupture transversale. On ne pense pas que l'âge soit une cause primordiale de rupture. Dans les grands diamètres, le nombre de fractures diminuait et les fentes logitudinales devenaient prévalentes. Il semble qu'il y ait une relation entre la température de l'air et les fractures, peut-être en raison de mouvements différentiels du sol.

Il est difficile de dire des généralités sur les conduites en plastique en raison des caractéristiques des divers matériaux. L'expérience passée a été décevante en ce que les ruptures ont été plus fréquentes que prévues, même avec un matériau neuf. L'amélioration des techniques d'extrusion s'est réalisée avec le temps et les conduites en plastique sont maintenant largement utilisées en de nombreux pays.

Les ruptures avec le béton précontraint ont été relativement rares et la plupart des incidents ont été complètement examinés et ont fait l'objet de rapports. Le danger posé par la présence d'ions agressifs dans le sol est maintenant bien apprécié, encore que le mécanisme de la transformation de non-agressif à agressif puisse ne pas être complètement élucidé.

Un pose généralement les conduites en amiante-ciment dans les sols corrosifs. C'est un matériau cassant, dont la manipulation doit être faite avec soin, mais s'il est convenablement posé et non soumis à des influences extérieures, il semble très satisfaisant.

L'acier durera pour toujours s'il est parfaitement protégé, mais c'est un idéal. En pratique des ruptures mineures sont apparues en raison de défauts dans le revêtement extérieur.

Un protège les tuyaux métalliques extérieurement en les trempant dans le goudront ou le bitume, ou en pulvérisant ces produits, en les revêtant, en les plaçant dans un manchon de polythène, en pulvérisant du zinc métallique, ou en installant des anodes réactives ou par soutirage de courant. La protection intérieure, quand on l'utilise, est généralement faite de ciment centrifugé. Le nettoyage des conduites anciennes se fait par cylindres de plastique ou par grattage. Il est nécessaire, après grattage, de refaire le revêtement intérieur pour éviter la réincrustation.

Des préoccupations ont été exprimées sur la tenue des joints en caoutchouc et des recherches sont nécessaires à ce sujet.

L'introduction de plastiques renforcés à la fibre de verre est mentionnée, mais l'expérience actuelle est limitée.

Pressure control in distribution systems

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1 Introduction

The following paper is merely a contribution to an ambitious subject. Experiences are drawn from operational aspects of the water supply to the large city of Stuttgart which has differences in elevation of more than 300 m. In normal circumstances, 90% of the city's water supply is imported in two long-distance supply lines, whose delivery points are on the highest peripheral locations of the supply area, with the result that a good deal of the water has to be reduced step by step to the pressure of the respective supply zone. The city's own waterworks, which are in lower-lying locations, serve to cover peak consumption, the supply of several large-scale users, and as a reserve supply. This water has to be pumped to the higher-lying consumption points.

In treating this subject, both technical and economic aspects of the supply pressure, as well as legal ones are considered.

2 Supply pressure

The overriding principle of any water supply is to guarantee the drinking water requirements at every location within the supply area and to ensure that it is at all times of sufficient quantity, best possible quality and adequate supply pressure (4).

By supply pressure is meant the static head p/γ for the consumer $\times$ with the geodetic height z of the highest consumption point. To transport the required amount of fluid to the consumer, the resistance caused by fluid friction has to be overcome, i.e. hydraulic energy is lost, which is denoted as loss of head. The energy balance is represented by the well-known equation

$$p/\gamma + z + h_v = H$$

(Velocity head disregarded)

h = Difference in height between two points
z = Height related to sea level
p = Normal force exerted by water on the surface unit
γ = Specific gravity
h_v = Pipe friction loss

A water supply consists of a collection of elemental systems; for treatment, boosting, pressure reduction, transport, storage, measuring and control. These are joined to supply the required amount of water at the necessary pressure.

The meaning of pressure control in this respect is the influence on and control of all elements which guarantee the supply: irrespective of the height of the consumption point.

The necessary supply pressure is not, however, accurately defined in Germany. It should also be taken into account that a pressure once set cannot be kept at the desired level without control.

There are, however, points of reference for the pressure limits:

1. Disregarding negative pressures (vacuums), which should not appear in any drinking water supply, because of the danger of germs and other extraneous materials penetrating into the pipe network, the lower pressure range is defined by the "minimum flow pressure" for the highest outlet point of a building. The minimum flow pressure amounts to 0,1 bar (tap)–2,5 bar (gas operated flow heater).
2. The highest permissible operating pressure is defined as follows in a German standard (DIN 1988): "Pipe and ancillaries should be designed for at least 10 bar internal pressure, insofar as higher operating pressures do not require larger dimensions."

Exceptions to this are boilers, which are frequently only designed for an internal pressure of 6 bar and which then mostly require fittings to reduce a higher pressure to 6 bar.

Whilst an operating pressure in the supply network of up to 10 bar is completely permissible, economic reasons and noise protection dictate against the supply of water at a pressure in excess of 6 bar. Generally, pressures between 2 and 8 bar are the rule in the supply network.

3 Pressure control

The following considerations are features of pressure control:

1. Pressure control is the requirement for *technical operation.*
2. Pressure control enables an easy to view, *economic operation.*
3. Pressure control in the sense of the *reduction of excess supply pressures* has advantages for:
   —the life of pipelines and connections,
   —the protection of consumer devices and installations,
   —the reduction of losses from leakages.
   and water is consumed more sparingly.
4. Pressure control is evidence of the correct and regular operation of the water supply, e.g. also from a legal point of view.

3.1 Pressure control in the technical operation

The design basis for much water supply equipment is the fluctuating demand for water. The flow or delivery rate results from this. The flow is significantly determined by consumption. Consumption, on the other hand, shows marked alterations, despite statistical superimposition of all individual consumptions, especially when comparing shorter intervals of time. To be able to cover peak consumption at all times, it is, however, not necessary or economic to construct the system to the absolute peak. The interval of time to be used for design is dependent on the type of system and topographic or other conditions [18]. The capacity of the various parts of the system is hence better exploited and economy increased. Occasional slight, short-term pressure reductions are quite permissible.

The ratio of flow and pressure drop, using the annual duration curve for the peak year 1964 as an example, as well as the temporary pressure fluctuations, which are always present in the network, are presented in Figs. 1 and 2.

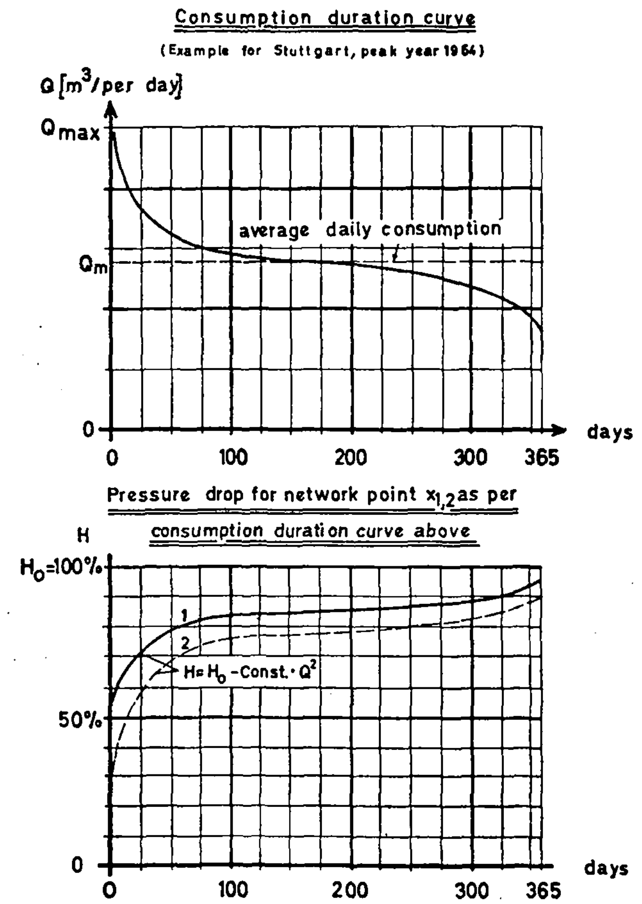


Figure 1

Large pressure fluctuations can have the following causes:

—Start up or shutdown of connection, treating and delivery systems
—Filling and emptying of lines or flushing
—Vibration of control or cut-off devices (e.g. float valves)
—Pulsations in the pump pressure lines
—Incorrect layout of systems' parts
—Fracturing of drive or control elements
—Consumption fluctuations
—Drawing off water for fire-fighting
—Pipe fractures
—Accumulations of air in lines (high points)
—Power failure

3.2 Pressure changing systems

All equipment, with which or via which a specified pressure is set or maintained, is pressure changing equipment, as for example

—Pumps and turbines
—Control valves, such as cylindrical piston valves, pressure relief valves
—Other fittings and
—Tanks

Problems with pressure reducing and booster systems, for operating conditions of larger lines and in the network respectively are to be touched upon in the following.

3.2.1 Pressure relief in feed lines

These are defined as lines with an internal diameter of approx. 600 mm and more, which feed into a tank. With this procedure, a considerable energy potential has very often to be reduced.

Both turbines and special control elements are used to transform pressure energy into velocity energy. An economic assessment should decide which solution is preferable in individual cases. The cylindrical piston valve to a special design is discussed as an example of a control element.

3.2.1.1 Cylindrical piston valves

Fittings which are to be used as a control element should meet the following requirements:

(a) Good control behaviour, i.e. the opening and closing procedures in the fittings are tuned to the pipeline, the respective flow rate and the opening or closing time such that water hammer only occurs within permissible limits. The shortest closing time can only be reached with constant lag for a specified water hammer increase, i.e. by steady velocity change in the pipeline. This condition should only be produced in the control element itself. In the case of supply lines with greater pipe friction losses use is made of an adjustment with a non-linear control unit opening function, i.e. a time or path dependent function.

(b) continuous operation over a broad opening range (stroke ratio s/s_0) without cavitation and with the least possible vibration.

(c) Trickle-tight seal.

While the conditions of (a) and (c) can be met most satisfactorily with a cylindrical piston valve (also called cylindrical piston gate) with a normal plunger, and (a) furthermore with a stepped law of closure, (b) presents problems if a relatively low local counter pressure is present with a tank of max. 4–5 m water level. The evaporation pressure of the liquid is partly attained as a result of the velocity head. Bursting of the vapour bubbles is accompanied by considerable noise generation. Pulsating of the interference leads to impermissible vibrations in the various parts of the system. Material destruction is the consequence.

These phenomena, which can considerably hinder the operation, can be reduced under certain conditions using a fitting provided with a flow-promoting shut-off device, and which for instance has orifice plates [14].

At very high pressure energies orifice plate elements are also used independently of a fitting, thus becoming non-controllable energy converters. Several elements such as these can be connected in series to form so-called cascade throttles.

With the cylindrical piston valve with orifice plates a cylinder with a certain number of concentrically arranged bores is mounted on the axially displaceable plunger (see Fig. 3).

In the closed position the piston sits on the sealing ledge. When opening, the cylinder first opens up, exposing in each case a discharge section corresponding to the sum of the opened orifice sections.

The liquid flowing from the upstream pressure side passes through the bores in a large number of thin jets, which disperse from outer to inner into the cylinder. The jets, which hit each other concentrically at great velocity, are decelerated. In doing so, part of the velocity energy is converted into thermal energy, deformation energy and sonic energy.

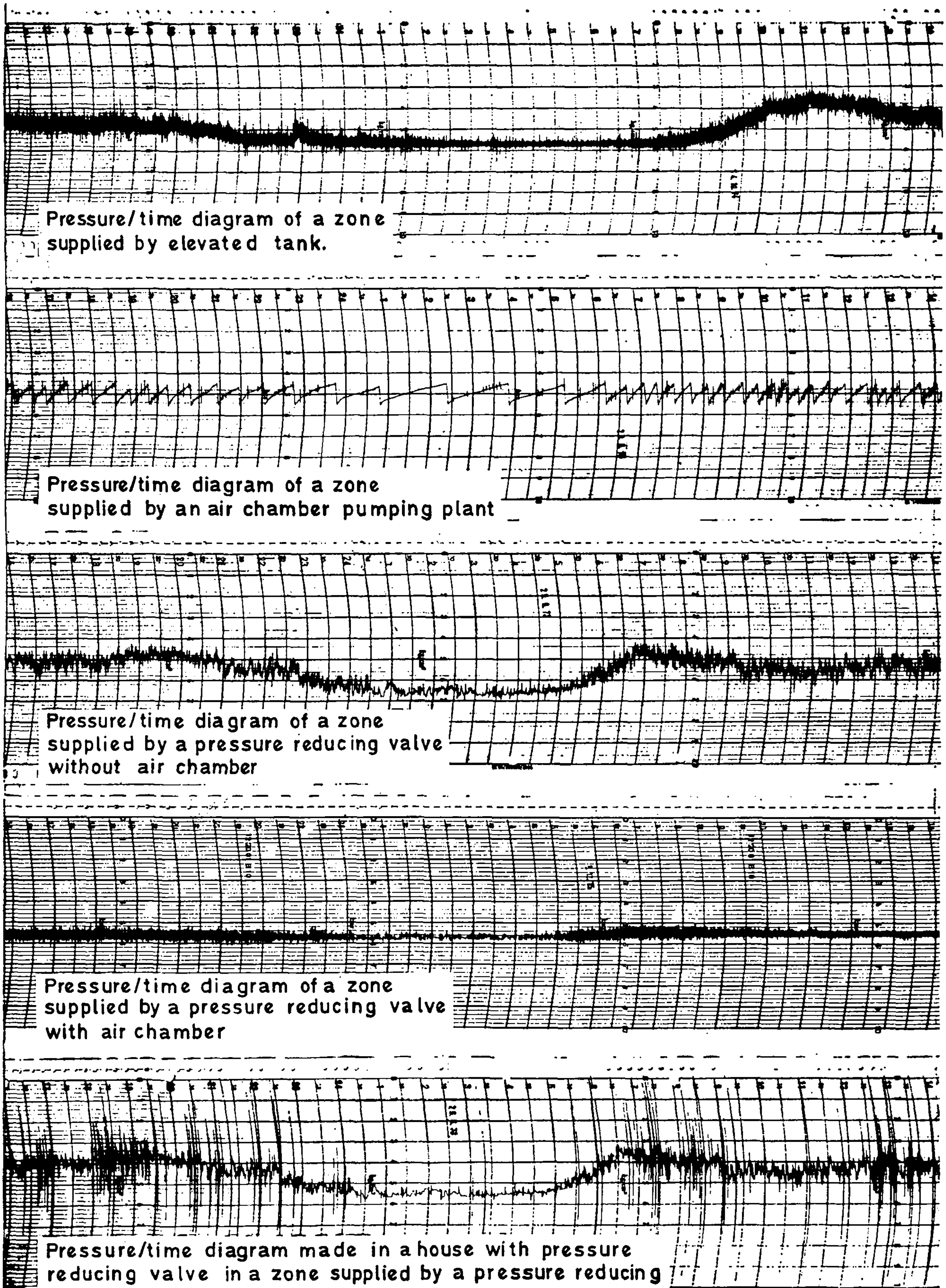

Pressure/time diagram of a zone supplied by elevated tank.
Pressure/time diagram of a zone supplied by an air chamber pumping plant
Pressure/time diagram of a zone supplied by a pressure reducing valve without air chamber
Pressure/time diagram of a zone supplied by a pressure reducing valve with air chamber
Pressure/time diagram made in a house with pressure reducing valve in a zone supplied by a pressure reducing valve without air chamber

Figure 2

Cylindrical piston valve

Cylindrical piston valve with orifice plate

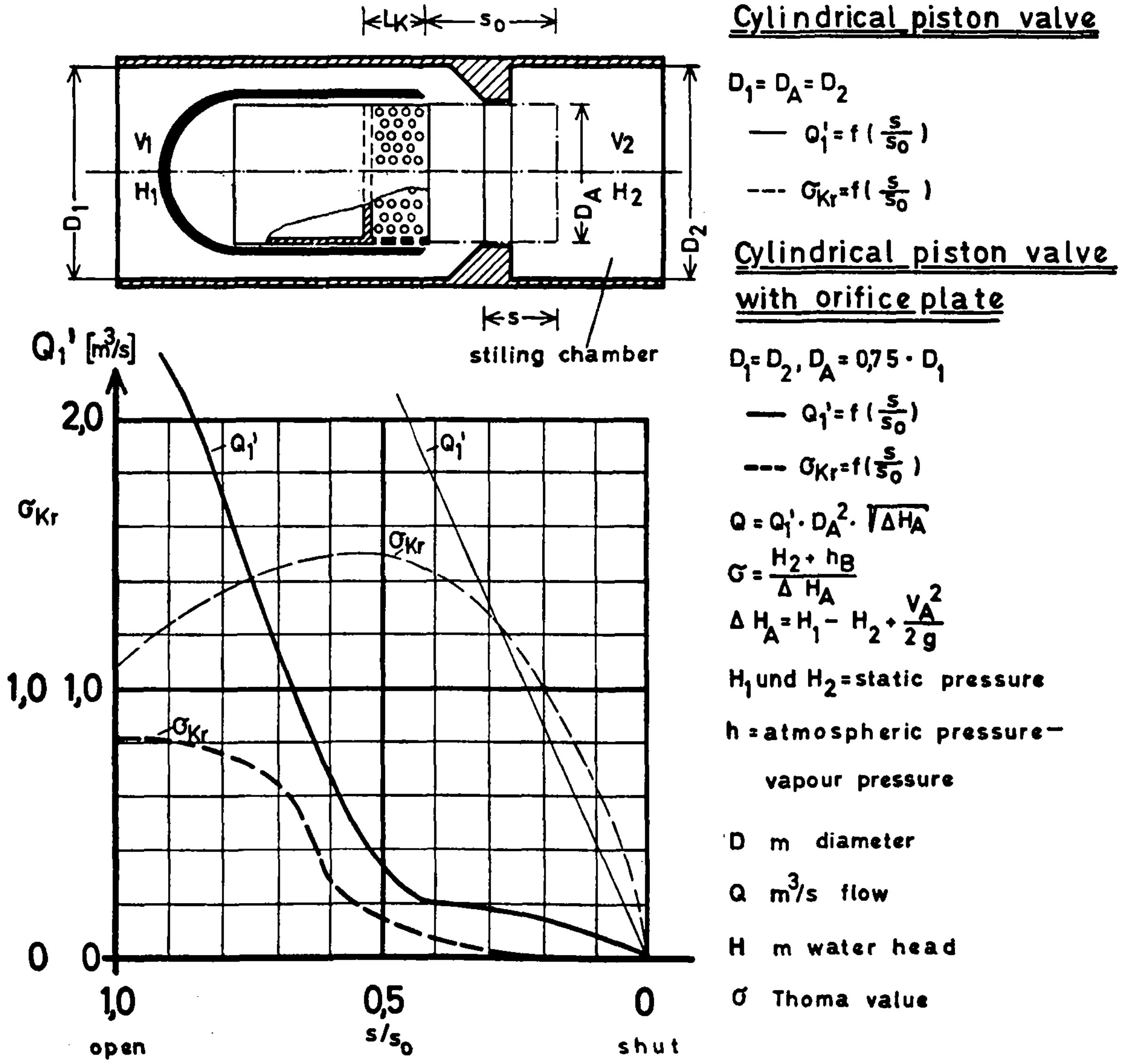


Figure 3

If the chosen gate is large enough, the whole operating quantity can be passed through the orifice plate cylinder. Otherwise, the cylinder has to be opened far enough for the normal cylindrical cross-section of the valve to be opened via the last part of the flow path.

Which size of valve is to be preferred in the planning should be based on the hydraulic system data and is not least a question of costs.

Should a permanent operation of the valve be necessary or possible also in the operating ranges with partial opening, the question of whether cavitation might in fact occur in the operating range should be clarified. In this respect the characteristic value σ (Thoma value) should be considered. The continuous operating range is defined by the cavitation factor σ.

The σ value for the system is defined by:

$$\sigma_{system} = \frac{H_2 + h_B}{H_1 - H_2 + \frac{C_A^2}{2g}} \geqslant \sigma_{perm.}$$

The condition that no cavitation occurs at the point of least pressure is as follows:

$$H_2 \geqslant \sigma_{perm.} \times \Delta H_A - h_B$$

$$\Delta H_A = H_1 - H_2 + \frac{C_A^2}{2g}$$

H_1, H_2 = Static pressure upstream of valve or end of stilling basin, mWH

C, C_A = Velocity, end of orifice plate cylinder, m/s

σ = Thoma value

h_B = Air pressure minus evaporation pressure, mWH

g = Acceleration due to gravity m/s^2

In Fig. 3, the flow ratio Q_1' depending on the stroke ratio s/s_0 for $\Delta H_A = 1$ m, $D_A = 1$ m is drawn both for a valve in normal design and for one with orifice plate cylinder. The most favourable path of the limit curve $\sigma_{critical}$ can be seen for the cylindrical piston valve with orifice plate.

"Goldberg" plant (Diagramatic view Fig 4)

The Goldberg tank with only 490 m^3 content, serving as pressure controller and corresponding tank to a larger tank six kilometres away, supplies a large suburb of a town with approx. 125 000 inhabitants. A cylindrical piston valve (nominal dia. 300 mm) with orifice plate operates as a reducing valve in the inlet to the tank, as described above. A second valve serves as a stand-by or operates alternately. The max. flow rate for the control fittings was specified as $Q_{max} = 0{,}834\ m^3/s$ with a static pressure difference of $H_{geo} = 100$ m and a max. pressure loss in the 2 km long feed line (nominal dia. 800/650 mm) of $h_v = 8{,}0$ m water head. The outlet goes into an ante-chamber with a water level of 4 m. The latter is connected to the main chamber by an overflow. The valve is controlled according to the water level in the main chamber, which should neither empty nor overflow. The closing time is fixed at 50 secs, with a max. permissible water hammer of $\Delta H_A = \pm 15$ m water head.

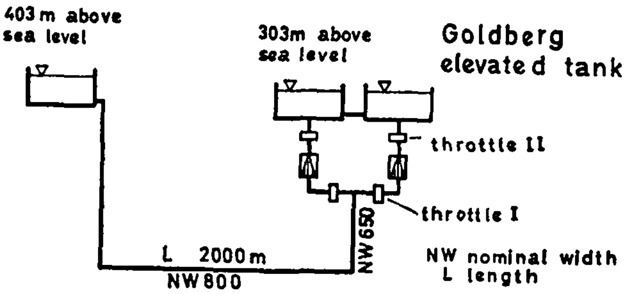


Figure 4

With a static counter pressure of only 4 m water head, relatively large ΔH_A values result, with the result that with correspondingly small σ values for the system, the σ value falls below $\sigma_{critical}$ at a certain partial opening range s/s_0. As the pressure reducer must operate cavitation-free in all operating ranges, it has been constructed as a two-stage design with upstream and downstream throttles. It has been shown that after start-up the system's σ value should be improved, e.g. by installation of a fixed orifice plate.

3.2.1.2 Turbines

Turbines can also operate as pressure reducers. The reduction of the pressure energy potential can in this case be used economically. Turbines, however, require more investment and higher operating costs. Moreoever, their control range is smaller.

The real task for turbines in a water supply system is pressure reduction. The electrical energy produced is in this sense a "waste product". Even so a waterworks' turbine cannot be operated with other than normal economy.

When specifying the size and number of turbines to be used, the starting point is the development of the water requirement over a certain period of time. The average consumptions $Q_{m\,1,2}$ ($Q_{m\,2}$ after 20 years) and the summer peaks resulting from them $Q_{max\,1,2}$ are considered (where $Q_{m\,1}$ and $Q_{max\,1}$ are current mean and maximum consumptions, and suffix 2 refers to a time 20 years later).

The result of an economic comparison derived therefrom is that a turbine, despite less use and lower efficiency, should be designed as per $Q_{max\,2}$ (see Fig. 5, and for profit and cost comparison curve Fig. 8).

A turbine fitted as a reserve unit does not operate economically. The reserve unit should therefore be designed as a control fitting. In Fig. 6, units with equal capacity are compared in each case ($Q_{max\,2}$ from Fig. 5).

Remote monitoring and control of turbines requires additional investment, the capital costs of which in fact correspond to the saving on personnel costs involved. Turbines should therefore be used as far as possible in systems in which operating personnel are used already (see Fig. 7 in comparison to Fig. 6),

Control fittings as a reserve unit cannot be dispensed with under any circumstances, as shutdown times for repairs cannot be disregarded. In a great number of applications, control fittings will remain the only suitable unit.

Profitability comparison between turbine systems with differing design

Turbines designed for	$Q_{m\,1}$ 417 l/s	$Q_{m\,2}$ 624 l/s	$Q_{max\,1}$ 742 l/s	$Q_{max\,2}$ 1 125 l/s
Investment costs per turbine system	DM 500 000	DM 600 000	DM 650 000	DM 700 000
Interest plus 12% depreciation per year	60 000	72 000	78 000	84 000
Operating costs/year	18 000	18 000	20 000	20 000
Maintenance costs/year	2 500	2 500	3 000	3 000
Total costs/year	80 500	92 500	101 000	107 000
Credit from power sales	209 320	256 492	268 132	283 758
Profit/year	128 820	163 992	167 132	176 758

Invoicing of electricity costs for day tariff = 0,09 DM/kwh
for night tariff = 0,05 DM/kwh

Night tariff applies between 21.00 and 06.00

Fig. 5

Profitability comparisons between turbine systems and pressure reducing systems

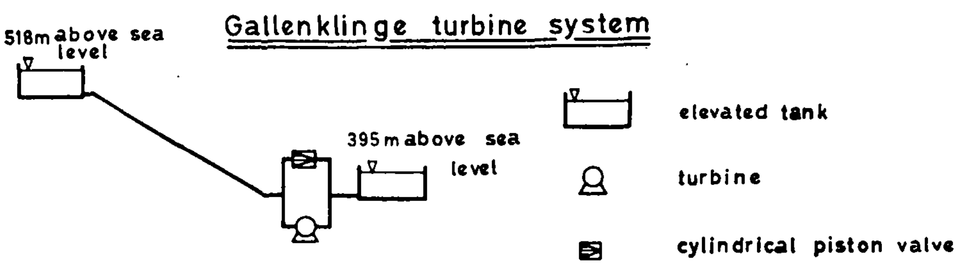
	Waterworks operation			Power station operation
	2 turbines	1 turbine+ 1 pressure reducer	2 pressure reducers	1 turbine
Investment costs	DM 1 400 000	DM 700 000	DM 250 000	DM 650 000
Interest+12% depreciation	168 000	84 000	30 000	78 000
Operating costs/year	40 000	25 000	15 000	20 000
Maintenance costs/year	6 000	4 000	2 000	3 000
Total costs/year	214 000	113 000	47 000	101 000
Credit from power sales	160 000	160 000	—	415 000
Residual expenditures	54 000	—	47 000	—
Profit/year	—	47 000	—	314 000

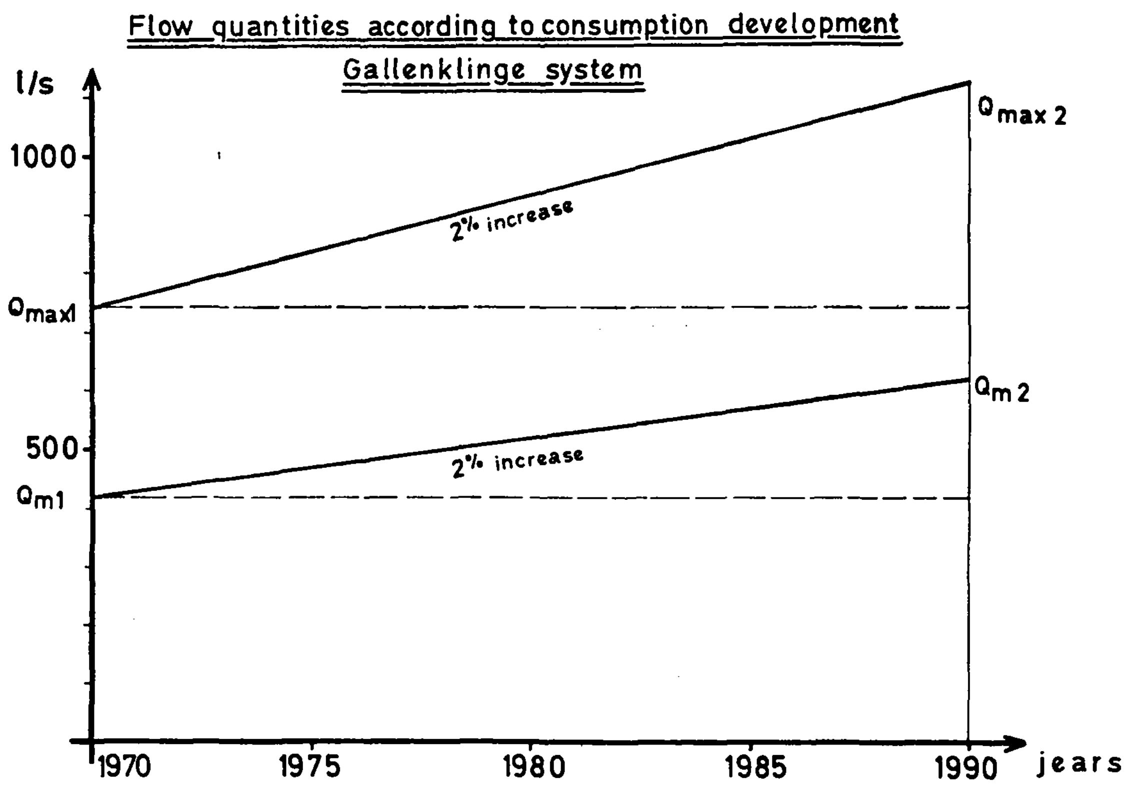
Fig. 6

Profitability comparison between turbine and pressure reducing systems with remote control through a central control station

	Waterworks operation			Power station operation
	2 turbines	1 turbine+ 1 pressure reducer	2 pressure reducers	1 turbine
Investment costs/year	DM 1 500 000	DM 820 000	DM 290 000	DM 700 000
Interest+12% depreciation/year	180 000	98 400	34 000	84 000
Operating costs/year	20 000	12 500	5 000	20 000
Maintenance costs/year	10 000	7 000	4 000	10 000
Total costs/year	210 000	117 900	43 000	114 000

Fig. 7


Gallenklinge turbine system
518m above sea level
395m above sea level
elevated tank
turbine
cylindrical piston valve


Flow quantities according to consumption development
Gallenklinge system
l/s
1000
500
Qmax1
Qm1
2% increase
2% increase
Qmax 2
Qm 2
1970
1975
1980
1985
1990
jears

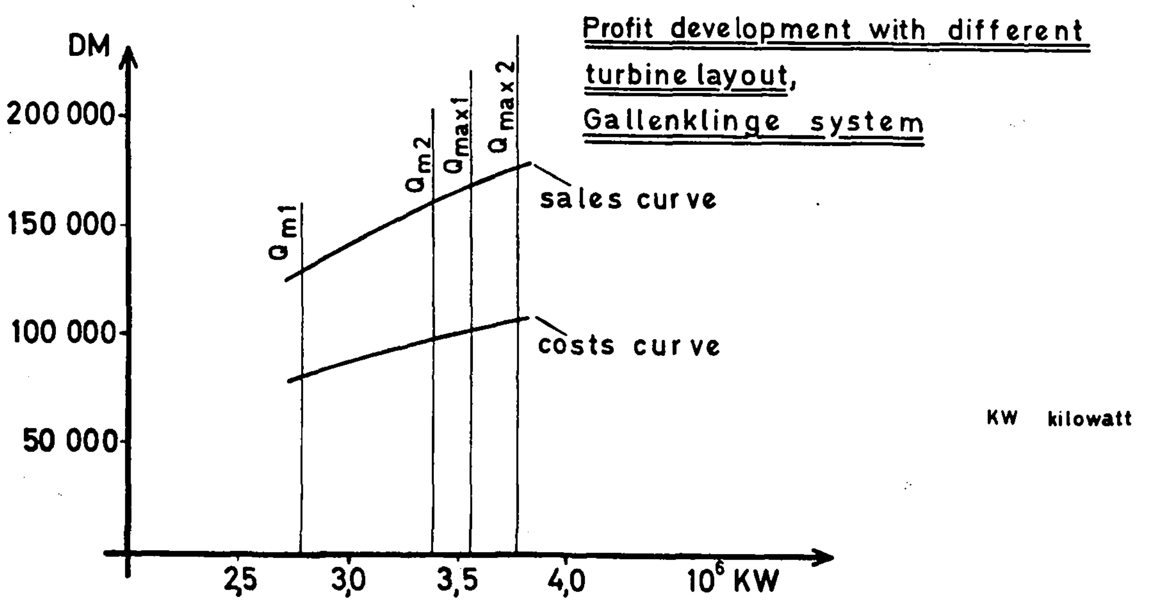

Profit development with different turbine layout,
Gallenklinge system
DM
200 000
150 000
100 000
50 000
Qm1
Qm2
Qmax1
Qmax 2
sales curve
costs curve
KW kilowatt
2,5
3,0
3,5
4,0
10⁶ KW

Figure 8

3.2.1.3 Pressure reducing valves in networks

It is proposed to deal only with pressure reducing valves smaller than those described in 3.2.1.1 which control automatically, i.e. without extraneous auxiliary energy. The auxiliary energy is removed from the controlled system. The control always takes place steady proportionally or proportionally integrally according to the selected downstream pressure.

Pressure gradient and flow rate are the criteria for selection of appropriate size.

The first uncontrolled pressure reducing valves were built as early as at the turn of the century. Controlled valves have been well-known in Germany for 25 years.

The most important demands on pressure reducing valves are [10, 11]:

1. The selected pressure should remain as constant as possible with fluctuating consumption and changing upstream pressures.
2. Tight seal at zero consumption and even over a long operating period (the pressure should not be transmitted).
3. Operational safety.
4. Inexpensive maintenance.

Pressure reducing valves are used particularly downstream of long-distance supply system branches and in municipal networks with large differences in elevation, either for the supply of pressure zones, insofar as the water is available on a higher level, or for the reduction of pressure in individual buildings as a house pressure reducer.

Whilst pressure reducers have their limitations, they also have the following advantages:

—They are relatively cheap and require no costly structures.

—They are self-controlling and can be re-adjusted within certain pressure limits.

—Under certain pre-conditions, they offer the possibility of a small pressure division, where there are large differences in elevation within the supply area.

—They allow pressure compensation to overloaded or distant network components, which are supplied via water tanks.

—They protect elements of the system against excess pressure and as such also serve to cut down on noise.

At present the pressure reducing valve does not yet offer the operational safety of a tank, which is exposed to atmospheric pressure and whose storage effect is a pre-requisite for a continuous supply. The pressure reducing valve is an essential aid for the applications considered above, under the following conditions:

—Feeding in against closed network only wherever short-term transmission of the upstream pressure does not give grounds for any serious damage (otherwise additional safeguarding by a safety valve, insofar as the latter can still draw off the water up to a certain upstream pressure, in case the valve should fail),

—Use of pressure control tanks with long feed lines,

—Regular maintenance,

—Shutdown times are not too long,

—No overdimensioning.

The pressure reducing valve with external, auxiliary energy has even better control characteristics and can be controlled according to the pressure profile of a distant network point. The operational safety of the pressure reducing valve with external energy is not, however, increased in comparison with that without external energy.

The possibility of failure of the pressure reducing valve must be considered and corresponding precautions taken. A disturbance in this case means more than just shutdown. It means possible impermissible pressure rise in the pipe network and household installations. If this is in excess of the stress limit of the pipelines and consumer devices corresponding damage will occur.

3.2.2.1 Pumps with long pressure lines

With the operation of pumps in long pressure lines, particular attention must be paid to the non-steady flow conditions. These include start up and shutdown of the pumps, and also control of them by means of changing the pump speed or by actuation of a control valve. If the speed decelerates rapidly, and as the delivery head of the pumps decreases as the square of the speed of rotation, an oscillation is caused in the water column. With longer lines, the full, direct, negative water hammer can take effect, as the discharge within the reflection time of the pressure waves returns to zero [7]. The effects of the velocity change are obviously different for individual systems, as also are the measures which should be taken to keep the pressure change within permissible limits. In any case, the potential damage from underdimensioning should be avoided just as much as the excessive system costs due to overdimensioning.

The permissible pressure change in the pipeline is derived from two conditions:

Tear-off condition: the pressure line should not drop below the height of the pipeline at any point.

Max. pressure condition: max. pressure head to be expected should not exceed the permissible operating pressure (rated pressure) of the line.

An assessment of whether there is a danger of water hammer can be derived from the formula $K_2 = (L \times c)/\sqrt{H}$ (length of pipeline L and delivery head H in m, flow velocity c in m/s), which for $K_2 > 70-100$ necessitates the installation of water hammer safeguards [12].

In addition to relief valves, energy accumulators are also very frequently used as water hammer safeguards. Their function is the control of the ratio of the kinetic/translational energy of the water in the line, to the kinetic/rotary energy of the pumping unit.

Energy accumulators are:

a flywheel with the flywheel mass of the pumping unit for the storage of rotary energy;

an air chamber for the storage of hydraulic energy.

Flywheels

Flywheels are suitable for horizontal pumps with higher speeds at average or large delivery heads and pipe lengths as far as possible not in excess of 3 000 m.

The determination of the pressure drop at pump run-down can be worked out using electronic computers. In practice, however, the graphic method of Schnyder–Bergeron [15, 16] is still proving very useful. The speed change during the pump rundown process can be determined using an iteration method or more quickly still with the aid of a ω-K diagram [12]. The approximate speed of rotation after the first reflection period is found using the values for the pipeline length, the velocity of the pressure transmission and the torque and flywheel effect of the pump. In the c–H diagram, the point of intersection of the water hammer straight lines with the

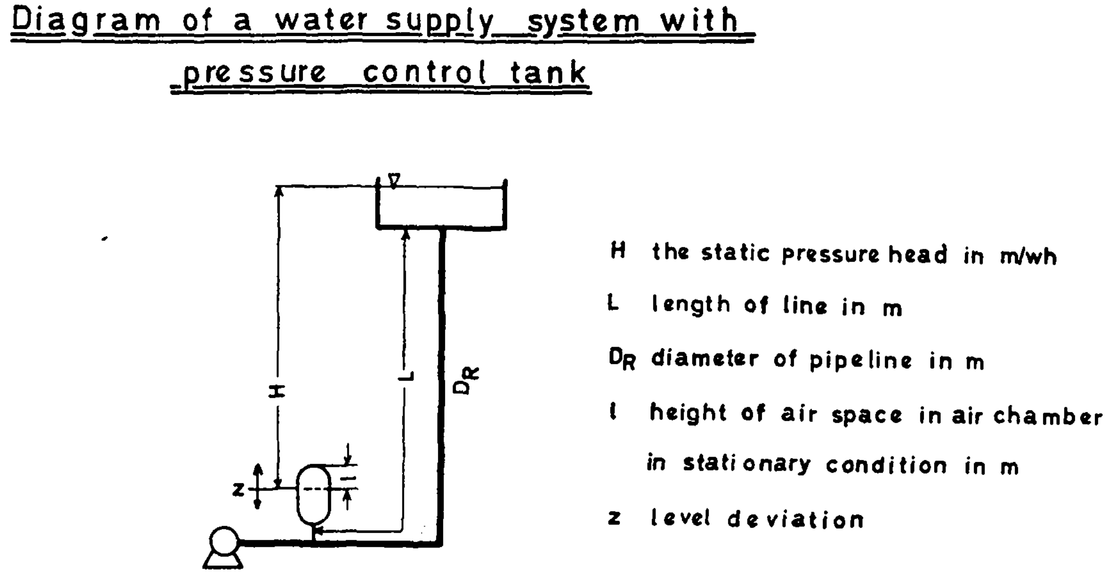


Figure 9

corresponding c–H line for the pump obtained from the relationship $\omega/\omega_1 = c/c_1 = \sqrt{(H/H_1)}$, gives the pressure drop after the first reflection period.

Air chambers

The air chamber is used with great success as a damping element in reducing stations with long supply lines and also as a control element for zonal pumping plants. Its main application is water hammer protection for pump pressure lines, Fig. 9.

There are several design criteria for the air chamber in pump pressure lines. In addition to the breaking and rated pressure conditions, there is a third condition, namely that the volume of the air chamber should be large enough to ensure that no air can escape into the pipeline when it is released.

It has become common practice to select the volume of the air chamber to within 1–2% of the content of the pump pressure line to the tank, as an initial approximation. However, in the majority of cases, a more accurate calculation of the volume of the air chamber should be made. However, to date no success has been achieved in combining all the phenomena of the oscillation process into one single formula. The step-by-step calculation of the Schnyder–Bergeron graphic method gives good results. The water which flows out from the chamber after pump failure results in a reduction of the volume of water and in a change in the pressure-volume state of the gas cushion in the chamber. This change of state, assumed for several volumes within the time unit, results in part of a curve in the c–H diagram, which at the intersection with the water hammer straight line, gives the pressure drop after the first reflection period (if this was chosen as a time unit). The other points are developed therefrom and, for reasons of simplicity, the change of state can be calculated as an isotherm, although its actual trace represents a variable polytropic curve.

Ludwig and Stack [13] solved the oscillation equation for the isothermic change of state by the incorporation of pipe friction, introducing the oscillation of the water level z in the air chamber as a variable. However, the elasticity of the water and pipeline (so-called "Inelastic Theory" as for example with Boerendans [3] and Evangelisti [6]) is not taken into account. This method gives good results except with small chamber volumes.

Starting from the differential equation for the downward directed motion

$$\frac{d^2z}{dt^2} - m\left(\frac{dz}{dt}\right)^2 + n\left\{p_o\left(\frac{l}{l-z} - 1\right) + z\right\} = 0$$

$$m = \frac{F_w \times \lambda}{2F_r \times D_r} - \frac{F_r}{2F_w \times L}$$

$$n = \frac{g \times F_r}{L \times F_w}$$

L = Length of line
z = Water level oscillation, air chamber
$F_{r,w}$ = Cross-section pipe, air chamber
l = Height of air space in chamber
λ = Characteristic value of friction loss

the solution equations are developed, which implicitly contain the extreme values of z, which can be easily derived by trial and error. Nevertheless, for numerical evaluation, values for the respective integral logarithms should be taken from mathematical tables.

The analogue computer also lends itself to the solution and evaluation of the above-mentioned differential equation. This has the advantage that the effect of parameter change can be rapidly recognised, so that different operating conditions can also be simulated. The results appear as curves on the plotter [17].

Non-return valves

Sudden closing of the valves when the pump fails is hazardous. It occurs if the time in which the velocity of the water column drops to zero (T_R) is shorter than the valve shutting time (T_S). The longer the pipeline, the greater the velocity, the smaller the static pressure and the greater the pump run-down time, the larger T_R will be.

Air chambers shorten the effective length of the pipe to the distance between pump and chamber. Water hammer is therefore the rule here, even weights on the valve lever are not particularly effective. The installation of non-return valves with a flap edge seat on one side should be disposed with. In contrast, nozzle-type non-return valves with spring pre-tension have proven successful. Attention should be given in every case to stable pump characteristics.

3.2.2.2 Pumps in the network

The size and design of a pumping plant are determined by flow rate and height and the type of layout of the pumps with regard to the suction and pressure end,

Four types of layout can be identified [8]:

1. Tank—pump—tank T-P-T
2. Network—pump—tank N-P-T
3. Tank—pump—network T-P-N
4. Network—pump—network N-P-N

The particular situation in a supply area is always decisive for the type of layout. Irrespective of whether the system is necessitated by the development of a built-up area or by its subsequent installation due to development (overloading of the network), the following factors are normally the basis for the layout:

—Topographic position of the respective procurement and storage point, as well as that of the supply area;

—Length of supply lines;

—Size and condition of pipe networks;

—Structure of consumption.

1. $T_1 - P - {}^{N_1}T_2{}^{N_2}$

With this layout it should be made clear whether the tank T_2 is operated as a flow tank, whether the feed line to the tank is only a pump pressure line and whether the network N_2 is fed separately from the tank; or whether T_2 is at the other end of the network as a corresponding tank or whether the pump pressure line is also the supply line. In the latter case, the delivery head will change according to the network consumption requirement; pumps with a steep characteristic should be used.

If the costs of a flow tank and a corresponding tank are compared, the latter is usually cheaper. The take-off line N_2, which is separately laid, is not necessary.

As the network is fed from two sides, the tank take-off line does not need to be designed for peak consumption. A further advantage of the corresponding tank system is the possibility of subsequent correction of the supply pressure by changing of the pumps. With the flow tank, the hypothetical energy curve is finally fixed, while this is super-imposed for the corresponding tank with the sum of all head losses from the network to the tank. The flow tank is simpler to operate, the geodetic percentage of the delivery head is constant and there is less danger of water stagnation during storage.

2. $N-P-T$

In this case, the network pressure is not always sufficient to supply the tank. By corresponding dimensioning of the tank, the pump can be completely adjusted to the network conditions. Pump or supply line failure does not lead to a short term failure of the water supply. The water supply for fire-fighting is in reserve.

3. $T-P-N$

The type of layout with direct delivery into the network is encountered, if

—Sufficient height is not available for the tank on flat ground, or

—Two supply zones are super-imposed in height and the tank for the lower lying zone simultaneously serves as the drawing off tank for the pumping plant delivery into the upper zone.

In the first case, speed controlled pumps are very suitable for the supply of larger parts of the network. Control of the flow is carried out according to water requirements, where in each case only the required delivery head is produced. The speed control can be carried out with gears or by control of the drive motor, which is normally more effective. The basic delivery flow can be met by pumps of a fixed speed. The most frequent form of zonal pumping plant is the air chamber pumping plant, which operates more economically with pressure controlled pumps up to a delivery rate of Q = 100 l/s, than a quantity controlled pumping plant.

The advantage of direct discharge into the network is its ability to be adapted to consumption by the switching on and off of corresponding pumping units. With less consumption and correspondingly less frictional head losses, pumps with less delivery head can be used.

4. $N_1 - P - N_2$

With this layout there must be sufficient capacity present in the network section N_1 to draw off the required amount of water by operation of the pumps, without impermissible pressure drop.

The feeding-in into larger networks is also carried out via speed controlled pumps according to requirements.

"Pipe pumps" have proved themselves for pressure boosting of individual network sections. In normal operation, as pipeline installations, these are bypassed if the pump is shut down, and are only switched on when required.

3.2.2.3 Booster systems on private properties (BS)

The normal operational pressure supply from the waterworks only extends to the highest situated consumer according to local conditions (compare with section 4). Multi-storey blocks can only be supplied with the normal supply pressure up to a height according to local conditions, while the floors above this height have to be supplied via a booster system which belongs to the building itself. Booster systems cause severe localised stress on the network. The following is demanded in a DVGW guideline [5]:

Booster systems should be so laid out, designed, operated and maintained that the continuous operational safety of the water supply is guaranteed and neither the public water supply, nor other consumption systems are interfered with. A subsequent change in the drinking water quality should be excluded.

Interference to the supply network is caused by:

—the possibility of backflow,

—cases where impermissibly large water hammer is produced at switch on and switch off, due either to too large dimensioning or too large a consumption, or if, due to the operation of the system, water is drawn off from neighbouring consumers.

If the network cannot provide a sufficient supply for the booster system an auxiliary storage tank should be used.

The direct connection of the booster system is only allowed if the velocity change in the connecting line, caused by the switching on or off of each pump, does not exceed $\Delta c = 0{,}15$ m/s or if an air chamber is installed on the suction side, or if this is guaranteed by other measures.

Some manufacturers of booster systems describe their product as being water hammer free. This means systems which produce smaller water hammer, insofar as their switching actions occur with smaller volumetric flow or with a time lag.

Many booster systems have pressure controlled switching with a large air chamber installed downstream as a control element. At switch on, full output from the pump is produced, regardless of whether there is consumption or not. If the pumps' dimensions are too large, as is mostly the case, this has its full effect on the network. The demand curve is the result of average peak values, which in practice are not met in every case. On

the basis of constant stress curves [9] for houses with flushing cisterns or flush valves, it can be shown that only the latter bring larger and short-term peak draw-off of water. Pumps for air chamber systems in booster systems should be closely attuned to the anticipated consumption structure and dimensioned accordingly. It is also better to have a stand-by in the chamber for this contingency. This is equally true of booster systems for department stores, office blocks, hotels and hospitals.

With important buildings, the installation of a further reserve pump should be considered to cover occasional, but rare consumption peaks.

Booster systems which have only a small diaphragm tank as a control element on the pressure side or even none at all, discharge only the amount drawn off by the consumer on each occasion. The system only switches off at the minimum consumption, i.e. at a small delivery flow. Water hammer at switch off is therefore normally within permissible limits. If there is a power failure, this system can also produce an impermissibly large water hammer, Fig. 10.

Although a system without an air chamber is better for preventing impermissibly large water hammers in normal operation, it will not displace the booster system with air chamber, because, despite lower investment cost, it cannot attain the profitability of a system whose pumps always operate at optimum efficiency.

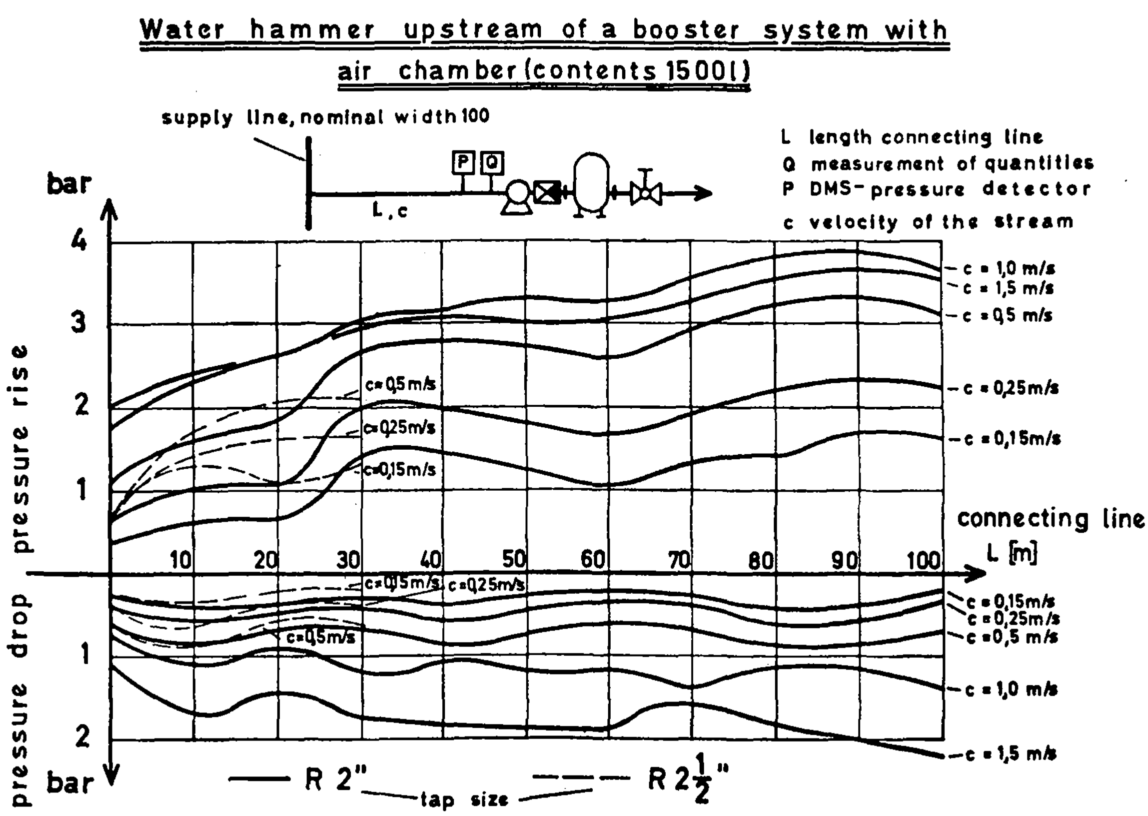


Figure 10

3.3 Control and monitoring

There is available a number of automatic control procedures for the maintenance of a given pressure and other important measuring parameters. Alternatively, control will be undertaken wholly or partly by a central station. Both systems can be manually controlled and require uninterrupted monitoring by the central station. Central, manual control can only be used with very small systems or with a limited number of switch commands. With larger systems, the evaluation of data and control can be carried out via a process computer.

4 Supply pressure and profitability

Whilst it is capital costs which significantly mount up with tanks, it is operating costs, in particular energy costs, which mount up in the case of pumping plants.

With current electricity costs (e.g. Stuttgart, 1975: basic charge 12,9 DM per KW and month, operating price 9,7 pfennigs per KWh) and an average pro capita consumption of 240 l/day, every pressure rise of approx. 1 bar provided by pumps results in total costs pro capita and per year of 53,6 pfennigs. All precautions for reduction of energy consumption should therefore be considered at the system planning stage, e.g.:

—Position and arrangement of the pumping plants in the network; number, graduation, control and degree of efficiency of the pumps.

—Least possible distance between the pumping plant and the focal point of consumption. This results in smaller investment costs and, as a consequence of lower pipe friction losses, smaller energy costs.

—Maintaining static pressure head at the absolute minimum necessary to guarantee the minimum flow pressure to the highest consumer in the area.

If consumers are supplied with pumping pressure by a single point to different locational heights, then the same delivery head z_i should be used for them even though this pressure head would only be necessary for the highest consumer. As this is normally unprofitable, the supply area is broken down into supply zones (mostly zones with graduated heights) [1].

The zoning can be caried within the specified permissible pressure limits; a break down into arbitrary numbers of sections of smaller pressure differences can, however, only be continued until the amount of investment caused by the increase in the number of operational devices is counterbalanced by the advantages and savings resulting from a smaller supply pressure. That limit is of

Waterflow-out from tap valves experimentally determined at different pressures

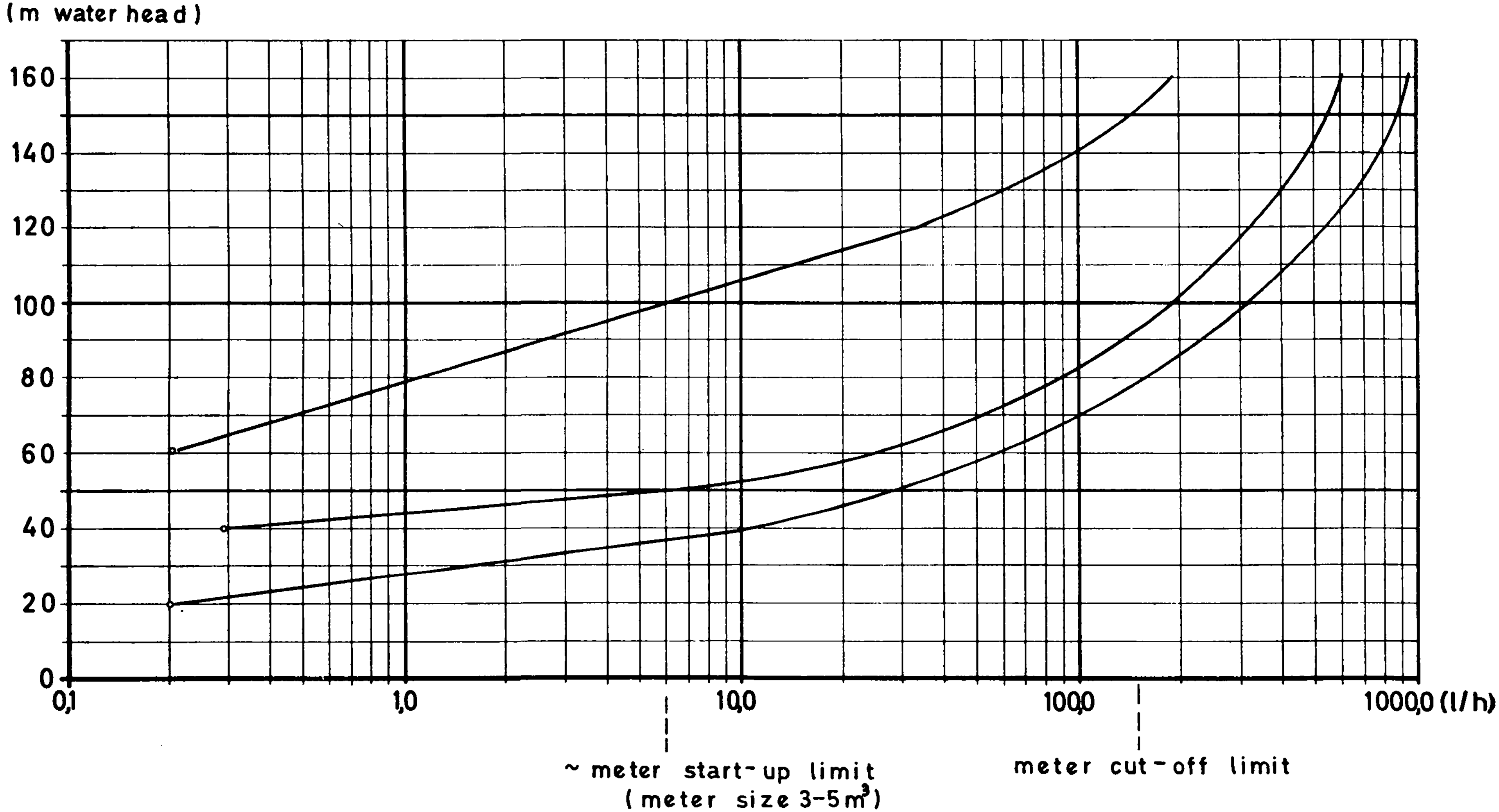


Figure 11

course dependent on local conditions and therefore varies. However, too many zones make operational surveillance more difficult.

4.1 Pressure zones

Supply to the individual zones can be in parallel from a central point or in series from one zone to its neighbouring zone.

A separate pressure line to every individual zone is necessary with parallel connection. This system should only be selected if transporting distances and differences in elevation are not too large. For larger transporting distances and differences in elevation, a series connection is more economical. But here, however, operating stations are necessary in the respective transition.

If the supply pressure is not to exceed 60 m water head (with a minimum pressure for the highest consumer of 20 m water head), the differences in land level may be approx. 40 m. The larger is the permissible max. supply pressure for the lowest-lying consumer the less the number of zones required. Too high pressures cause increased stress on fittings, pipe materials and seals and therefore an increased amount of damage. An increase in consumption and leakage losses can also be proved, as shown in the following.

5 Supply pressure and water consumption

From tests carried out it has been shown that there is a connection between pressure rise and consumption quantity or the quantity delivered into the network. There is no doubt that water losses (trickle losses) and leakage losses (pipe connections, cracks, hair line fractures, pitting) are involved in the increase in consumption.

5.1 Trickle test

An ordinary tap R ½″ with normal seal was used. The fitting was connected to a testing system and so adjusted to several set-points (20 m water head, 40 m water head, . . .) that a drop formed every three seconds, following which the pressure was increased in each case.

The following can be read off and deduced from Fig. 11:

With a normal supply pressure, a valve may lose one drop (= 0,3 l/h) every three seconds. If the supply pressure increases during the night over a period of 12 h by 10 m water head, the trickle loss of the same valve correspondingly increases to 6,0 l/h. This leakage loss is too small to be recorded by the water meter and is therefore not paid for.

For a supply area containing 80 000 water meters and assuming one trickle point allocated per water meter, the result is that 2,1 million m^3/a of consumption is not metered (= 30% of the difference between the water supplied and that paid for by consumers).

The above reasoning is somewhat hypothetical because it is not known how many actual trickle points there are. Also only one trickle point is allocated to each property. However, in practice slow losses in flushing cisterns, tap valves etc. can be observed everywhere so that the figure estimated is probably not too far out. The following example bears this out.

5.2 Verification in a block of flats

A Woltmann water head meter (50 mm) was bypassed in an apartment block of 56 flats (built in 1963) and the nightly consumption measured with a 7 m^3 meter. The actual consumption times and the leakage losses can be clearly read off from the diagram (Fig. 12). From the gradient of the straight lines, an overall consumption in four hours of 480 l results with leakage losses of 230 l (= 48% or 57 l/h leakage loss) corresponding to 1 l/housing unit. This value appears extraordinarily large, but confirms the result of 5.1.

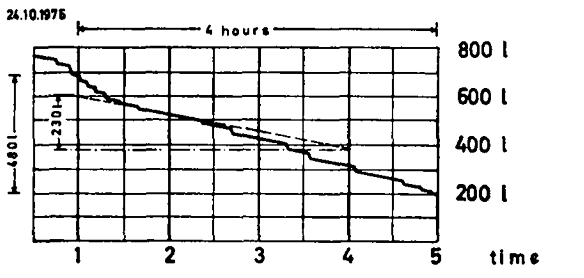


Figure 12

5.3 Zonal consumption with various pressures

In a zone supplied by pressure reducing valves (2 860 inhabitants), the consumption was measured starting from normal pressure and then at smaller pressures reduced in each case by 5 m, 10 m water head and so on. The curve in Fig. 13 resulted from the basic values and clearly shows a fall off in consumption dependent on the reduced pressure.

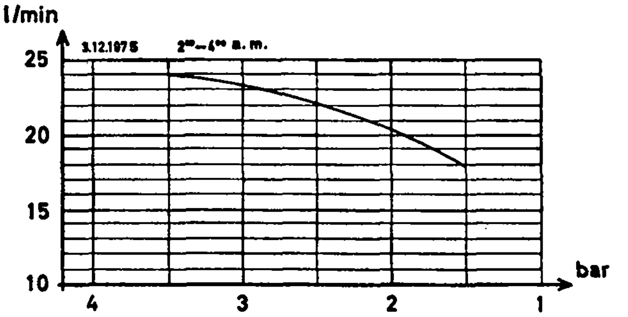


Figure 13

5.4 Zone with air chamber pumping plant

A pressure quantity curve was deduced from the characteristic circuit diagram of the air chamber pumping plant and this is shown in Fig. 14.

The consumption peaks shown during the first 20% of the tap-off phase (nightly consumption) can be reproduced from other graphs. No explanation for the steep initial slope of the curves (increased consumption could be found other than the increased pressure level.

Unfortunately, the example in 5.1 is not sufficient to be able to prove the relationship between the leakage loss and the increased consumption at increasing pressure; the influence of the pressure level on consumption can, however, be shown.

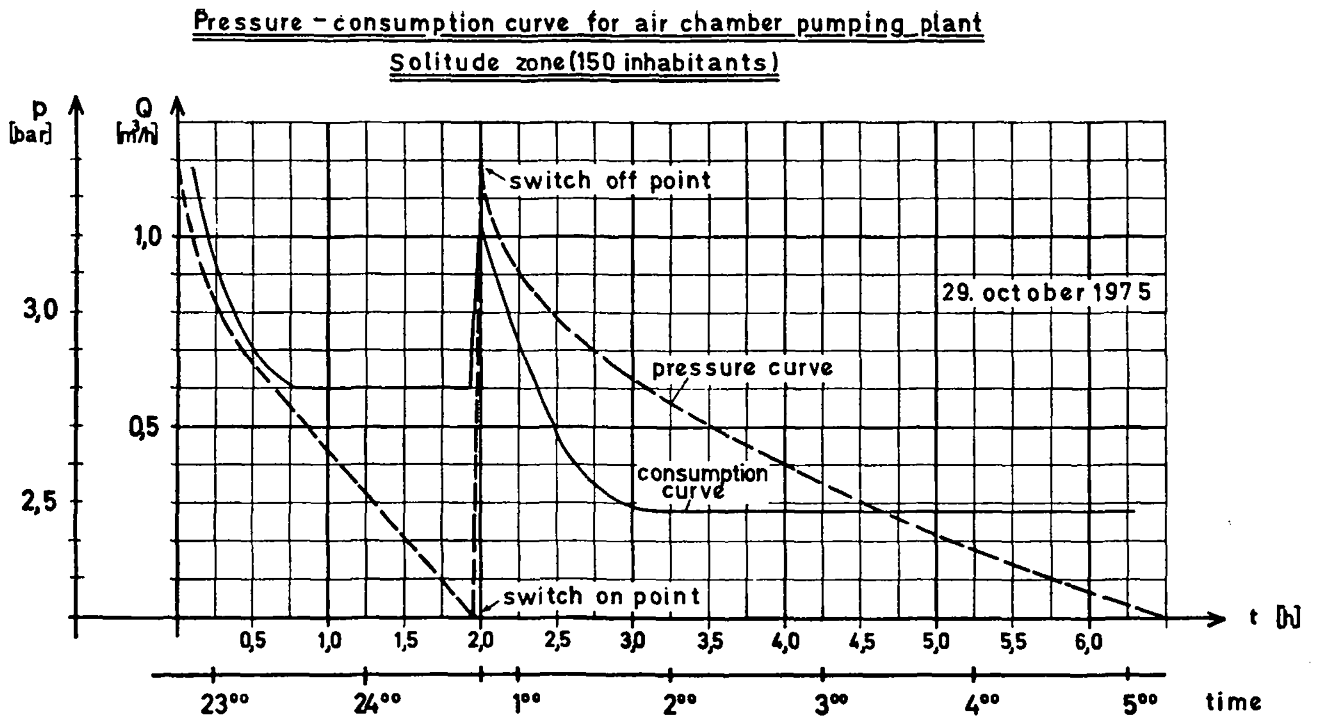


Figure 14

6 Legal aspects of pressure control

In Germany there is liability for risks* in the field of electricity and gas; in the field of water supply, there is liability for intentional acts†. The judiciary in the Federal Republic of Germany has, however, demanded a duty of care of the public supply companies such that there are not too many differences between this and absolute liability in the final analysis.

In the field of electricity and gas, the General Supply Conditions are standardised by the General Liability Declaration of 1942. These regulations have the character of statutory orders and conclusively control the contractual relations between public supply companies and consumers in the Federal Republic of Germany.

No such General Liability Declaration has been made in the field of water. With the General Supply Conditions in this case, these are actual General Operating Conditions, which control the rights and duties of the waterworks and the consumer in civil law. In accordance with this the water supply companies have reserved the right to make pressure changes. It is regarded as an accessory, contractual consideration that the public supply company will inform the consumer about any intended pressure change. The consumers, however, have no claim upon the maintenance of a particular pressure.

If damage occurs due to a change in pressure, it is for the judiciary to decide in each case whether the damage occurred as a result of typical operational dangers for water supply companies (e.g. failure of technical equipment) and therefore whether it is a risk to be borne by the consumer.

To sum up, there are no adequate criteria given by the legislature in the Federal Republic of Germany for the pressure field.

* Liability for risks means responsibility for a certain material or operational danger for social reasons, where a culpable behaviour of the party liable for damages does not have to be given.

† Liability for intentional acts means responsibility for culpable behaviour.

7 Conclusion

In this paper, an attempt has been made to derive the correlation between Pressure Control and technical operation and to look at these from an economic point of view as well. In each case, examples of operating conditions for energy changing systems, both with long lines and in networks have been examined and in addition water hammer problems have been discussed.

The necessity of reducing excess supply pressures as a requirement for the reduction of the energy used in networks supplied with pumping pressure has been discussed, as has the requirement for the reduction of operational malfunctions and network losses.

In the final analysis, Pressure Control and evidence of its preservation by uninterrupted recording of operations is important for protecting the public supply company against unjustified claims for compensation.

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Résumé

En traitant ce sujet, les aspects techniques, économiques et légaux ont été considérés.

Le principe essentiel de toute distribution d'eau est de garantir les besoins en eau potable en tous points de la zone desservie, en quantité suffisante, de la meilleure qualité possible et sous une pression d'alimentation adéquate à tout moment.

Ce que l'on entend par pression d'alimentation est la pression statique présente dans le réseau: elle doit être au moins suffisante pour garantir le minimum de pression de débit aux points de soutirage les plus éloignés et les plus élevés dans la région desservie. La pression maximale admissible dans la réseau est fixée à 10 bars. D'une façon générale, les pressions normales dans le réseau sont de 2 à 8 bars.

Ce que l'on entend par contrôle de la pression est l'influence sur et le contrôle de tous les paramètres, en vue de garantir une pression d'alimentation adéquate en tous temps, quelle que soit l'altitude du point de consommation.

Pour la contrôle de la pression, il faut dériver les relations suivantes: le contrôle de la pression est une exigence pour l'exploitation technique; le contrôle de la pression permet une exploitation claire, facile à suivre, économique. Le contrôle de la pression, entendu comme réduction des pressions d'alimentation excessives, est avantageux pour:

—la vie des conduites et des branchements,

—la protection des appareils et installations du consommateur,

—la réduction des pertes causées par les fuites possibles.

Le contrôle de la pression est nécessaire comme preuve de l'exploitation régulière de la distribution d'eau, par exemple du point de vue légal.

On discute de l'équipement de modification de la pression en technique d'exploitation, par exemple:

pompes et réducteurs de pression, rattachés chaque fois à l'exploitation d'une longue conduite et dans le réseau.

Description est donnée d'une vanne à piston cylindrique avec plaque d'orifice comme exemple de réducteur de pression, dispositif de contrôle à l'extrémité d'une longue conduite avant déversement dans le réservoir.

Les problèmes spéciaux et mesures pour éviter la cavitation en exploitation continue dans les régions où il y a des ouvertures partielles sont discutées pour un cas donné.

Pour l'emploi des turbines en vue de réduire l'énergie potentielle hydraulique, une comparaison économique est donnée à titre d'exemple, avec les accessoires de contrôle comme les réducteurs de pression.

Bien que les turbines exigent un investissement supérieur aux réducteurs de pression, elles peuvent être utilisées avec bénéfice. Mais cependant on ne peut pas se dispenser d'un réducteur de pression comme unité de réserve.

Les utilisations possibles des vannes réductrices de pression dans le réseau sont mentionnées. Les conditions pour leur bon usage sont énumérées.

Référence est faite aux changements admissibles de pression dans la conduite lors des manoeuvres sur les pompes pour les longues conduites ou aux mesures à prendre pour étouffer les coups de bélier.

Des valeur empiriques des domaines d'application sont énumérées pour les volants et les réservoirs à air qui sont les accumulateurs d'énergie les plus utilisés.

Référence est faite à quatre types de schéma d'insertion des pompes dans le réseau, et les conditions d'exploitation qui en résultent sont décrites. Une attention particulière doit être apportée aux pompes de surpression installées dans les propriétés privées, car elles peuvent amener des désordres dans le réseau de distribution.

Si la pression d'alimentation est produite au moyen d'une énergie de pompage, la pression peut être rettachée directement au coût de l'énergie. Le rendement de l'exploitation est augmenté si l'on évite les pressions excessives. Les régions où il y a d'importantes différences d'altitude sont divisées en zones d'altitude graduée, une différence étant faite entre les connections en série et parallèles.

Lorsque la pression d'alimentation augmente, on observe une augmentation de la consommation dont une part est attribuable aux pertes par les fuites.

Finalement, le contrôle de la pression et la conservation de sa preuve par un enregistrement général de toutes les manoeuvres est important pour protéger le service de distribution d'eau contre les plaintes en dédommagement injustifiées.

Advances in the protection of distribution systems against backflow

by Ir. W. C. Wijntjes

Gemeentelijk Waterbedrijf Groningen

Introduction

At the I.W.S.A. Congress of Barcelona in 1966, G. J. Angele gave a survey of some historical accidents leading to the contamination of drinking water supplied by the water company (Ref. 1). In his lecture he made (along with a number of technical possibilities to prevent the pollution of the central drinking water system, proceeding from the drinking water installation as a result of particular, unpermitted connections or situations) a number of remarks on the responsibilities of the authorities and persons involved in the water supply of a water company. Some of Mr. Angele's important theses were in this connection: "Pure drinking water is a vital component of life. It is a tremendous undertaking to maintain a potable water supply through the vastly complicated public and private pipelines of a nation today. The duty to investigate the water supply and to ascertain possible sources of pollution rests on the water company together with the further duty of taking such positive action as is necessary for the protection of its customers!"

Now that ten years have elapsed since the Congress of Barcelona, the discussion should be opened as to which way developments in the possible pollution of drinking water installations and grids of water companies have been evolving, and how a better safeguarding (in spite of the connection of an increasing number of appliances to the drinking-water installations) can ensure that the drinking water that is supplied to the consumer remains drinking water.

Risks in drinking-water installations

An obvious question to pose is that of what possibility is there that eventually, under particular circumstances, a drinking water installation may be polluted, and what dangers for the consumers may arise from it.

Our English colleagues inquired closely into this difficult problem in the years 1970/71. The inquiry was made by the Committee on Backsiphonage of the Department of the Environment and was delivered as Technical Paper TP 82 of the Water Research Association (Now Water Research Centre at Reading) (Ref. 2).

Although the report refers to English circumstances, some conclusions are not unimportant for the problem posed:

(a) 61% of domestic properties are at risk in terms of the requirements. If temporary connections are included as a risk this figure rises to approximately 85%.

(b) The majority of risks (about 95%) relate to cisterns and taps.

(c) The actual probability of occurrence of backsiphonage from domestic properties is a function of the risk as investigated herein, the occurrence of low or zero pressure and other factors such as taps actually being flooded or hosepipes being immersed in some contaminant during the occurrence of low or zero pressure.

The probability of backsiphonage occurring is anything other than very low.

It should be observed that in this investigation only household connections were involved. The results of similar inquiries connect naturally on a large scale with the question, if, and to what degree the construction or the modifications of drinking water installations are followed by an inspection. Moreover we shall have to accept on the grounds of practical experience of those who have something to do with the inspection of drinking water installations of factories, offices and other household installations, that countless examples can be given where the situation in the installation contains a serious threat to the health of a number of people. In fact it is for that reason that since 1968 in a number of countries ample attention has been paid to these problems, in which case in those countries increasing attention has been given to techniques of installation.

Pollution of the drinking-water

The pollution of drinking water from consumers' installations is a problem not easy to approach. The problem has been in existence since the origin of central drinking water supply and is, in fact, indissoluble from central supply, in particular where the drinking water installation in the case of direct connection is connected directly to the mains.

It means equally, based both on economic and practical considerations and experience that the quality of the drinking water from a draw-off point is sometimes inferior to that which it should be.

We have to make it our object to take such measures and precautions that the chance of pollution of the water supplied will be extremely small. In fact the basis of the problems is laid down by the necessity that for the transport of water, "pressure" is required.

In many devices connected to a drinking water installation, water is used as a solvent for other substances, e.g. the washing-machine.

In these appliances contact with other substances takes place, and it is possible that the drinking water in the consumer's installation and also in the grid of the water company becomes polluted with this matter.

And now, what are the dangers that may threaten, and what measures may be taken to prevent possible pollution?

When studying the question of which dangers may threaten, two elements have to be distinguished.

(a) What technical conditions have to be fulfilled so that the foreign matter pollutes the drinking water.

(b) If the drinking water has been polluted with such foreign matter, what may be the consequences for public health.

In the Netherlands a distinction was made in 1967 (Ref. 3) between so-called "press-cross" connections and "suck-cross" connections.

By a "press-cross" connection is understood a connection between the drinking water installation in

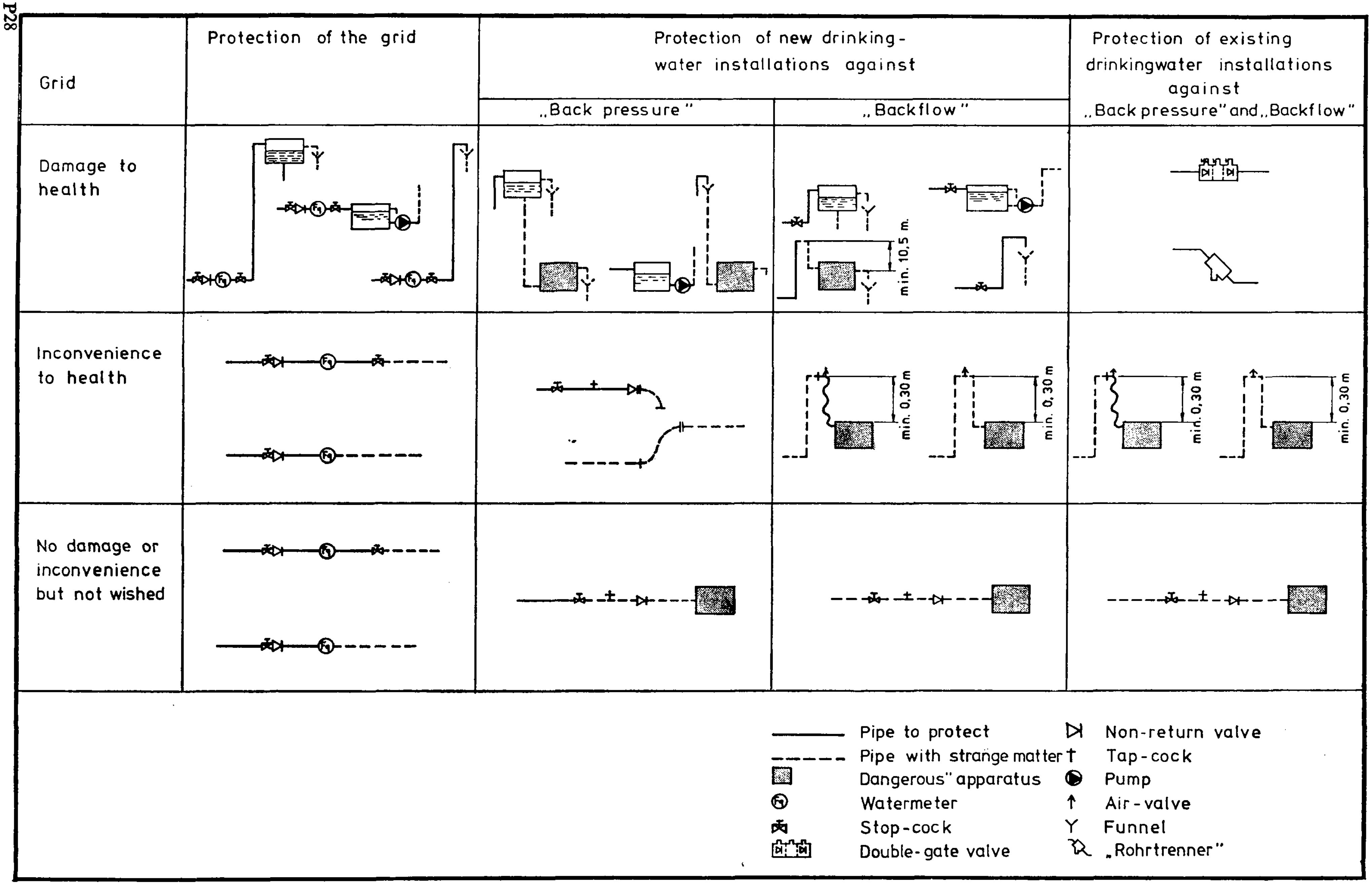


Figure 1—The classification table.

which a foreign substance is present under a higher pressure than that of the atmosphere.

The "suck-cross" connection is a connection between the drinking water installation and an apparatus or a pipe system in which a foreign substance occurs under a pressure not higher than that of one atmosphere.

The difference indicates practically, that in case of the press-cross connection only one condition is sufficient before contamination of the drinking water appears (e.g. the opening of a stop cock or where a non-return valve becomes inactive), and for the suck-cross connection there are at least two conditions (e.g. at a certain moment a suction must occur, and at the same time the connected apparatus must be in operation).

Fortunately it is evident, at least in Dutch practice, that press-cross connections may occur sporadically, but that suck-cross connections may occur in many places. This leads to the important conclusion that undertakings that have at their disposal an evenly balanced and well dimensioned grid, and can see to it that drinking water installations, too, meet similar demands, will have the least trouble with the consequences of a suck-cross connection. Because in practice, however, there are numerous situations in which they can occur, the necessary vigilance should be exercised even for those undertakings. But where there is the chance that pollution will appear, the possible consequences of the pollution will be important. In several countries attention has been paid to this aspect, and there has been a division into classes of hazard.

In the U.S.A. (Ref. 4) two categories with respect to potential or actual cross connections are distinguished:

Category 1. A physical or toxic hazard which could be dangerous to health.

Category 2. A non-health hazard which would cause aesthetic problems or have a detrimental effect on the quality of the water in the system.

In England (Ref. 5) and in the Netherlands (Ref. 3) they strive for a more extensive classification into three categories:

Class 1: Lasting damage to health appears (e.g. cyanide of potassium).

Class 2: Temporary trouble for public health appears (e.g. paratyphoid fever).

Class 3: The connection is undesirable (e.g. coffee).

Apart from the above, this categorical division does not give any practical guidance as to when and under what circumstances what demands have to be made. Types of protection that have to be allowed are still lacking in every class and furthermore all possible substances that may pollute the water, eventually with their concentrations, should be subdivided in the same way.

Means of prevention

Amongst the various types of safeguards an important difference can be made. There are a number of ways of preventing a foreign substance from flowing back into the drinking water installation as a result of a "backpressure" or a "suction".

An example of this is the so-called interruption. Provided that it is well executed, **backflow** is out of the question. Technical safeguards are liable to wear, age or corrosion and there are consequently no so-called absolute safeguards. In the course of time, dependent on construction, water quality and situation it is to be expected that they will lose their functional vitality and consequently not exercise their protective task any longer. The backflow preventer (non-return valve) is a good example of these.

From practical experience it is to be expected, however, that for this category of safeguard a difference can still be made between the quality of the safeguard as a construction and the time that the latter will last. A certain order would, so to speak, be drawn up.

With a view to the classification of dangers, it is clear that for the highest class of danger nothing less than absolute safeguarding will suffice, whereas in the remaining classes (referring to the English and Netherlands principal classification) safety devices will be sufficient.

In case of this classification the difference between "backpressure" and "backflow" is an important factor. The high loop (H = 10,5 m.) can be applied in the second case, but not, however, in the first case. Furthermore we have to observe that, on principle, a single safeguard will have to be sufficient.

The starting point has to be that the quality of the safety devices applied provide excellent and proper control.

The combination of safety devices or the formation of series of them leads to extra cost, and to higher pressure losses, but scarcely to higher safety.

In fig. 1 an essential survey of the possible safeguards in the various categories is given.

In some situations the "double gate valve" with a spacer between is acceptable. This shows by means of an exhaust pipe if both "valves", which for the rest must be sealed by the water company, are closed. Naturally the use of this type of safeguard depends upon the degree of danger (fig. 2).

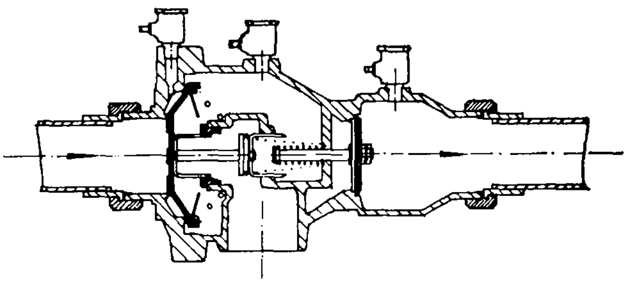


Figure 2—Double gate valve.

For the time being this type of safeguard has been found equivalent to a non-return valve in England. As a result of its signalling function it **does** give a higher protection, however, and application of this "device" might be considered in a number of cases to be on the line between categories 1 and 2 of fig. 1. In addition, a similar apparatus might be used in existing installations where reconstruction or adaptation of the existing installation would induce additional difficulties. In Germany (Ref. 6) also, an apparatus has been developed that might fit in with the whole. It is the so-called "Rohrtrenner" ("Pipe Separator"). Fig. 3 gives an example of the construction.

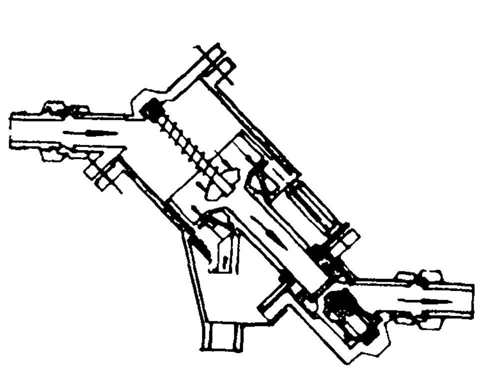


Figure 3—The " Rohrtenner ".

Safeguarding of the grid of the water company

Theoretically speaking it would be sufficient with regard to the safeguarding of the quality of drinking water to introduce protective devices at those "points of use" that possibly would present danger for water quality.

From experience of the daily practice of the inspection department of a water company it appears, however, that it is impossible to obtain a 100% safe and protected situation in drinking water installations. This is in spite of the fact that in some countries only approved specialists may work on drinking water installations, and of a good inspection of the installation. Particularly in those categories of consumers where a so-called technical service is extant (hospitals, factories etc.), the possibilities of wrong situations are particularly great. This has been the reason that, particularly in the Netherlands, not only are special tapcocks and/or apparatus protected but an extra safeguard is provided at the point of the supply by the water company: namely near the main cock and/or the water meter.

Here, too, the class of hazard to which the premises must be assigned is weighed, and then the right protection is chosen. This means that to hospitals, sewage purification plants, large factory buildings, the water is only supplied via a reservoir as an interruptor.

In England (Ref. 5) they advise the separation of industrial water, or that part of the installation in which no drinking water in the sense of drinking water has to be tapped, from the pure drinking water installation.

In America (Ref. 4) the dangers of cross connections in dual industrial and potable water supply systems are emphasised.

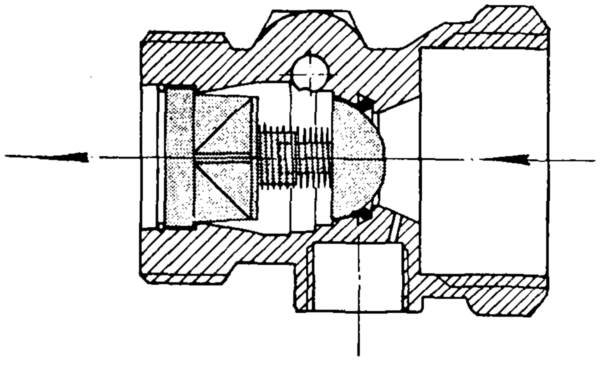
In the classes of hazards 2 and 3 the situation has been developing in a favourable sense in the Netherlands since 1967.

Conventional return valves generally close against a high difference of back pressure. Certainly they do not do so against a low pressure, which must be considered of real interest. Therefore types of so called hygienic non-return valves have been developed which also close in cases of a low difference of counterpressure or, in other words close when the direction of the current is still positive. For that reason the sealing is obtained by means of spring-loaded valves (Ref. 7).

In order to come up to these requirements the testing requirements contain not only a necessary flow rate with 10 m.w.h. headloss, but also a minimum flow rate with a headloss of 0,5 m.w.h. Moreover the non-return valve must be closed at 0,03 m.w.h. on the out-flow side. The construction has been designed in such a way that the closing-body is subjected to very little wear. In fig. 4 a similar construction is shown in more detail.

Since these non-return valves have been developed in the Netherlands, some millions of them have been installed, notably in service connections (already more than 50% of all connections have been safeguarded in this way). So far experience with them has been excellent. Furthermore, the building-in of the body of a non-return valve to the outlet of the water meter was developed in the Netherlands (see fig. 5).

Hydraulic experiments have shown that installing such valves has no influence on the measuring qualities of the water meter. Of course the advantages of this construction are particularly interesting. Not only are there practically no installation costs for fitting the non-return valves, but in addition the non-return valve can be preserved in a simple way; along with the water meter the non-return valve is developed also. In Germany, too, (Ref. 8) a trial has been carried out with similar non-return valves in a large town. As expected (Ref. 7) we may suppose that by introducing these non-return valves the risk of backflow into the mains system of a water company has been reduced by a factor of 20.



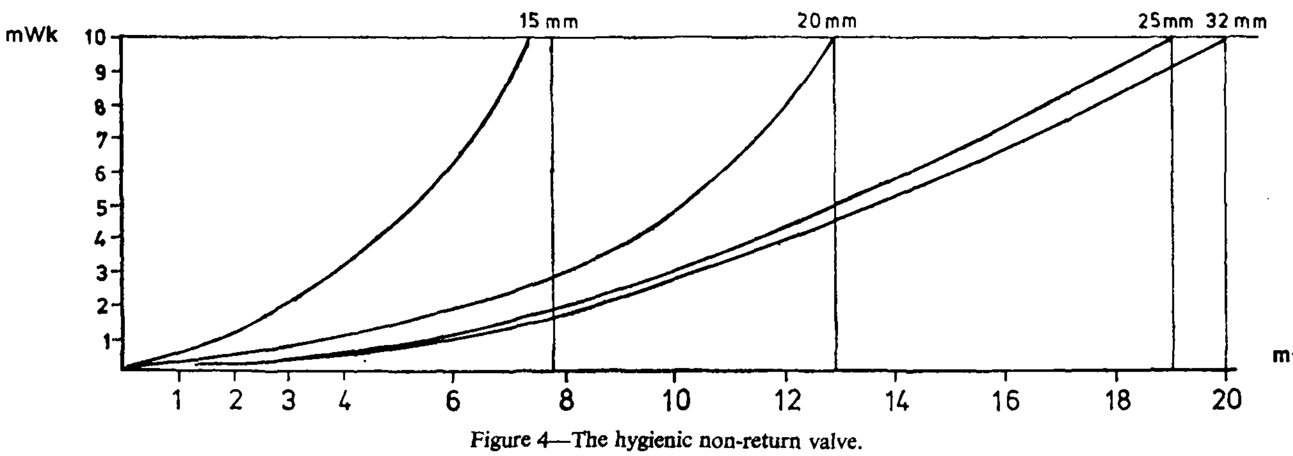


Figure 4—The hygienic non-return valve.

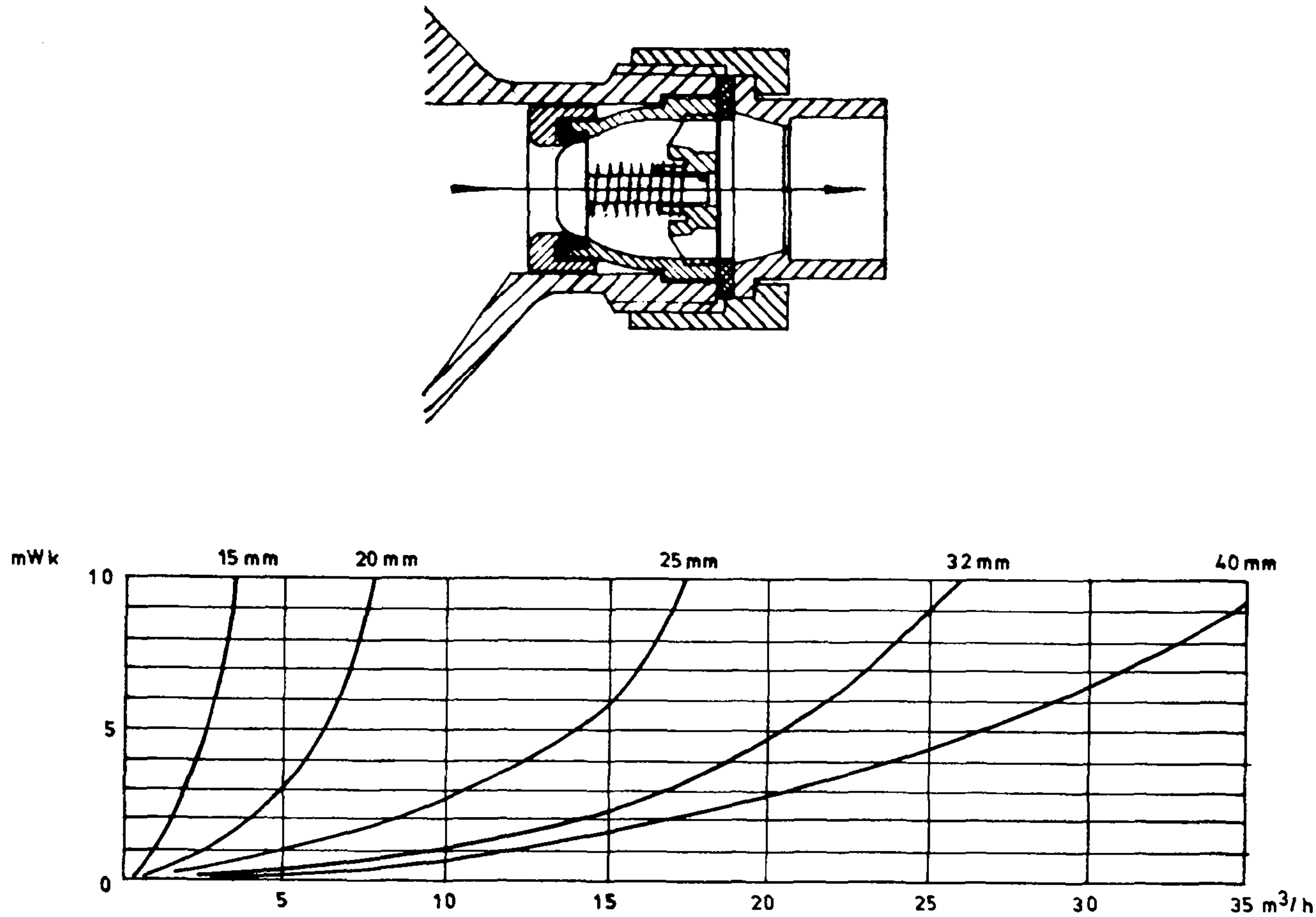


Figure 5—The combination of hygienic non-return valve and water meter.

The introduction of similar non-return valves is leading consequently to a much safer quality of the drinking water that is distributed. It must be emphasised that Netherlands water companies believe that when taking non-return valves into use, they did not take over the responsibility of a possible backflow of water. That remains the responsibility of the user of the installation. The non-return valve only serves to provide a higher level of protection.

The solution as advocated in America (Ref. 1), where the consumer has to place the non-return valve, has not been adopted in the Netherlands for fear that the maintenance of these valves would not come up to the demands made.

Naturally, special experience has been gathered with every type of connection when introducing the system of the non-return valve.

Loading effect

In practice it was observed that after the introduction of non-return valves in the supply pipe to premises in the Netherlands certain complications could arise, even if it concerned only a few cases. Among others a phenomenon may be observed that is known as the "supercharging effect".

By loading effect we understand the step-by-step or gradual course of a pressure increase in a closed system downstream of a closing element, such as a non-return valve. This pressure increase may arise from the following thermal (a–e) and hydraulic (f and g) causes:

(a) transfer of heat from the neighbourhood of the pipe system;

(b) transfer of heat from the waste gas of a bypass in a gas-heated hot water appliance, which contains cold water;

(c) transfer in a water heater of the accumulated heat of the water in the tap spiral;

(d) normal heating of the water in boilers;

(e) continual burning of water heaters caused by faults in the heater itself;

(f) pressure variations in the pipe system on the upstream side of the non-return valve;

(g) pressure impulses in the pipe system on the downstream side of the non-return valve (e.g. from the shutting of cocks).

The principal pressure increases have been measured by the Dutch K.I.W.A.-laboratory in the situations c and e. Pressures of ca. 35 bar. appeared there. Under Dutch circumstances lower pressures appear normally in the drinking water installations upstream of the float of the cistern as a relief-valve. An important conclusion of the working-group which accompanied the Dutch inquiry was that pressure increases in drinking water installations with a small capacity especially if they have originated from the functioning of water heaters, may have disastrous consequences.

It was suggested to increase the spontaneous ignition pressure of water heaters by application of a thinnen upward pressing pin to at least 35 bar. The fitting of relief valves must principally take place close to the apparatus. The principal problem that may arise when placing the non-return valves is the situation where the present non-return valve near the water heater did not work any more. In that case, and in addition, the relief valve on the water heater has probably not been working for a long time, and consequently may have stuck to the water heater.

An explosion may be the result if a non-return valve has been fixed into the inlet pipe by the water company.

Internal safeguarding of the appliances

With the possible kinds of safeguard(s) of the preceding paragraph it is possible to construct safely a drinking-water installation. On the whole it must be stated, however, that the safeguards that have been judged necessary are **minimum** safeguards.

The arm of the water world must in any case strive to take such measures as are necessary to obtain absolutely safe installations, without any risk. This means that we must make it our objective that only such pieces of apparatus are connected that have been provided with an integral interruption, i.e. internally safe appliance.

Tested apparatus

In case of efficient testing by inspection it is necessary that we can observe easily whether or not the apparatus connected has been constructed with an integral device to interrupt supply. For that purpose it is particularly useful, if types approved, whether by a nationally or generally acknowledged institute or acknowledged organisation on the basis of testing requirements, can bear the certificate of "internally safe". Also the safeguard to be applied in non-internally safeguarded apparatus, together with the eventual placing of non-return valves in the supply-pipes to the premises, need to be inspected for preference by one central entity.

International aspects of washing machines

A good example of this development may be noted in West Germany. The "Deutsche Verein von Gas- und Wasserfachmännern" (D.V.G.W.) published Working-Paper W 503 in 1966 in which regulations were given for so-called internally safeguarded pieces of apparatus. Machines with the "D.V.G.W. Prüfzeichen" have come up to these requirements (Refs. 10, 11).

In these appliances care has been taken that the shut down according to the class of hazard 1 has been built into the apparatus itself so that extra safeguarding can be omitted, and direct connection to the drinking water system is permitted. Among others, washing-machines, dish-washers, urinals and slop pails are included in these regulations. Each year a survey is published by the D.V.G.W. of the appliances (manufacturers and types) that come up to the demands made. In the Netherlands and in Belgium (Ref. 12) there are a number of water companies which occupy themselves with the framing of regulations for so-called internally foolproof appliances. One development should be applauded particularly; that special appliances, among which are those named above, should only be allowed to be fitted if they meet the requirements of internal safety. In view of the fact that such appliances as washing-machines and dish-washers are put on the international market, an internationalisation of the regulations is urgently required. In this connection one good development is the drawing-up by the Common Market of similar directives for washing-machines and dish-washers, which in the year 1976 will be introduced officially as law in the countries which have joined the Common Market. It would be far better if these rules were handled at a still higher international level. Fortunately many makers of washing-machines and dish-washers will, when designing new machines, take the claims of the water companies into account.

Responsibilities

In connection with the possible contamination of drinking water from drinking water installation(s), an interesting point is the responsibilities of the various parties involved in the construction of such an installation. In the various countries this may involve several parties. In principle, at any draw-off point in the drinking water installation, it must be possible to draw drinking water which meets the demands that have to be made of it. The total distribution system, except in those countries where they work with an interruption (e.g. England), is seen technically as an uninterrupted whole. In addition there are two owners: the water company as owner of the main pipe system and the consumer as owner of the drinking water installation. Therefore, there are two responsibilities: that one of the water company and the one of the house-owner. On the part of the water company there is the responsibility for the water that is on the point of being supplied, both in a qualitative and a quantitative sense. In addition the water company is responsible for the qualitative aspects with regard to the water supplied.

As to the quantitative aspects, there are generally fewer prescriptions in use. In this connection we have to refer to the regulations for installations in Barcelona (Spain) (Ref. 9). There we can find what minimum quantities have to be supplied by several draw-off points. At the same time a classification is given of the types of dwellings on hand of the present drinking water installation, while in relation to it, the maximum withdrawal at their disposal is laid down. Generally no obligations of this kind are accepted by the water company. In various existing regulations it is stated that sufficient drinking water, being harmless to public health has to be supplied. In England the following phrase is added: "sufficient for the domestic purposes of all owners and occupiers of premises who are entitled to demand a supply for those purposes." In the "Landesgesetzblatt für Wien" of 23rd May 1960 (Ref. 14), it has been pointed out in para. 3 that a claim on a fixed quantity of drinking water or a special pressure cannot be made. In technical regulations for drinking water installations, if existing, the diameter of the main supplying it is chosen by adding a number of so-called consumption units, by which, if pressure permits, a fixed quantity of drinking water is supplied (Switzerland) (Ref. 13).

The problems of "backflow" have, as mentioned before, both qualitative and quantitative aspects. The first care is that a drinking water installation consists of a pipe system of sufficient diameter and that for special apparatus to be connected, the necessary allowances must be made. In several countries there exist directives which are declared binding by the water company concerned when entering into an agreement with the customer. In this connection a greater necessity for the good execution of drinking water installations has been ascertained. In Israel, for instance, new regulations are being prepared; in Switzerland they want to give the existing recommendations the force of law. In Belgium they are working with national directives. In most countries no clear demand, however, is made for the quality of water as it must be supplied at the draw-off point in the drinking water installation. The formula is often "wholesome" water. Further demands can sometimes be made by the responsible minister. This was the case in Spain 2 years ago, when in a situation that cholera was diagnosed, the water at the taps had to contain at least 0,5 mg Cl_2/l.

For the rest, responsibility for the quality of drinking water in drinking water installations has been laid in all situations at the owner c.q. the consumer. One often confines oneself to the owner, who, in this case, by way of the tenancy agreement, can hold the consumer responsible. In Vienna, in addition to a fine, imprisonment may be imposed in a similar situation (Landesgesetzblatt für Wien par. 28, 23 May 1960) (Ref. 14).

Supervision of drinking water installations

Having good regulations for the construction of drinking water installations together with having these installations executed by a well-educated and specially trained staff does not prove sufficient guarantee that every installation has been constructed in the desired manner. On the grounds of past experience, an inspection of the construction, maintenance and modification of the drinking water installation is indispensable.

In various countries the company itself exercises supervision of the installations that are connected to its mains system. This does not mean that with this supervision, the responsibility of the house-owner c.q. the consumer is taken over by the water company! In view of the know-how the water company has of the

FIGURE 6. Categories of premises to be converted, and safeguards to be adopted in connections to mains networks.

Category	Description of the premises	Manner of safeguarding	Minimal interference of w. company		Remarks
			Before and after construction	During use	
A	Dwellings.	Aim at placing a non-return valve at the end of the service pipe, installation and maintenance by the water company.	Advice and testing of the drinking water installation.	No periodic inspection of dr. water installation. Periodic change of non-return valve. Connection of dangerous apparatus must have prior approval. Normally no periodic test of non-return valve at end of service pipe.	
B	Small undertakings and institutions which, as to the danger and the chance of backflow into the mains network, can be put on a level with dwellings.	Non-return valve to be placed at the end of service pipe; installation and maintenance by the water company.	Advice and testing of dr. water installation.	Random spot check of installation. See further under category A.	
C	Undertakings and institutions where they do not work with dangerous matter.	See under category B.	See under category B.	Periodic inspection (once in 1 or 3 years) of installation and check of safeguards. Periodic change every 3 or 5 years of non-return valve at the end of service pipe. Every modification of the installation must be notified.	
D	Undertakings or institutions where matter is present which in case of backflow into the grid would cause danger to health of consumers.	Non-return valve to be placed at the end of the service pipe. Installation and maintenance by the company. Break (via a reservoir) near the dangerous object, or break (as near as possible to the end of the service pipe) of the whole installation.	Advice and testing of the installation.	Yearly inspection of the whole installation (both upstream and downstream of the break). Every change to the installation must be notified.	If BEFORE the break, hygienically inadmissible situations are found, modification of such situations can be forced by the w. company (in the last extremity the supply can be turned off). If AFTER the break hygienically inadmissible situations are found, advice will be given by the w. company. If the advice of the w. company is not followed, the Labour Inspectorate (e.g. with factories and the like) or the competent Health Officer (e.g. in hospitals, etc.) must be informed.
E	Undertaking or institution where inspection of the drinking water installation or part of it by the water company is in fact impractical, e.g. extraordinary complexes of which it is to be expected that frequent changes will be introduced without forewarning to the water company.	Non-return valve at the end of the service pipe, installation and maintenance by the water company. Break (via a reservoir) before the unverifiable part of the break (as nearly as possible at the end of the service pipe of the whole installation).	Advice and testing of the installation in co-operation with the technical service of the undertaking concerned.	Yearly inspection of the directly connected part of the installation. Every change of the installation before the break has to be notified in advance.	To the undertaking in question and eventually to the Labour inspector or the Health Inspection, a communication must be made that the installation has not been inspected by the water company. In addition the water company must be willing to give advice on special problems.

quality and the quantity of the water supplied by it, this supervision is considered to be of social importance. In most countries the state supervision of public health is in any case—with or without the inspection by the water company—equipped to intervene in cases of unpermitted situations. The water company can and will co-operate in similar situations by turning off the supply to the premises in question.

Besides making inspections we have to find an answer to the question in what measure, and at what periods the inspection of a drinking water installation has to take place. Here, too, a relationship has to be found with the possibility of dangers to public health. About this last aspect a growing concern exists owing to:

(a) the many new appliances placed upon the market some of which are not altogether undangerous from the point of hygiene;
(b) increasing do-it-yourself activities by non-experts that we have to accept;
(c) internal installation activities by the technical services of large industries and institutions, e.g. factories, laboratories and hospitals.

In the Netherlands detailed advice was given in 1975 by the Union of Proprietors of Water companies to their members with regard to this matter. This advice is set down in fig. 6.

In this advice the premises connected are subdivided into some 5 categories, depending on the nature of the possible danger from backflow into the mains system. For each category the means of safeguarding the mains network together with the measure of the inspection of the drinking water installation has been indicated. In the case of a higher class of hazard the premises have to be visited more often.

In practice one has observed that, for example, yearly supervision of a "dangerous" connection is of real interest. A continual flow of information to those concerned about dangerous situations is an absolute necessity with this work. A good relationship has developed with the technical services of those undertakings or industries which formerly carried out incorrect installations. For the sake of completeness one should observe that the advice given in fig. 6 has to be seen as minimum conditions in the Netherlands.

Recapitulation

In the last decade an increasing interest has been observed in the construction and maintenance of drinking water installations.

For the construction of good, and for sanitary reasons, safe water installations one has to comply with the following conditions:

(a) to have available good instructions for the construction and maintenance of drinking water installations, which refer both to the quantity and the quality of the water that has to be supplied;
(b) the construction of the installations has to be carried out by skilled staff;
(c) drinking water installations have to be connected in such a way that the chance that they will influence adversely the quality of the water to be supplied is reduced to a minimum;
(d) the construction of domestic appliances should be such as to be fitted by preference with an "internal" safety device;
(e) the supervision of the construction and the modification of drinking water installations must take place in a measure and with a frequency directly related to the possible dangers that may appear in the installation;
(f) the safeguards c.q. protected appliances should be tested preferably by a central inspection service.

The demands judged necessary for it should preferably be organised on international lines.

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Résumé

Depuis le congrès de Barcelone, où M. G. J. Angeli Sr. P.E. a traité le sujet "Protection des réseaux de distribution d'eau contre la pollution par retour d'eau", environ 10 ans se sont écoulés. On a constaté pendant cette période un intérêt croissant pour les installations d'eau potable.

Les spécialistes anglais qui ont étudié les dangers qui menacent la qualité de l'eau potable par les appareils d'utilisation sont parvenus à des conclusions intéressantes.

Les dangers éventuels qui peuvent menacer non seulement l'eau potable dans l'installation, mais aussi directement le réseau de distribution lui-même sont subdivisés en trois classes de danger, indépendamment des conséquences possibles qui peuvent résulter des joints défectueux.

Un aperçu a été donné des protections possibles qui peuvent être appliquées pour les différentes classes de dangers. Le rapport traite plus amplement les expériences aux Pays-Bas avec les nouveaux types de clapets de non retour hygiéniques qui peuvent être utilisés en combinaison avec le compteur d'eau. Il recommande de munir les appareils qui doivent être reliés aux installations d'eau potable d'une protection interne, dont il signale spécialement les développements favorables en Allemagne.

Il plaide pour une solution internationale de ce problème. La responsabilité de la qualité de l'eau qui est livrée au robinet, doit être divisée entre le service d'eau (jusqu'au point de livraison) et le propriétaire c'est-à-dire le consommateur (installation intérieure).

Malgré de bonnes prescriptions pour la construction et l'entretien des installations d'eau potable et malgré la bonne formation du personnel qui doit être chargé de ce travail, une instance d'inspection est nécessaire. D'après un exemple, on indique plus amplement sur quel rythme le contrôle doit avoir lieu.

En outre la liaison entre les responsables de cette inspection et la direction des services d'entretien des établissements industriels est importante.

Pressure control in distribution systems

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1 Introduction

The following paper is merely a contribution to an ambitious subject. Experiences are drawn from operational aspects of the water supply to the large city of Stuttgart which has differences in elevation of more than 300 m. In normal circumstances, 90% of the city's water supply is imported in two long-distance supply lines, whose delivery points are on the highest peripheral locations of the supply area, with the result that a good deal of the water has to be reduced step by step to the pressure of the respective supply zone. The city's own waterworks, which are in lower-lying locations, serve to cover peak consumption, the supply of several large-scale users, and as a reserve supply. This water has to be pumped to the higher-lying consumption points.

In treating this subject, both technical and economic aspects of the supply pressure, as well as legal ones are considered.

2 Supply pressure

The overriding principle of any water supply is to guarantee the drinking water requirements at every location within the supply area and to ensure that it is at all times of sufficient quantity, best possible quality and adequate supply pressure (4).

By supply pressure is meant the static head p/γ for the consumer × with the geodetic height z of the highest consumption point. To transport the required amount of fluid to the consumer, the resistance caused by fluid friction has to be overcome, i.e. hydraulic energy is lost, which is denoted as loss of head. The energy balance is represented by the well-known equation

$$p/\gamma + z + h_v = H$$

(Velocity head disregarded)

h = Difference in height between two points
z = Height related to sea level
p = Normal force exerted by water on the surface unit
γ = Specific gravity
h_v = Pipe friction loss

A water supply consists of a collection of elemental systems; for treatment, boosting, pressure reduction, transport, storage, measuring and control. These are joined to supply the required amount of water at the necessary pressure.

The meaning of pressure control in this respect is the influence on and control of all elements which guarantee the supply: irrespective of the height of the consumption point.

The necessary supply pressure is not, however, accurately defined in Germany. It should also be taken into account that a pressure once set cannot be kept at the desired level without control.

There are, however, points of reference for the pressure limits:

1. Disregarding negative pressures (vacuums), which should not appear in any drinking water supply, because of the danger of germs and other extraneous materials penetrating into the pipe network, the lower pressure range is defined by the "minimum flow pressure" for the highest outlet point of a building. The minimum flow pressure amounts to 0,1 bar (tap)–2,5 bar (gas operated flow heater).
2. The highest permissible operating pressure is defined as follows in a German standard (DIN 1988): "Pipe and ancillaries should be designed for at least 10 bar internal pressure, insofar as higher operating pressures do not require larger dimensions."

Exceptions to this are boilers, which are frequently only designed for an internal pressure of 6 bar and which then mostly require fittings to reduce a higher pressure to 6 bar.

Whilst an operating pressure in the supply network of up to 10 bar is completely permissible, economic reasons and noise protection dictate against the supply of water at a pressure in excess of 6 bar. Generally, pressures between 2 and 8 bar are the rule in the supply network.

3 Pressure control

The following considerations are features of pressure control:

1. Pressure control is the requirement for *technical operation.*
2. Pressure control enables an easy to view, *economic operation.*
3. Pressure control in the sense of the *reduction of excess supply pressures* has advantages for:
   —the life of pipelines and connections,
   —the protection of consumer devices and installations,
   —the reduction of losses from leakages.
   and water is consumed more sparingly.
4. Pressure control is evidence of the correct and regular operation of the water supply, e.g. also from a legal point of view.

3.1 Pressure control in the technical operation

The design basis for much water supply equipment is the fluctuating demand for water. The flow or delivery rate results from this. The flow is significantly determined by consumption. Consumption, on the other hand, shows marked alterations, despite statistical superimposition of all individual consumptions, especially when comparing shorter intervals of time. To be able to cover peak consumption at all times, it is, however, not necessary or economic to construct the system to the absolute peak. The interval of time to be used for design is dependent on the type of system and topographic or other conditions [18]. The capacity of the various parts of the system is hence better exploited and economy increased. Occasional slight, short-term pressure reductions are quite permissible.

The ratio of flow and pressure drop, using the annual duration curve for the peak year 1964 as an example, as well as the temporary pressure fluctuations, which are always present in the network, are presented in Figs. 1 and 2.

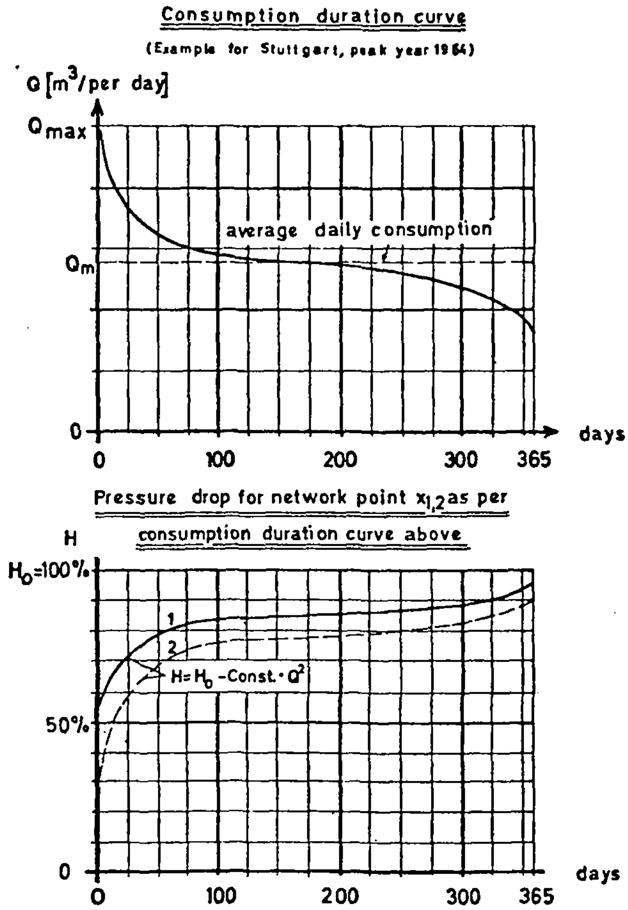


Figure 1

Large pressure fluctuations can have the following causes:

—Start up or shutdown of connection, treating and delivery systems
—Filling and emptying of lines or flushing
—Vibration of control or cut-off devices (e.g. float valves)
—Pulsations in the pump pressure lines
—Incorrect layout of systems' parts
—Fracturing of drive or control elements
—Consumption fluctuations
—Drawing off water for fire-fighting
—Pipe fractures
—Accumulations of air in lines (high points)
—Power failure

3.2 Pressure changing systems

All equipment, with which or via which a specified pressure is set or maintained, is pressure changing equipment, as for example

—Pumps and turbines
—Control valves, such as cylindrical piston valves, pressure relief valves
—Other fittings and
—Tanks

Problems with pressure reducing and booster systems, for operating conditions of larger lines and in the network respectively are to be touched upon in the following.

3.2.1 Pressure relief in feed lines

These are defined as lines with an internal diameter of approx. 600 mm and more, which feed into a tank. With this procedure, a considerable energy potential has very often to be reduced.

Both turbines and special control elements are used to transform pressure energy into velocity energy. An economic assessment should decide which solution is preferable in individual cases. The cylindrical piston valve to a special design is discussed as an example of a control element.

3.2.1.1 Cylindrical piston valves

Fittings which are to be used as a control element should meet the following requirements:

(a) Good control behaviour, i.e. the opening and closing procedures in the fittings are tuned to the pipeline, the respective flow rate and the opening or closing time such that water hammer only occurs within permissible limits. The shortest closing time can only be reached with constant lag for a specified water hammer increase, i.e. by steady velocity change in the pipeline. This condition should only be produced in the control element itself. In the case of supply lines with greater pipe friction losses use is made of an adjustment with a non-linear control unit opening function, i.e. a time or path dependent function.

(b) continuous operation over a broad opening range (stroke ratio s/s_0) without cavitation and with the least possible vibration.

(c) Trickle-tight seal.

While the conditions of (a) and (c) can be met most satisfactorily with a cylindrical piston valve (also called cylindrical piston gate) with a normal plunger, and (a) furthermore with a stepped law of closure, (b) presents problems if a relatively low local counter pressure is present with a tank of max. 4–5 m water level. The evaporation pressure of the liquid is partly attained as a result of the velocity head. Bursting of the vapour bubbles is accompanied by considerable noise generation. Pulsating of the interference leads to impermissible vibrations in the various parts of the system. Material destruction is the consequence.

These phenomena, which can considerably hinder the operation, can be reduced under certain conditions using a fitting provided with a flow-promoting shut-off device, and which for instance has orifice plates [14].

At very high pressure energies orifice plate elements are also used independently of a fitting, thus becoming non-controllable energy converters. Several elements such as these can be connected in series to form so-called cascade throttles.

With the cylindrical piston valve with orifice plates a cylinder with a certain number of concentrically arranged bores is mounted on the axially displaceable plunger (see Fig. 3).

In the closed position the piston sits on the sealing ledge. When opening, the cylinder first opens up, exposing in each case a discharge section corresponding to the sum of the opened orifice sections.

The liquid flowing from the upstream pressure side passes through the bores in a large number of thin jets, which disperse from outer to inner into the cylinder. The jets, which hit each other concentrically at great velocity, are decelerated. In doing so, part of the velocity energy is converted into thermal energy, deformation energy and sonic energy.

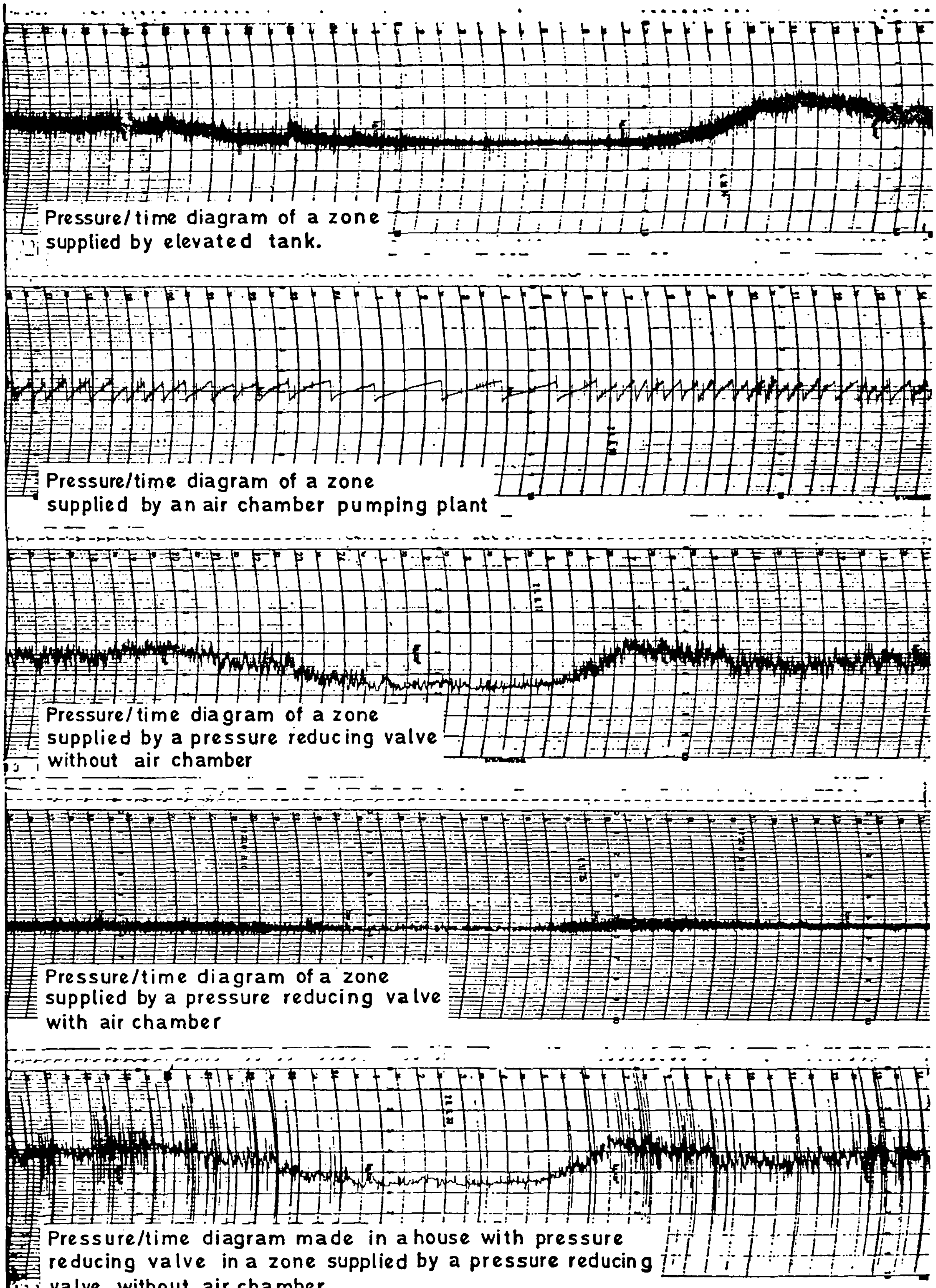

Pressure/time diagram of a zone supplied by elevated tank.
Pressure/time diagram of a zone supplied by an air chamber pumping plant
Pressure/time diagram of a zone supplied by a pressure reducing valve without air chamber
Pressure/time diagram of a zone supplied by a pressure reducing valve with air chamber
Pressure/time diagram made in a house with pressure reducing valve in a zone supplied by a pressure reducing valve without air chamber

Figure 2

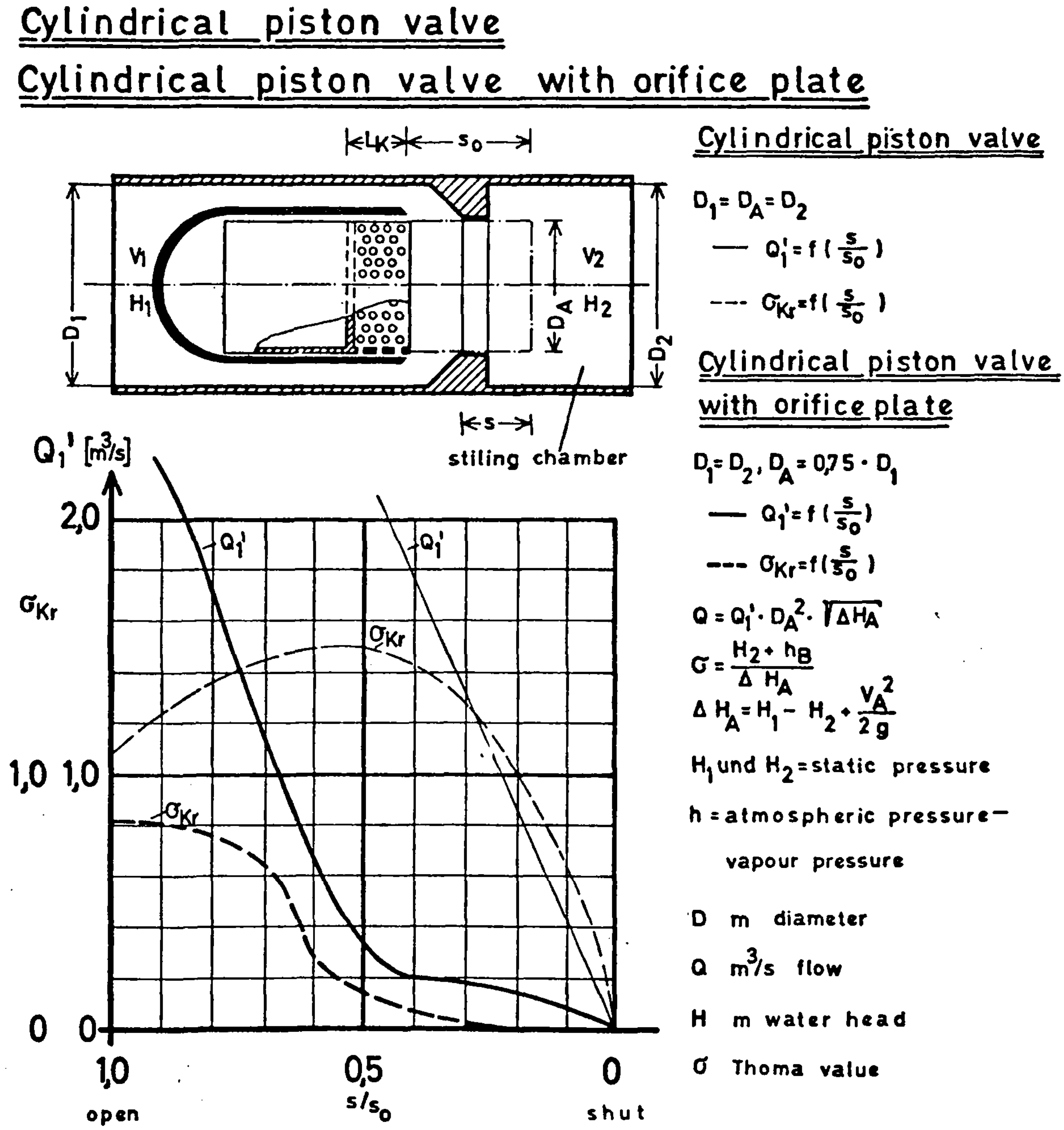


Figure 3

If the chosen gate is large enough, the whole operating quantity can be passed through the orifice plate cylinder. Otherwise, the cylinder has to be opened far enough for the normal cylindrical cross-section of the valve to be opened via the last part of the flow path.

Which size of valve is to be preferred in the planning should be based on the hydraulic system data and is not least a question of costs.

Should a permanent operation of the valve be necessary or possible also in the operating ranges with partial opening, the question of whether cavitation might in fact occur in the operating range should be clarified. In this respect the characteristic value σ (Thoma value) should be considered. The continuous operating range is defined by the cavitation factor σ.

The σ value for the system is defined by:

$$\sigma_{system} = \frac{H_2 + h_B}{H_1 - H_2 + \frac{C_A{}^2}{2g}} \geq \sigma_{perm.}$$

The condition that no cavitation occurs at the point of least pressure is as follows:

$$H_2 \geq \sigma_{perm.} \times \Delta H_A - h_B$$

$$\Delta H_A = H_1 - H_2 + \frac{C_A{}^2}{2g}$$

H_1, H_2 = Static pressure upstream of valve or end of stilling basin, mWH

C, C_A = Velocity, end of orifice plate cylinder, m/s

σ = Thoma value

h_B = Air pressure minus evaporation pressure, mWH

g = Acceleration due to gravity m/s^2

In Fig. 3, the flow ratio Q_1' depending on the stroke ratio s/s_0 for $\Delta H_A = 1$ m, $D_A = 1$ m is drawn both for a valve in normal design and for one with orifice plate cylinder. The most favourable path of the limit curve $\sigma_{critical}$ can be seen for the cylindrical piston valve with orifice plate.

"Goldberg" plant (Diagramatic view Fig 4)

The Goldberg tank with only 490 m^3 content, serving as pressure controller and corresponding tank to a larger tank six kilometres away, supplies a large suburb of a town with approx. 125 000 inhabitants. A cylindrical piston valve (nominal dia. 300 mm) with orifice plate operates as a reducing valve in the inlet to the tank, as described above. A second valve serves as a stand-by or operates alternately. The max. flow rate for the control fittings was specified as $Q_{max} = 0{,}834\ m^3/s$ with a static pressure difference of $H_{geo} = 100$ m and a max. pressure loss in the 2 km long feed line (nominal dia. 800/650 mm) of $h_v = 8{,}0$ m water head. The outlet goes into an ante-chamber with a water level of 4 m. The latter is connected to the main chamber by an overflow. The valve is controlled according to the water level in the main chamber, which should neither empty nor overflow. The closing time is fixed at 50 secs, with a max. permissible water hammer of $\Delta H_A = \pm 15$ m water head.

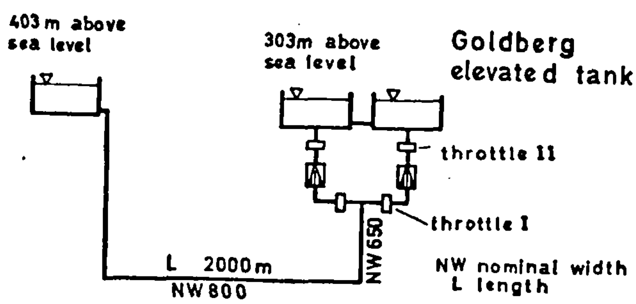


Figure 4

With a static counter pressure of only 4 m water head, relatively large ΔH_A values result, with the result that with correspondingly small σ values for the system, the σ value falls below $\sigma_{critical}$ at a certain partial opening range s/s_0. As the pressure reducer must operate cavitation-free in all operating ranges, it has been constructed as a two-stage design with upstream and downstream throttles. It has been shown that after start-up the system's σ value should be improved, e.g. by installation of a fixed orifice plate.

3.2.1.2 Turbines

Turbines can also operate as pressure reducers. The reduction of the pressure energy potential can in this case be used economically. Turbines, however, require more investment and higher operating costs. Moreoever, their control range is smaller.

The real task for turbines in a water supply system is pressure reduction. The electrical energy produced is in this sense a "waste product". Even so a waterworks' turbine cannot be operated with other than normal economy.

When specifying the size and number of turbines to be used, the starting point is the development of the water requirement over a certain period of time. The average consumptions $Q_{m\,1,2}$ ($Q_{m\,2}$ after 20 years) and the summer peaks resulting from them $Q_{max\,1,2}$ are considered (where $Q_{m\,1}$ and $Q_{max\,1}$ are current mean and maximum consumptions, and suffix 2 refers to a time 20 years later).

The result of an economic comparison derived therefrom is that a turbine, despite less use and lower efficiency, should be designed as per $Q_{max\,2}$ (see Fig. 5, and for profit and cost comparison curve Fig. 8).

A turbine fitted as a reserve unit does not operate economically. The reserve unit should therefore be designed as a control fitting. In Fig. 6, units with equal capacity are compared in each case ($Q_{max\,2}$ from Fig. 5).

Remote monitoring and control of turbines requires additional investment, the capital costs of which in fact correspond to the saving on personnel costs involved. Turbines should therefore be used as far as possible in systems in which operating personnel are used already (see Fig. 7 in comparison to Fig. 6),

Control fittings as a reserve unit cannot be dispensed with under any circumstances, as shutdown times for repairs cannot be disregarded. In a great number of applications, control fittings will remain the only suitable unit.

Profitability comparison between turbine systems with differing design

Turbines designed for	$Q_{m\,1}$ 417 l/s	$Q_{m\,2}$ 624 l/s	$Q_{max\,1}$ 742 l/s	$Q_{max\,2}$ 1 125 l/s
Investment costs per turbine system	DM 500 000	DM 600 000	DM 650 000	DM 700 000
Interest plus 12% depreciation per year	60 000	72 000	78 000	84 000
Operating costs/year	18 000	18 000	20 000	20 000
Maintenance costs/year	2 500	2 500	3 000	3 000
Total costs/year	80 500	92 500	101 000	107 000
Credit from power sales	209 320	256 492	268 132	283 758
Profit/year	128 820	163 992	167 132	176 758

Invoicing of electricity costs for day tariff = 0,09 DM/kwh
for night tariff = 0,05 DM/kwh

Night tariff applies between 21.00 and 06.00

Fig. 5

Profitability comparisons between turbine systems and pressure reducing systems

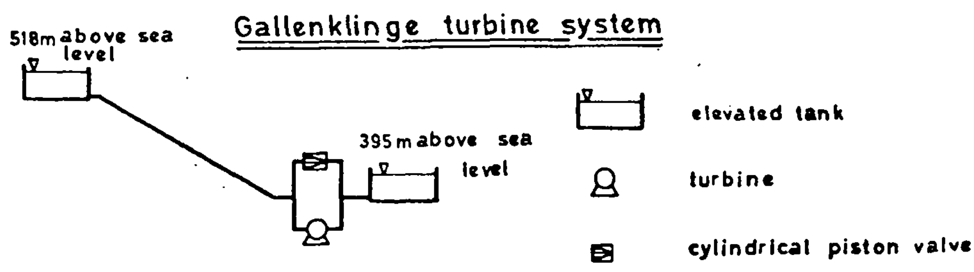
	Waterworks operation			Power station operation
	2 turbines	1 turbine + 1 pressure reducer	2 pressure reducers	1 turbine
Investment costs	DM 1 400 000	DM 700 000	DM 250 000	DM 650 000
Interest + 12% depreciation	168 000	84 000	30 000	78 000
Operating costs/year	40 000	25 000	15 000	20 000
Maintenance costs/year	6 000	4 000	2 000	3 000
Total costs/year	214 000	113 000	47 000	101 000
Credit from power sales	160 000	160 000	—	415 000
Residual expenditures	54 000	—	47 000	—
Profit/year	—	47 000	—	314 000

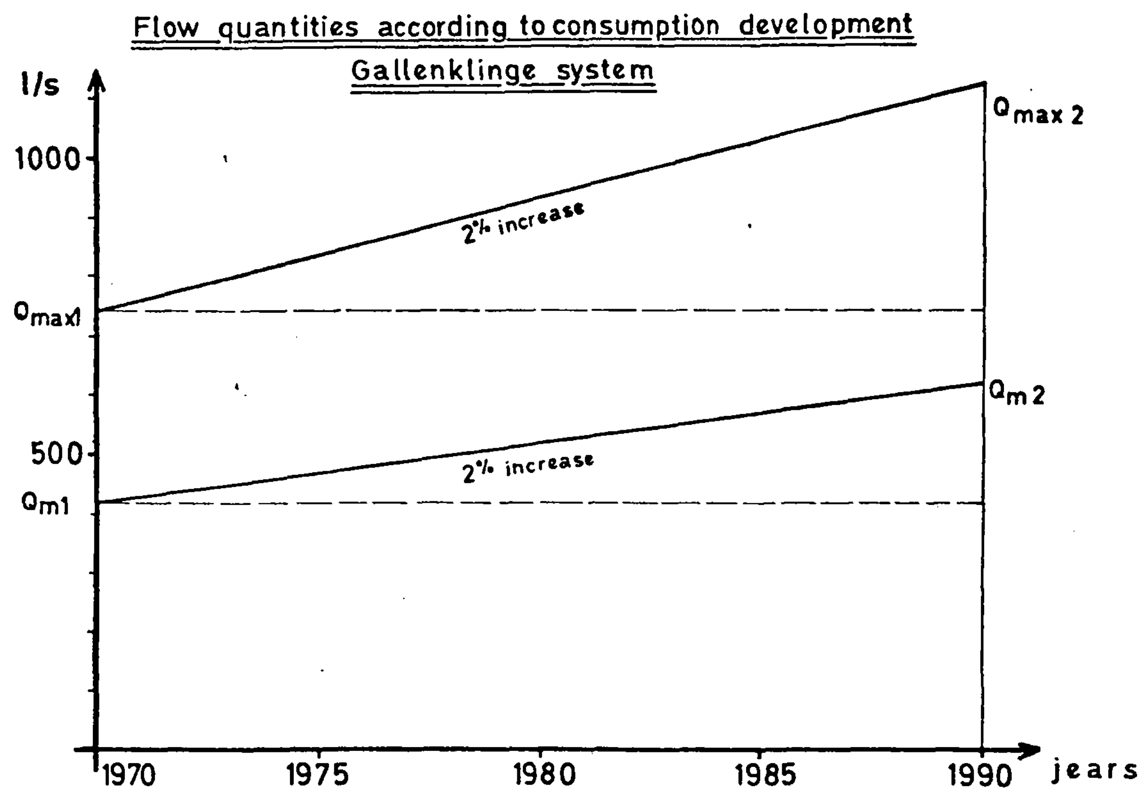
Fig. 6

Profitability comparison between turbine and pressure reducing systems with remote control through a central control station

	Waterworks operation			Power station operation
	2 turbines	1 turbine + 1 pressure reducer	2 pressure reducers	1 turbine
Investment costs/year	DM 1 500 000	DM 820 000	DM 290 000	DM 700 000
Interest + 12% depreciation/year	180 000	98 400	34 000	84 000
Operating costs/year	20 000	12 500	5 000	20 000
Maintenance costs/year	10 000	7 000	4 000	10 000
Total costs/year	210 000	117 900	43 000	114 000

Fig. 7


Gallenklinge turbine system
518m above sea level
395m above sea level
elevated tank
turbine
cylindrical piston valve


Flow quantities according to consumption development
Gallenklinge system
l/s
1000
500
Qmax1
Qm1
Qmax 2
Qm 2
2% increase
2% increase
1970
1975
1980
1985
1990
jears

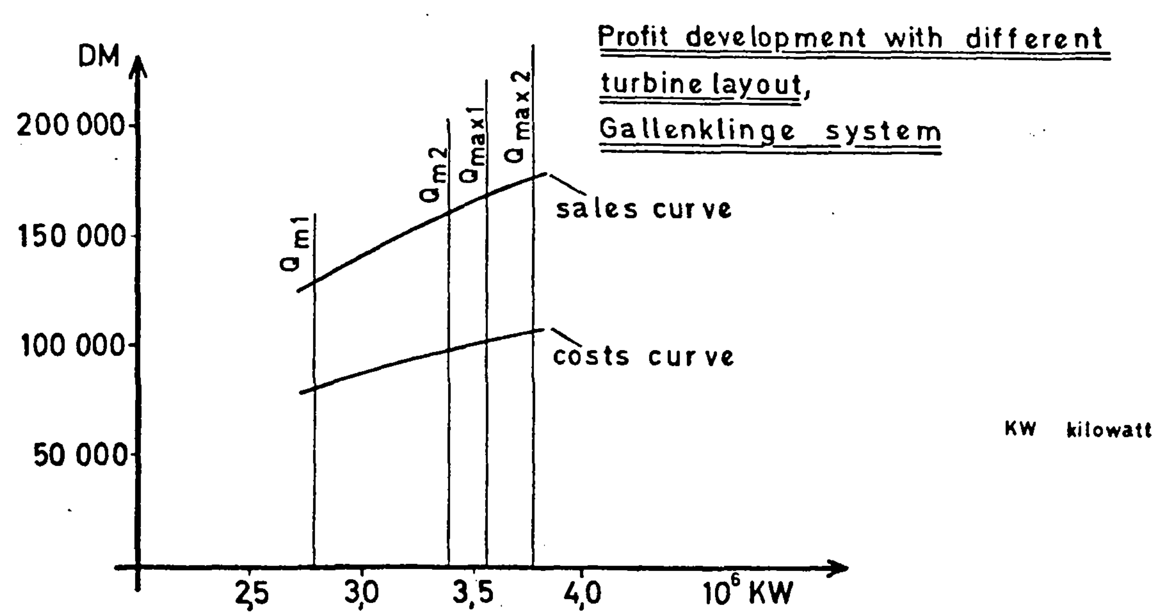

Profit development with different turbine layout, Gallenklinge system
DM
200 000
150 000
100 000
50 000
Qm1
Qm2
Qmax1
Qmax 2
sales curve
costs curve
2,5
3,0
3,5
4,0
10⁶ KW
KW kilowatt

Figure 8

3.2.1.3 Pressure reducing valves in networks

It is proposed to deal only with pressure reducing valves smaller than those described in 3.2.1.1 which control automatically, i.e. without extraneous auxiliary energy. The auxiliary energy is removed from the controlled system. The control always takes place steady proportionally or proportionally integrally according to the selected downstream pressure.

Pressure gradient and flow rate are the criteria for selection of appropriate size.

The first uncontrolled pressure reducing valves were built as early as at the turn of the century. Controlled valves have been well-known in Germany for 25 years.

The most important demands on pressure reducing valves are [10, 11]:

1. The selected pressure should remain as constant as possible with fluctuating consumption and changing upstream pressures.
2. Tight seal at zero consumption and even over a long operating period (the pressure should not be transmitted).
3. Operational safety.
4. Inexpensive maintenance.

Pressure reducing valves are used particularly downstream of long-distance supply system branches and in municipal networks with large differences in elevation, either for the supply of pressure zones, insofar as the water is available on a higher level, or for the reduction of pressure in individual buildings as a house pressure reducer.

Whilst pressure reducers have their limitations, they also have the following advantages:

—They are relatively cheap and require no costly structures.

—They are self-controlling and can be re-adjusted within certain pressure limits.

—Under certain pre-conditions, they offer the possibility of a small pressure division, where there are large differences in elevation within the supply area.

—They allow pressure compensation to overloaded or distant network components, which are supplied via water tanks.

—They protect elements of the system against excess pressure and as such also serve to cut down on noise.

At present the pressure reducing valve does not yet offer the operational safety of a tank, which is exposed to atmospheric pressure and whose storage effect is a pre-requisite for a continuous supply. The pressure reducing valve is an essential aid for the applications considered above, under the following conditions:

—Feeding in against closed network only wherever short-term transmission of the upstream pressure does not give grounds for any serious damage (otherwise additional safeguarding by a safety valve, insofar as the latter can still draw off the water up to a certain upstream pressure, in case the valve should fail),

—Use of pressure control tanks with long feed lines,

—Regular maintenance,

—Shutdown times are not too long,

—No overdimensioning.

The pressure reducing valve with external, auxiliary energy has even better control characteristics and can be controlled according to the pressure profile of a distant network point. The operational safety of the pressure reducing valve with external energy is not, however, increased in comparison with that without external energy.

The possibility of failure of the pressure reducing valve must be considered and corresponding precautions taken. A disturbance in this case means more than just shutdown. It means possible impermissible pressure rise in the pipe network and household installations. If this is in excess of the stress limit of the pipelines and consumer devices corresponding damage will occur.

3.2.2.1 Pumps with long pressure lines

With the operation of pumps in long pressure lines, particular attention must be paid to the non-steady flow conditions. These include start up and shutdown of the pumps, and also control of them by means of changing the pump speed or by actuation of a control valve. If the speed decelerates rapidly, and as the delivery head of the pumps decreases as the square of the speed of rotation, an oscillation is caused in the water column. With longer lines, the full, direct, negative water hammer can take effect, as the discharge within the reflection time of the pressure waves returns to zero [7]. The effects of the velocity change are obviously different for individual systems, as also are the measures which should be taken to keep the pressure change within permissible limits. In any case, the potential damage from underdimensioning should be avoided just as much as the excessive system costs due to overdimensioning.

The permissible pressure change in the pipeline is derived from two conditions:

Tear-off condition: the pressure line should not drop below the height of the pipeline at any point.

Max. pressure condition: max. pressure head to be expected should not exceed the permissible operating pressure (rated pressure) of the line.

An assessment of whether there is a danger of water hammer can be derived from the formula $K_2 = (L \times c)/\sqrt{H}$ (length of pipeline L and delivery head H in m, flow velocity c in m/s), which for $K_2 > 70-100$ necessitates the installation of water hammer safeguards [12].

In addition to relief valves, energy accumulators are also very frequently used as water hammer safeguards. Their function is the control of the ratio of the kinetic/translational energy of the water in the line, to the kinetic/rotary energy of the pumping unit.

Energy accumulators are:

a flywheel with the flywheel mass of the pumping unit for the storage of rotary energy;

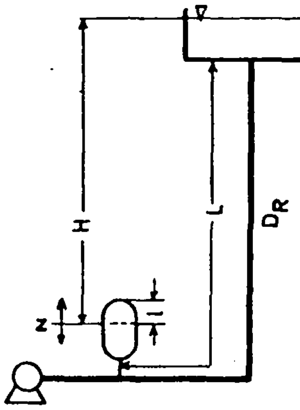
an air chamber for the storage of hydraulic energy.

Flywheels

Flywheels are suitable for horizontal pumps with higher speeds at average or large delivery heads and pipe lengths as far as possible not in excess of 3 000 m.

The determination of the pressure drop at pump run-down can be worked out using electronic computers. In practice, however, the graphic method of Schnyder–Bergeron [15, 16] is still proving very useful. The speed change during the pump rundown process can be determined using an iteration method or more quickly still with the aid of a ω–K diagram [12]. The approximate speed of rotation after the first reflection period is found using the values for the pipeline length, the velocity of the pressure transmission and the torque and flywheel effect of the pump. In the c–H diagram, the point of intersection of the water hammer straight lines with the

Diagram of a water supply system with pressure control tank



H the static pressure head in m/wh

L length of line in m

D_R diameter of pipeline in m

l height of air space in air chamber in stationary condition in m

z level deviation

Figure 9

corresponding c–H line for the pump obtained from the relationship $\omega/\omega_1 = c/c_1 = \sqrt{(H/H_1)}$, gives the pressure drop after the first reflection period.

Air chambers

The air chamber is used with great success as a damping element in reducing stations with long supply lines and also as a control element for zonal pumping plants. Its main application is water hammer protection for pump pressure lines, Fig. 9.

There are several design criteria for the air chamber in pump pressure lines. In addition to the breaking and rated pressure conditions, there is a third condition, namely that the volume of the air chamber should be large enough to ensure that no air can escape into the pipeline when it is released.

It has become common practice to select the volume of the air chamber to within 1–2% of the content of the pump pressure line to the tank, as an initial approximation. However, in the majority of cases, a more accurate calculation of the volume of the air chamber should be made. However, to date no success has been achieved in combining all the phenomena of the oscillation process into one single formula. The step-by-step calculation of the Schnyder–Bergeron graphic method gives good results. The water which flows out from the chamber after pump failure results in a reduction of the volume of water and in a change in the pressure-volume state of the gas cushion in the chamber. This change of state, assumed for several volumes within the time unit, results in part of a curve in the c–H diagram, which at the intersection with the water hammer straight line, gives the pressure drop after the first reflection period (if this was chosen as a time unit). The other points are developed therefrom and, for reasons of simplicity, the change of state can be calculated as an isotherm, although its actual trace represents a variable polytropic curve.

Ludwig and Stack [13] solved the oscillation equation for the isothermic change of state by the incorporation of pipe friction, introducing the oscillation of the water level z in the air chamber as a variable. However, the elasticity of the water and pipeline (so-called "Inelastic Theory" as for example with Boerendans [3] and Evangelisti [6]) is not taken into account. This method gives good results except with small chamber volumes.

Starting from the differential equation for the downward directed motion

$$\frac{d^2z}{dt^2} - m\left(\frac{dz}{dt}\right)^2 + n\left\{p_0\left(\frac{1}{1-z} - 1\right) + z\right\} = 0$$

$$m = \frac{F_w \times \lambda}{2F_r \times D_r} - \frac{F_r}{2F_w \times L}$$

$$n = \frac{g \times F_r}{L \times F_w}$$

L = Length of line
z = Water level oscillation, air chamber
$F_{r,w}$ = Cross-section pipe, air chamber
l = Height of air space in chamber
λ = Characteristic value of friction loss

the solution equations are developed, which implicitly contain the extreme values of z, which can be easily derived by trial and error. Nevertheless, for numerical evaluation, values for the respective integral logarithms should be taken from mathematical tables.

The analogue computer also lends itself to the solution and evaluation of the above-mentioned differential equation. This has the advantage that the effect of parameter change can be rapidly recognised, so that different operating conditions can also be simulated. The results appear as curves on the plotter [17].

Non-return valves

Sudden closing of the valves when the pump fails is hazardous. It occurs if the time in which the velocity of the water column drops to zero (T_R) is shorter than the valve shutting time (T_S). The longer the pipeline, the greater the velocity, the smaller the static pressure and the greater the pump run-down time, the larger T_R will be.

Air chambers shorten the effective length of the pipe to the distance between pump and chamber. Water hammer is therefore the rule here, even weights on the valve lever are not particularly effective. The installation of non-return valves with a flap edge seat on one side should be disposed with. In contrast, nozzle-type non-return valves with spring pre-tension have proven successful. Attention should be given in every case to stable pump characteristics.

3.2.2.2 Pumps in the network

The size and design of a pumping plant are determined by flow rate and height and the type of layout of the pumps with regard to the suction and pressure end,

Four types of layout can be identified [8]:

1. Tank—pump—tank T-P-T
2. Network—pump—tank N-P-T
3. Tank—pump—network T-P-N
4. Network—pump—network N-P-N

The particular situation in a supply area is always decisive for the type of layout. Irrespective of whether the system is necessitated by the development of a built-up area or by its subsequent installation due to development (overloading of the network), the following factors are normally the basis for the layout:

—Topographic position of the respective procurement and storage point, as well as that of the supply area;

—Length of supply lines;

—Size and condition of pipe networks;

—Structure of consumption.

1. $T_1 - P - {}^{N_1}T_2{}^{N_2}$

With this layout it should be made clear whether the tank T_2 is operated as a flow tank, whether the feed line to the tank is only a pump pressure line and whether the network N_2 is fed separately from the tank; or whether T_2 is at the other end of the network as a corresponding tank or whether the pump pressure line is also the supply line. In the latter case, the delivery head will change according to the network consumption requirement; pumps with a steep characteristic should be used.

If the costs of a flow tank and a corresponding tank are compared, the latter is usually cheaper. The take-off line N_2, which is separately laid, is not necessary.

As the network is fed from two sides, the tank take-off line does not need to be designed for peak consumption. A further advantage of the corresponding tank system is the possibility of subsequent correction of the supply pressure by changing of the pumps. With the flow tank, the hypothetical energy curve is finally fixed, while this is super-imposed for the corresponding tank with the sum of all head losses from the network to the tank. The flow tank is simpler to operate, the geodetic percentage of the delivery head is constant and there is less danger of water stagnation during storage.

2. $N-P-T$

In this case, the network pressure is not always sufficient to supply the tank. By corresponding dimensioning of the tank, the pump can be completely adjusted to the network conditions. Pump or supply line failure does not lead to a short term failure of the water supply. The water supply for fire-fighting is in reserve.

3. $T-P-N$

The type of layout with direct delivery into the network is encountered, if

—Sufficient height is not available for the tank on flat ground, or

—Two supply zones are super-imposed in height and the tank for the lower lying zone simultaneously serves as the drawing off tank for the pumping plant delivery into the upper zone.

In the first case, speed controlled pumps are very suitable for the supply of larger parts of the network. Control of the flow is carried out according to water requirements, where in each case only the required delivery head is produced. The speed control can be carried out with gears or by control of the drive motor, which is normally more effective. The basic delivery flow can be met by pumps of a fixed speed. The most frequent form of zonal pumping plant is the air chamber pumping plant, which operates more economically with pressure controlled pumps up to a delivery rate of Q = 100 l/s, than a quantity controlled pumping plant.

The advantage of direct discharge into the network is its ability to be adapted to consumption by the switching on and off of corresponding pumping units. With less consumption and correspondingly less frictional head losses, pumps with less delivery head can be used.

4. $N_1 - P - N_2$

With this layout there must be sufficient capacity present in the network section N_1 to draw off the required amount of water by operation of the pumps, without impermissible pressure drop.

The feeding-in into larger networks is also carried out via speed controlled pumps according to requirements.

"Pipe pumps" have proved themselves for pressure boosting of individual network sections. In normal operation, as pipeline installations, these are bypassed if the pump is shut down, and are only switched on when required.

3.2.2.3 Booster systems on private properties (BS)

The normal operational pressure supply from the waterworks only extends to the highest situated consumer according to local conditions (compare with section 4). Multi-storey blocks can only be supplied with the normal supply pressure up to a height according to local conditions, while the floors above this height have to be supplied via a booster system which belongs to the building itself. Booster systems cause severe localised stress on the network. The following is demanded in a DVGW guideline [5]:

Booster systems should be so laid out, designed, operated and maintained that the continuous operational safety of the water supply is guaranteed and neither the public water supply, nor other consumption systems are interfered with. A subsequent change in the drinking water quality should be excluded.

Interference to the supply network is caused by:

—the possibility of backflow,

—cases where impermissibly large water hammer is produced at switch on and switch off, due either to too large dimensioning or too large a consumption, or if, due to the operation of the system, water is drawn off from neighbouring consumers.

If the network cannot provide a sufficient supply for the booster system an auxiliary storage tank should be used.

The direct connection of the booster system is only allowed if the velocity change in the connecting line, caused by the switching on or off of each pump, does not exceed $\Delta c = 0{,}15$ m/s or if an air chamber is installed on the suction side, or if this is guaranteed by other measures.

Some manufacturers of booster systems describe their product as being water hammer free. This means systems which produce smaller water hammer, insofar as their switching actions occur with smaller volumetric flow or with a time lag.

Many booster systems have pressure controlled switching with a large air chamber installed downstream as a control element. At switch on, full output from the pump is produced, regardless of whether there is consumption or not. If the pumps' dimensions are too large, as is mostly the case, this has its full effect on the network. The demand curve is the result of average peak values, which in practice are not met in every case. On

the basis of constant stress curves [9] for houses with flushing cisterns or flush valves, it can be shown that only the latter bring larger and short-term peak draw-off of water. Pumps for air chamber systems in booster systems should be closely attuned to the anticipated consumption structure and dimensioned accordingly. It is also better to have a stand-by in the chamber for this contingency. This is equally true of booster systems for department stores, office blocks, hotels and hospitals.

With important buildings, the installation of a further reserve pump should be considered to cover occasional, but rare consumption peaks.

Booster systems which have only a small diaphragm tank as a control element on the pressure side or even none at all, discharge only the amount drawn off by the consumer on each occasion. The system only switches off at the minimum consumption, i.e. at a small delivery flow. Water hammer at switch off is therefore normally within permissible limits. If there is a power failure, this system can also produce an impermissibly large water hammer, Fig. 10.

Although a system without an air chamber is better for preventing impermissibly large water hammers in normal operation, it will not displace the booster system with air chamber, because, despite lower investment cost, it cannot attain the profitability of a system whose pumps always operate at optimum efficiency.

4 Supply pressure and profitability

Whilst it is capital costs which significantly mount up with tanks, it is operating costs, in particular energy costs, which mount up in the case of pumping plants.

With current electricity costs (e.g. Stuttgart, 1975: basic charge 12,9 DM per KW and month, operating price 9,7 pfennigs per KWh) and an average pro capita consumption of 240 l/day, every pressure rise of approx. 1 bar provided by pumps results in total costs pro capita and per year of 53,6 pfennigs. All precautions for reduction of energy consumption should therefore be considered at the system planning stage, e.g.:

—Position and arrangement of the pumping plants in the network; number, graduation, control and degree of efficiency of the pumps.

—Least possible distance between the pumping plant and the focal point of consumption. This results in smaller investment costs and, as a consequence of lower pipe friction losses, smaller energy costs.

—Maintaining static pressure head at the absolute minimum necessary to guarantee the minimum flow pressure to the highest consumer in the area.

If consumers are supplied with pumping pressure by a single point to different locational heights, then the same delivery head z_i should be used for them even though this pressure head would only be necessary for the highest consumer. As this is normally unprofitable, the supply area is broken down into supply zones (mostly zones with graduated heights) [1].

The zoning can be caried within the specified permissible pressure limits; a break down into arbitrary numbers of sections of smaller pressure differences can, however, only be continued until the amount of investment caused by the increase in the number of operational devices is counterbalanced by the advantages and savings resulting from a smaller supply pressure. That limit is of

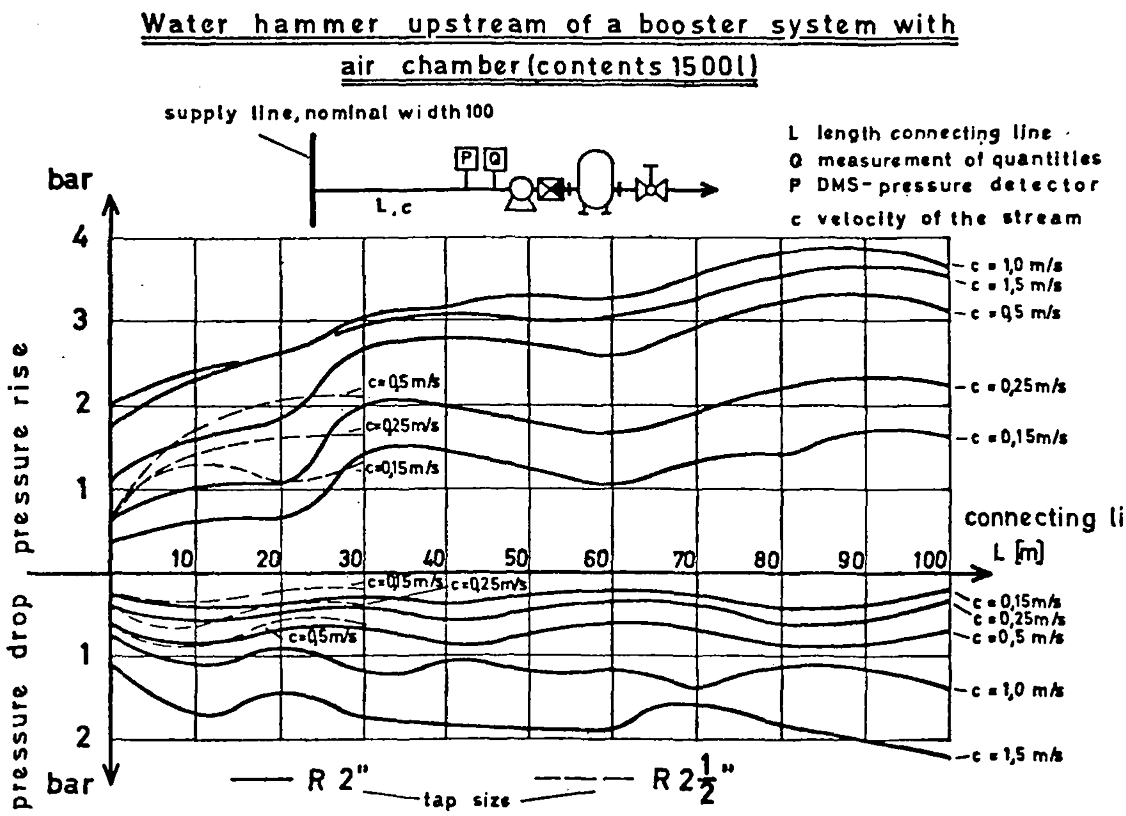


Figure 10

3.3 Control and monitoring

There is available a number of automatic control procedures for the maintenance of a given pressure and other important measuring parameters. Alternatively, control will be undertaken wholly or partly by a central station. Both systems can be manually controlled and require uninterrupted monitoring by the central station. Central, manual control can only be used with very small systems or with a limited number of switch commands. With larger systems, the evaluation of data and control can be carried out via a process computer.

Waterflow - out from tap valves experimentally determined at different pressures

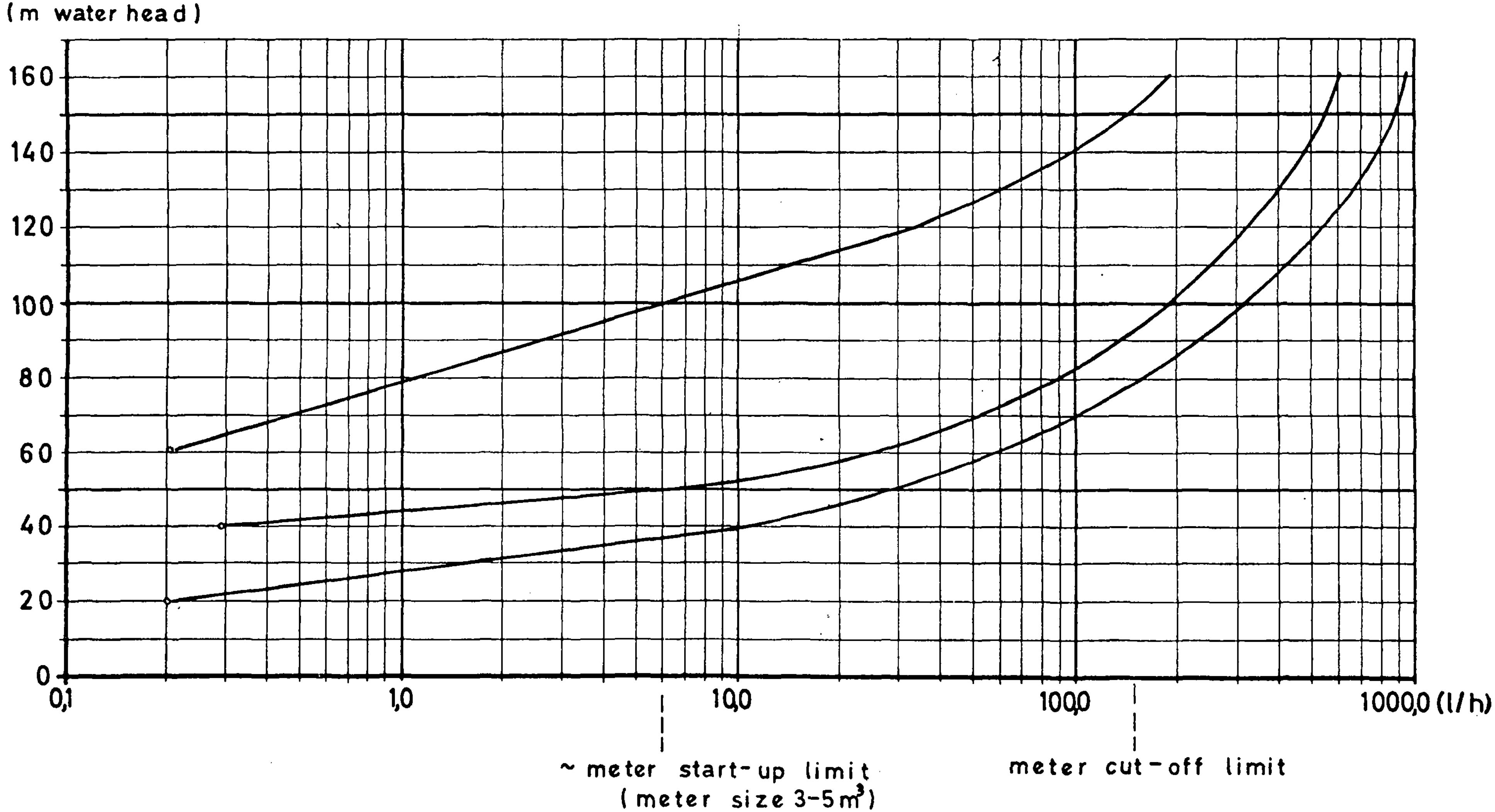


Figure 11

course dependent on local conditions and therefore varies. However, too many zones make operational surveillance more difficult.

4.1 Pressure zones

Supply to the individual zones can be in parallel from a central point or in series from one zone to its neighbouring zone.

A separate pressure line to every individual zone is necessary with parallel connection. This system should only be selected if transporting distances and differences in elevation are not too large. For larger transporting distances and differences in elevation, a series connection is more economical. But here, however, operating stations are necessary in the respective transition.

If the supply pressure is not to exceed 60 m water head (with a minimum pressure for the highest consumer of 20 m water head), the differences in land level may be approx. 40 m. The larger is the permissible max. supply pressure for the lowest-lying consumer the less the number of zones required. Too high pressures cause increased stress on fittings, pipe materials and seals and therefore an increased amount of damage. An increase in consumption and leakage losses can also be proved, as shown in the following.

5 Supply pressure and water consumption

From tests carried out it has been shown that there is a connection between pressure rise and consumption quantity or the quantity delivered into the network. There is no doubt that water losses (trickle losses) and leakage losses (pipe connections, cracks, hair line fractures, pitting) are involved in the increase in consumption.

5.1 Trickle test

An ordinary tap R ½" with normal seal was used. The fitting was connected to a testing system and so adjusted to several set-points (20 m water head, 40 m water head, . . .) that a drop formed every three seconds, following which the pressure was increased in each case.

The following can be read off and deduced from Fig. 11:

With a normal supply pressure, a valve may lose one drop (= 0,3 l/h) every three seconds. If the supply pressure increases during the night over a period of 12 h by 10 m water head, the trickle loss of the same valve correspondingly increases to 6,0 l/h. This leakage loss is too small to be recorded by the water meter and is therefore not paid for.

For a supply area containing 80 000 water meters and assuming one trickle point allocated per water meter, the result is that 2,1 million m^3/a of consumption is not metered (= 30% of the difference between the water supplied and that paid for by consumers).

The above reasoning is somewhat hypothetical because it is not known how many actual trickle points there are. Also only one trickle point is allocated to each property. However, in practice slow losses in flushing cisterns, tap valves etc. can be observed everywhere so that the figure estimated is probably not too far out. The following example bears this out.

5.2 Verification in a block of flats

A Woltmann water head meter (50 mm) was bypassed in an apartment block of 56 flats (built in 1963) and the nightly consumption measured with a 7 m^3 meter. The actual consumption times and the leakage losses can be clearly read off from the diagram (Fig. 12). From the gradient of the straight lines, an overall consumption in four hours of 480 l results with leakage losses of 230 l (= 48% or 57 l/h leakage loss) corresponding to 1 l/housing unit. This value appears extraordinarily large, but confirms the result of 5.1.

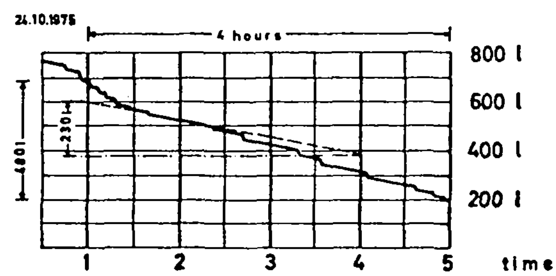


Figure 12

5.3 Zonal consumption with various pressures

In a zone supplied by pressure reducing valves (2 860 inhabitants), the consumption was measured starting from normal pressure and then at smaller pressures reduced in each case by 5 m, 10 m water head and so on. The curve in Fig. 13 resulted from the basic values and clearly shows a fall off in consumption dependent on the reduced pressure.

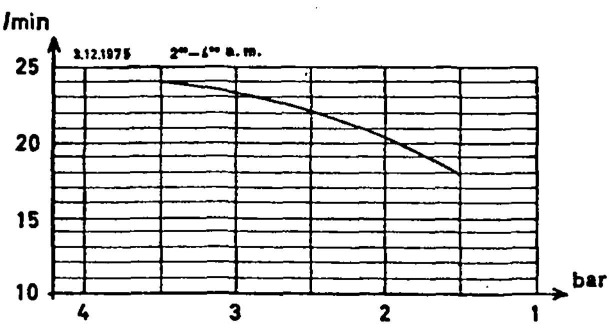


Figure 13

5.4 Zone with air chamber pumping plant

A pressure quantity curve was deduced from the characteristic circuit diagram of the air chamber pumping plant and this is shown in Fig. 14.

The consumption peaks shown during the first 20% of the tap-off phase (nightly consumption) can be reproduced from other graphs. No explanation for the steep initial slope of the curves (increased consumption could be found other than the increased pressure level.

Unfortunately, the example in 5.1 is not sufficient to be able to prove the relationship between the leakage loss and the increased consumption at increasing pressure; the influence of the pressure level on consumption can, however, be shown.

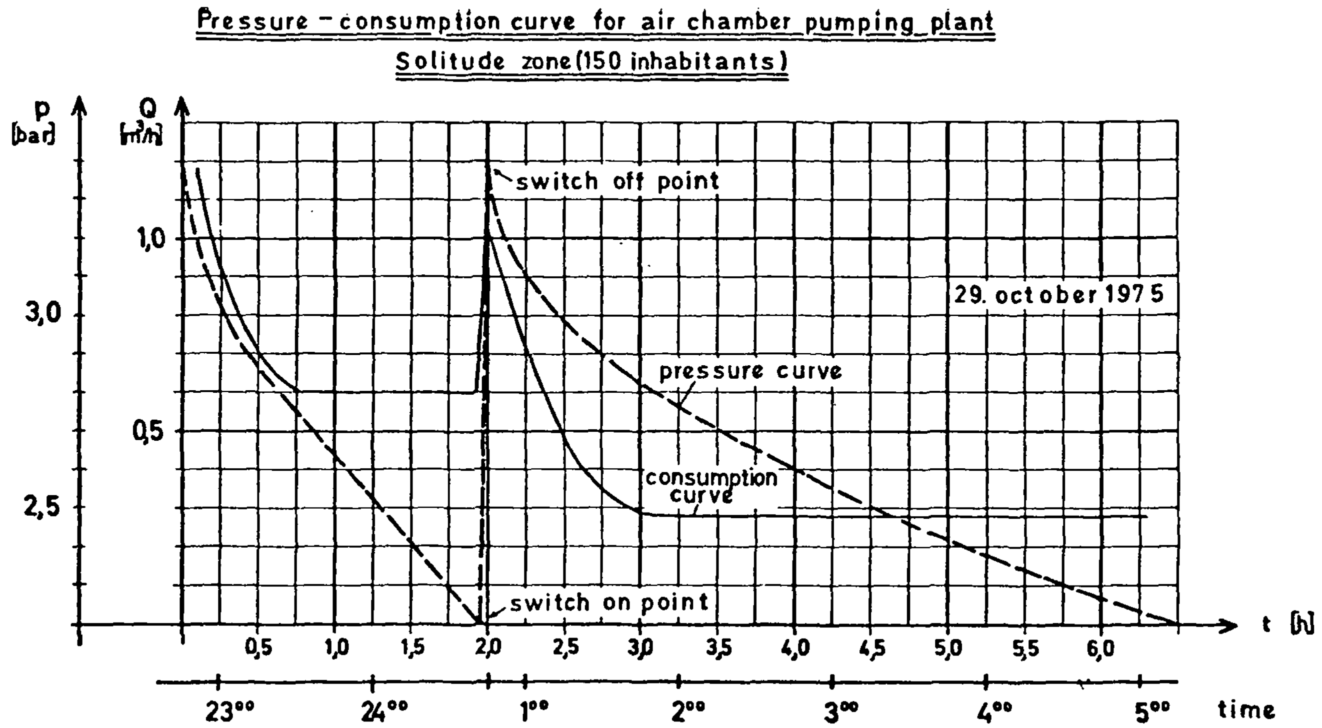


Figure 14

6 Legal aspects of pressure control

In Germany there is liability for risks* in the field of electricity and gas; in the field of water supply, there is liability for intentional acts†. The judiciary in the Federal Republic of Germany has, however, demanded a duty of care of the public supply companies such that there are not too many differences between this and absolute liability in the final analysis.

In the field of electricity and gas, the General Supply Conditions are standardised by the General Liability Declaration of 1942. These regulations have the character of statutory orders and conclusively control the contractual relations between public supply companies and consumers in the Federal Republic of Germany.

No such General Liability Declaration has been made in the field of water. With the General Supply Conditions in this case, these are actual General Operating Conditions, which control the rights and duties of the waterworks and the consumer in civil law. In accordance with this the water supply companies have reserved the right to make pressure changes. It is regarded as an accessory, contractual consideration that the public supply company will inform the consumer about any intended pressure change. The consumers, however, have no claim upon the maintenance of a particular pressure.

If damage occurs due to a change in pressure, it is for the judiciary to decide in each case whether the damage occurred as a result of typical operational dangers for water supply companies (e.g. failure of technical equipment) and therefore whether it is a risk to be borne by the consumer.

To sum up, there are no adequate criteria given by the legislature in the Federal Republic of Germany for the pressure field.

* Liability for risks means responsibility for a certain material or operational danger for social reasons, where a culpable behaviour of the party liable for damages does not have to be given.

† Liability for intentional acts means responsibility for culpable behaviour.

7 Conclusion

In this paper, an attempt has been made to derive the correlation between Pressure Control and technical operation and to look at these from an economic point of view as well. In each case, examples of operating conditions for energy changing systems, both with long lines and in networks have been examined and in addition water hammer problems have been discussed.

The necessity of reducing excess supply pressures as a requirement for the reduction of the energy used in networks supplied with pumping pressure has been discussed, as has the requirement for the reduction of operational malfunctions and network losses.

In the final analysis, Pressure Control and evidence of its preservation by uninterrupted recording of operations is important for protecting the public supply company against unjustified claims for compensation.

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Résumé

En traitant ce sujet, les aspects techniques, économiques et légaux ont été considérés.

Le principe essentiel de toute distribution d'eau est de garantir les besoins en eau potable en tous points de la zone desservie, en quantité suffisante, de la meilleure qualité possible et sous une pression d'alimentation adéquate à tout moment.

Ce que l'on entend par pression d'alimentation est la pression statique présente dans le réseau: elle doit être au moins suffisante pour garantir le minimum de pression de débit aux points de soutirage les plus éloignés et les plus élevés dans la région desservie. La pression maximale admissible dans la réseau est fixée à 10 bars. D'une façon générale, les pressions normales dans le réseau sont de 2 à 8 bars.

Ce que l'on entend par contrôle de la pression est l'influence sur et le contrôle de tous les paramètres, en vue de garantir une pression d'alimentation adéquate en tous temps, quelle que soit l'altitude du point de consommation.

Pour la contrôle de la pression, il faut dériver les relations suivantes: le contrôle de la pression est une exigence pour l'exploitation technique; le contrôle de la pression permet une exploitation claire, facile à suivre, économique. Le contrôle de la pression, entendu comme réduction des pressions d'alimentation excessives, est avantageux pour:

—la vie des conduites et des branchements,

—la protection des appareils et installations du consommateur,

—la réduction des pertes causées par les fuites possibles.

Le contrôle de la pression est nécessaire comme preuve de l'exploitation régulière de la distribution d'eau, par exemple du point de vue légal.

On discute de l'équipement de modification de la pression en technique d'exploitation, par exemple:

pompes et réducteurs de pression, rattachés chaque fois à l'exploitation d'une longue conduite et dans le réseau.

Description est donnée d'une vanne à piston cylindrique avec plaque d'orifice comme exemple de réducteur de pression, dispositif de contrôle à l'extrémité d'une longue conduite avant déversement dans le réservoir.

Les problèmes spéciaux et mesures pour éviter la cavitation en exploitation continue dans les régions où il y a des ouvertures partielles sont discutées pour un cas donné.

Pour l'emploi des turbines en vue de réduire l'énergie potentielle hydraulique, une comparaison économique est donnée à titre d'exemple, avec les accessoires de contrôle comme les réducteurs de pression.

Bien que les turbines exigent un investissement supérieur aux réducteurs de pression, elles peuvent être utilisées avec bénéfice. Mais cependant on ne peut pas se dispenser d'un réducteur de pression comme unité de réserve.

Les utilisations possibles des vannes réductrices de pression dans le réseau sont mentionnées. Les conditions pour leur bon usage sont énumérées.

Référence est faite aux changements admissibles de pression dans la conduite lors des manoeuvres sur les pompes pour les longues conduites ou aux mesures à prendre pour étouffer les coups de bélier.

Des valeur empiriques des domaines d'application sont énumérées pour les volants et les réservoirs à air qui sont les accumulateurs d'énergie les plus utilisés.

Référence est faite à quatre types de schéma d'insertion des pompes dans le réseau, et les conditions d'exploitation qui en résultent sont décrites. Une attention particulière doit être apportée aux pompes de surpression installées dans les propriétés privées, car elles peuvent amener des désordres dans le réseau de distribution.

Si la pression d'alimentation est produite au moyen d'une énergie de pompage, la pression peut être rattachée directement au coût de l'énergie. Le rendement de l'exploitation est augmenté si l'on évite les pressions excessives. Les régions où il y a d'importantes différences d'altitude sont divisées en zones d'altitude graduée, une différence étant faite entre les connections en série et parallèles.

Lorsque la pression d'alimentation augmente, on observe une augmentation de la consommation dont une part est attribuable aux pertes par les fuites.

Finalement, le contrôle de la pression et la conservation de sa preuve par un enregistrement général de toutes les manoeuvres est important pour protéger le service de distribution d'eau contre les plaintes en dédommagement injustifiées.

Long term planning of water supply

by Prof. P. L. Knoppert

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0 Preface

0.1 At the invitation of the Scientific and Technical Council this General Report will emphasise three aspects of long term planning of water supply:

(a) Systems of prognosis; possibilities and problems
(b) Large scale planning of water resources areas and water supply systems
(c) The role of the waterworks engineer in waterworks planning.

These sections will be preceded by a general chapter on long term planning of water supply; the need, the goals, the practical aspects, the topics.

0.2 The General Report has been drawn up using information and data supplied by National Reporteurs from the following countries:

Belgium	Mr. H. Marechal
Finland	Mr. K. J. A. Tammela
Germany	Mr. K. Klotz
Great Britain	Mr. B. Rydz
Ghana	Mr. K. M. Addison
Japan	Mr. H. Aoki
Netherlands	Prof. P. L. Knoppert
Russia	Prof. F. Chevelev
Spain	Mr. M. P. Pennella

The general rapporteur would like to express his sincere thanks to the national rapporteurs. As he also acted as national rapporteur for the Netherlands he would like to express thanks for the important contributions of Mr. J. A. C. Snijders and Mr. P. J. Verkerk and for the many data he received from a number of Dutch water undertakings.

The general rapporteur acknowledges with special thanks the important assistance of his cooperator Mr. H. F. Kaltenbrunner.

1 Long term planning of water supply—general

1.1 Planning defined

We define planning as a process for determining appropriate future action through a sequence of decisions. The word determining is used here in two senses: finding out and assuring. The decisions which constitute the planning process are made on three levels: first, the determination of ends and criteria; second, the identification of a set of alternatives and the selection of the most suitable alternative; and third, the guidance of action toward determined ends. Each of these decisions requires specialist knowledge and the exercise of judgement. The specialist knowledge has to provide the scientific background for collecting, evaluating and processing the information which serves as the basis for responsible judgement.

The framework for planning activities is formed by technical possibilities and legal regulations; the internal structure of the planning activities is determined by the applied methodology.

1.2 The need for long term planning

We define long term planning as the planning activity for a period of 20 to 50 years ahead. Until the beginning of the 'sixties the water industry and governments in most countries rarely applied a long term planning approach of any consequence for the systematic and priority orientated extension of water supply for the population and industry. Except for very few situations the need for long term planning was not obvious then. Water resources were not fully used at that time and the water industry was organised locally and in small units. This situation did not make it too difficult to follow yearly increases in water consumption. The necessary long term prediction of water consumption for the construction of water treatment works and distribution systems was nearly always obtained by simple extrapolation of the historical situation. Because of the high growth rate of water consumption, the low water price and the predictable development pattern of society, this planning approach caused little or no problems. The only consequence was that extensions mostly had to take place earlier than expected. These extensions however could be realised nearly always without problems.

During the 'sixties a change took place in the approach towards planning. There were several reasons for this. First of all it became obvious that a further increase of water consumption would result in a total exploitation of groundwater resources within a few decades, and, in addition, an intensified use of surface water of ever deteriorating quality for water supply purposes. Secondly, the increasing scale and density of the infrastructure for public utilities caused the collision of interests as well as growing public concern. These circumstances necessitated centralised and comprehensive long term planning.

1.3 The goals of long term planning

The ends of long term planning for water supply are the provision of a sufficient quantity of wholesome drinking water for the population, and of water with a sufficiently good quality for industry; both in the most economic way. The methods for achieving these ultimate goals will largely depend on the circumstances in the country, region or town concerned.

These circumstances are characterised by constant factors such as hydrology, topography and geographical situation, and by variable factors such as the economic, technical and social situation. It is obvious that these factors in relation to each other determine to what extent quantity or/and quality problems become the most crucial point in the long term planning of water supply. It should however not go unnoticed that the quality and quantity aspects of water resources are always closely related to each other. In most cases there is a quite limited amount of high quality water resources—either ground-water or unpolluted surface water—which is supplemented by large water resources of lower quality.

In this situation it is only a matter of time before the long term planning of water supply extends beyond the mere activity of planning towards policy making. This can concern water resources management in terms of quality and quantity, water tariffs, various aspects of physical planning etc. This is where the long term planning of water supply becomes a valuable contribution to the efforts of man to maximise social well-being in all human communities.

1.4 Practical aspects of long term planning of water supply

It is not the external or constant factors such as the hydrology, topography and geographical situation of a country, region or town which influence the planning process. These elements form merely the input into a decision model; they determine the kind of technical solution of a certain problem.

The process of planning is, however, strongly influenced by variable, man-related factors such as the technical, economic and social pattern of the environment which has to be planned.

In discussing practical aspects of long term planning of water supply in different countries it seems to be useful to reach back to the classification principle in the first paragraph which referred to the three decision levels in planning: first, the determination of ends and criteria; second, the identification of a set of alternatives and the selection of the most suitable alternative; and third, the guidance of action toward determined ends.

The biggest differences in the planning process on all three of these decision levels occur between countries whose economic, technical and social patterns are entirely different; between industrialised countries and developing countries.

1.4.1 The determination of ends and criteria

Industrialised countries generally face big problems in the final selection of ends and criteria for the long term planning of water supply because of the many circumstances which have to be considered in a modern and complex society. In addition to the technical and economic elements in planning, the infrastructure and legal background influences the planning of water supply more and more. In recent years, this already difficult situation has been further complicated by increasing public participation in planning. This development actually leads to a planning process where the weighing of different issues is required for various steps in planning and where it is possible that decisions already taken have to be scrutinised and reviewed.

It is obvious that the planning process as such, and its procedures, have to be adapted to this situation. As could be expected, this adaptation results in different planning procedures for different countries. These planning procedures are then characteristic for the historical situation, the form of government and its policy and the legal regulations of a particular country.

In spite of all dissimilarities resulting from these circumstances there are some common features in the long term planning of water supply in all industrialised countries:

- —Preliminary planning which considers not only orders of magnitude but also structural and legal aspects and possible interferences with other interests or projects becomes more and more important in relation to the technical side of planning.
- —The ratio of the time and manpower which is spent on planning projects and the time and manpower necessary for their construction is increasing sharply.
- —Planning procedures on the one hand adapt to existing legislation, on the other hand they create the need for new and rather specific legislation.
- —In all planning procedures there are several routine possibilities for feedback between earlier and later stages of planning.

In summarising, it can be said that the role of long term planning of water supply in industrialised countries is to formulate the needs, and to represent and advocate the interests of the water industry in an already existing overall planning activity which comprises all public utilities in a modern society.

The determination of ends and criteria for the long term planning of water supply is less complicated in developing countries. There, priorities are mostly quite obvious and the criteria generally can be clearly identified. The needs of developing countries call in the first instance for a technical and economic solution of the problems of water supply. The number of parties concerned is generally smaller and their interests do not interfere at so many points.

The planning of water supply in developing countries is more fundamental and task-orientated rather than problem-orientated. The long term planning of water supply is quite often one of the first coherent planning concepts in a developing country.

1.4.2 The identification of alternatives and the selection of the most suitable alternative

For industrialised countries the complex situation of the previous chapter continues in the two subsequent steps: the identification of possible alternatives and the selection of the most suitable, or optimum, alternative.

After a careful determination of the ends and criteria for long term planning of water supply, the identification of the possible alternatives is a comparatively simple step. The reason for this is that the approach underlying the identification of alternatives is a qualitative one because this phase is merely a compilation of the possibilities which were not excluded by the applied criteria.

The selection of the most suitable alternative requires an entirely different approach which causes considerably more difficulties and is preceded by the comparison and evaluation of alternatives. This necessitates quantification and analysis in terms of costs and benefits of all possible alternatives. At this stage, the planner faces the problem of quantifying "soft" criteria such as social or environmental considerations. It goes without saying that public participation in long term planning of water supply will focus on this selection of the most suitable alternative. Approval of the preferred alternative by public authorities is required.

In developing countries, the identification of possible alternatives consists mainly of a compilation of the various technically and economically feasible solutions to a water supply problem. In this context, the terms technically and economically feasible have a somewhat more specific meaning than usual. Feasible means in this case that a certain solution is possible and practicable using extensively locally available know-how, technology and materials.

In developing countries, the selection of the most suitable alternative is mostly a planning phase which is difficult to separate from the previous one, the identification of possible alternatives. The reason is that where more explicit and mostly technical planning criteria exist, the planner or the planning authority naturally quite soon develop a certain preference for one or the other alternative. There the problem of quantifying and analysing "soft" criteria in terms of costs and benefits does not arise to the same extent as in industrialised countries.

Long term planning of water supply in developing countries carries a great deal of responsibility for the future social welfare of the country.

1.4.3 Guidance of action toward determined ends

For the long term planning of water supply, the guidance of action toward the determined ends basically consists of three elements:

(a) Policy making determines the planning programme and sets the outer limits for the implementation of a long term plan.

(b) Administration carries out the planning programme taking account of statutory procedures and the legal background.

(c) Planning, or more specifically, the planning authority assists policy makers by continually observing the direction of a given long term planning programme and by suggesting means for redirecting it towards its intended goals. This role of a feedback control mechanism of planning is essential because:

—Administration, consciously or unconsciously, is inclined to modify a planning programme. This is mainly due to the many levels with their different interpretations of facts and responsibilities.

—Unpredictable consequences can arise which may have a significant impact on the long term plan. Such unanticipated events can concern technical, administrative and legal matters.

It is obvious that these three elements in the implementation of a long term plan for water supply are not always as strictly separated as indicated above, but these three areas of concern are to be found in every long term planning process.

It can be stated generally that industrialised countries have a strong tendency to separate these three elements in the implementation of a long term plan and to subdivide even further, certain fields of activity (in particular in administration). In such cases the statutory procedures and laws which regulate the control and administration of a long term plan are mostly of an incredible complexity.

In developing countries the guidance of action toward determined ends for a long term plan for water supply essentially comprises the same three elements of policy making, administration and planning. However, they exist in a far more rudimentary form and technical problems tend to dominate this planning phase.

1.5 Topics related to the preparation and implementation of long term plans for water supply

In the preparation and implementation of long term plans for water supply, two main areas of concern soon take quite definite shape:

1.5.1 Prognosis of water consumption

We defined planning as a process for determining appropriate future action through a sequence of decisions. These decisions have to be based on information which is extracted from properly collected, processed and interpreted data.

By far the most important information for long term planning of water supplies consists of data for the expected water consumption in a certain area for a certain future period. This is why a lot of know-how and manpower is used in the prognosis of future water consumption.

It goes without saying that the methods of prognosis applied are of paramount importance for the relevance and reliability of a long term plan for water supply.

1.5.2 Large scale planning of water resources and water supply systems

Long term planning of water supply concerns mainly planning on a large scale.

In this case planning activities focus in particular on two main aspects:

—The planning of water resources is essential in order to ensure that future water demands can be fulfilled in terms of quantity and quality. From what has been said already it is obvious that the priorities in the planning of water resources are largely determined by constant factors such as hydrology, topography and geographical situation and by variable factors such as the economic, technical and social situation. Procedures for the planning of water resources will be dependent on the legal background and the number and status of parties concerned.

—The planning of water supply systems is essential in order to ensure that water can be delivered in adequate quantity and of sufficient quality from the resources to the consumers. These water supply systems include works and installations for the pumping, storage, treatment and distribution of water. Besides all the other aspects of long term planning, the planning of water supply systems is a highly technical matter where future developments of mechanical and chemical technologies and of materials have to be carefully considered. Other important developments in this field concern, for instance, optimisation methods for distribution networks, rationalisation methods etc.

1.6 The interests and activities of the water industry in long term planning

The water industry in general, and waterworks associations in particular, have an important role in the framework of total planning activities of water supply. This includes all aspects of long term and large scale planning of water resources and water supply systems. The activities of the water industry will be dealt with extensively in the following chapters but it is certainly most appropriate to identify and to give reasons for the interest of the water industry in planning:

—The economic production and distribution of a sufficient quantity of water of good quality necessitates long term planning of production and distribution works. Long term planning includes a responsible approach to the setting-up of the total infrastructure of water supply systems and to water resources planning. This is where the extra dimension of large scale planning is added to the pure long term planning approach.

Both aspects are based on information from reliable water consumption prognoses.

—The water industry is the only group which combines availablility of relevant information and know-how for cooperation in large scale planning with other mainly government authorities.

2 Methods of prognosis for water consumption

2.1 Introduction

Until the beginning of the 'seventies, various forms of trend extrapolation were used generally for the prognosis of future water consumption. This means that the past development of water consumption was used as a basis for the projection of water consumption into the future. Nevertheless there was a number of developments in the course of time. Quite some time ago the total water consumption was divided into two parts, domestic consumption and industrial consumption. This distinction was made in particular where these two consumption categories followed a different trend pattern, as for instance a linear consumption increase for domestic consumption and an exponential growth for industrial consumption.

Incidentally, an S-shaped saturation curve was used for the extrapolation of water consumption (e.g. in Finland, Netherlands). In this method water consumption approaches a certain upper limit which is mostly determined separately. In dealing with the various methods of extrapolation of trend patterns their advantages and disadvantages will be shown. From this can be concluded the kind of research necessary to arrive at improved approaches. One of these improvements is the introduction of statistical methods which come under the term regression analysis. The most important improvement of this method in the analysis of time space series data is the possibility of indicating a trend with a confidence limit. This method can be used also for finding correlations with data other than time.

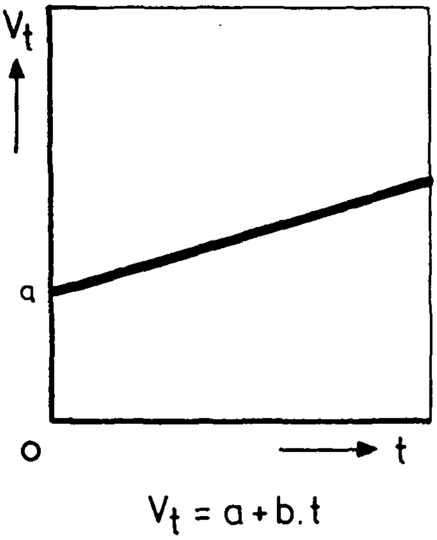
The most recent development is the introduction of the systems analysis approach. In this method, the influence factors for water consumption are identified and analysed and finally integrated into a mathematical model. Such prognoses have a conditional character, their structure is quite obvious and the consequences of assumption variations can be determined.

The possibilities of this method and the problems which arise will be demonstrated in some examples from practice which are given in the Appendices.

2.2 Conventional methods of trend extrapolation

2.2.1 Survey

2.2.1.1 *The linear extrapolation of trends* (Graph 1)



Graph 1

With this model, the absolute amount of growth per unit time is a constant. The extrapolation line can be expressed as:

$$V_t = a + b \cdot t \qquad (1)$$

V_t = water consumption at the time t

t = time

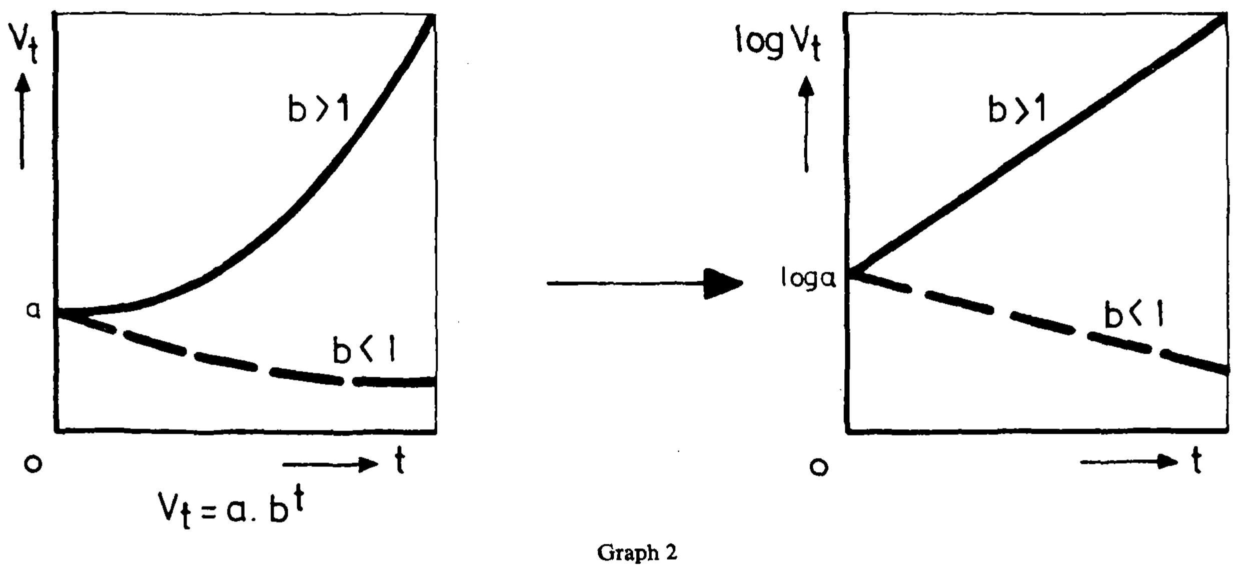
a and b are estimated or calculated constants.

2.2.1.2 *The exponential extrapolation of trends.*

With this model, the relative growth per unit time (e.g. in per cent) is a constant. This approach is quite satisfactory in periods of rapid expansion of water consumption.

The extrapolation line can be expressed in two formulae:

(a) (Graph 2) $V_t = a \cdot b^t$ (2)



Graph 2

Such a formula can be linearised by transformation into:

$$\log V_t = \log a + t.\ \log b \qquad (3)$$

This means that using a logarithmic scale for V_t and a linear scale for t results in a straight line.

The purpose of such a transformation is to make recognition of a certain trend more easy. Such a linearised pattern can be more easily extrapolated than a curved line.

(b) (Graph 3) $V_t = a\ .\ t^b$ (4)

If $b=1$ the formula is linear.
If $b \neq 1$ the formula can be linearised into:

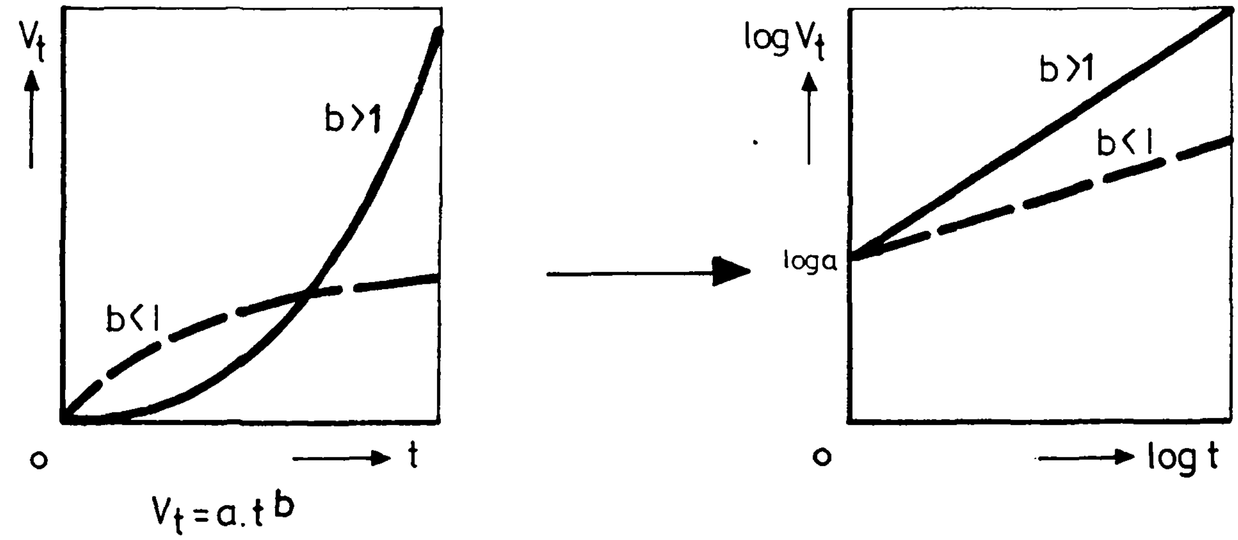
$$\log V_t = \log a + b.\ \log t \qquad (5)$$

In this case V_t as well as t have to be drawn to a logarithmic scale in order to get a straight line.

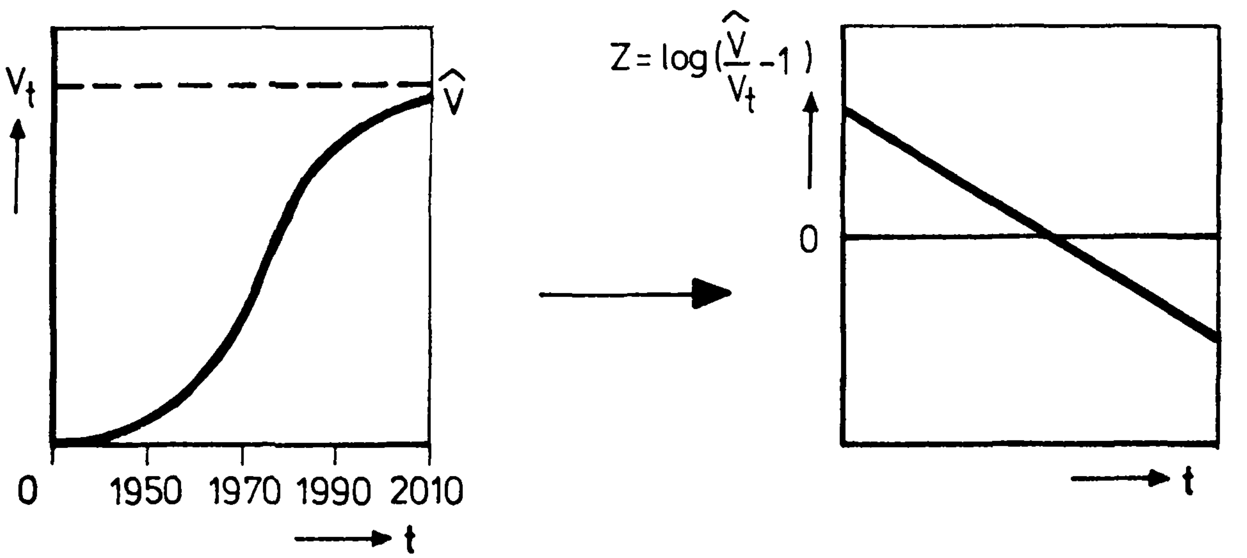
2.2.1.3 *The logarithmic extrapolation of trends*

(Graph 4)

Such a trend pattern is expressed in so-called S-curves or saturation curves. They are characterised by a slow



Graph 3



Example :

$\log \left(\frac{\hat{V}}{V_t} - 1\right) = a - bt$

$z = a - bt$

$a = 0{,}125$
$b = 0{,}0586$
$1975 : t = 0$

Gives :

$t = -45$ (1930) $\longrightarrow$ $z = 2{,}51$
$t = -2$ (1973) $\longrightarrow$ $z = -0{,}01$
$t = 0$ (1975) $\longrightarrow$ $z = -0{,}13$
$t = 35$ (2010) $\longrightarrow$ $z = -2{,}18$

Graph 4

start which is followed by fast (often exponential) growth which slows down when the curve approaches a certain saturation level. These curves were used sometimes in Europe when it became obvious that the enormous growth of the 'sixties could not last for ever. One possible form of expressing such a curve is:

$$V_t = \frac{\hat{V}}{1 + 10^{a-b \cdot t}} \qquad (6)$$

$\hat{V}$ = maximum water consumption in the future (saturation level).

The saturation level is roughly determined and then varied. The most probable curve is chosen from the set of various possible growth curves by taking that one which best fits the recent developments in the consumption pattern.

This method is not a real extrapolation of a trend because the saturation level is determined separately.

A variation is an S-curve with a so-called expanding limit, where the S-curve approaches a slightly inclined asymptote. This method is in fact already an improvement on conventional extrapolation methods because it takes future changes in the trend into account.

In addition the S-curve can be linearised by the following transformations:

$$\frac{V_t}{\hat{V}} = \frac{1}{1 + 10^{a-b \cdot t}} \qquad (7)$$

$$1 + 10^{a-b \cdot t} = \frac{\hat{V}}{V_t} \qquad (8)$$

$$10^{a-b \cdot t} = \frac{\hat{V}}{V_t} - 1 \qquad (9)$$

$$(a - b \,.\, t). \;\; \log 10 = \log\left(\frac{\hat{V}}{V_t} - 1\right) \qquad (10)$$

or:

$$a - b \,.\, t = z. \qquad (11)$$

The value of z can be determined for each V_t. The z-axis as well as the time-axis are on a linear scale. The corresponding V_t-scale can be drawn next to the z-axis. The S-curve is applied also in prognosis methods which use a systems analysis approach, in particular in dealing with certain elements of the prognosis such as the prediction of developments in the use of water consuming equipment. For a lot of products for instance, it is usual that after their introduction on the market they have a relatively slow start which changes into a fast increase of sales when the product becomes more popular. Finally the market becomes saturated and sales decrease; the total number of products approaches saturation level. After saturation of the market, the total number of products in use can nevertheless still grow, for instance because of an increase in the number of dwellings. This leads then to an S-curve with an expanding limit. This kind of curve can hardly be used for calculations. It is therefore preferable to work with a fixed limit and to take into account separately an eventual expansion.

2.2.2 Application in practice

In applying the above mentioned methods of prognosis the first step is to try to identify the trend pattern by means of a graphical presentation of data.

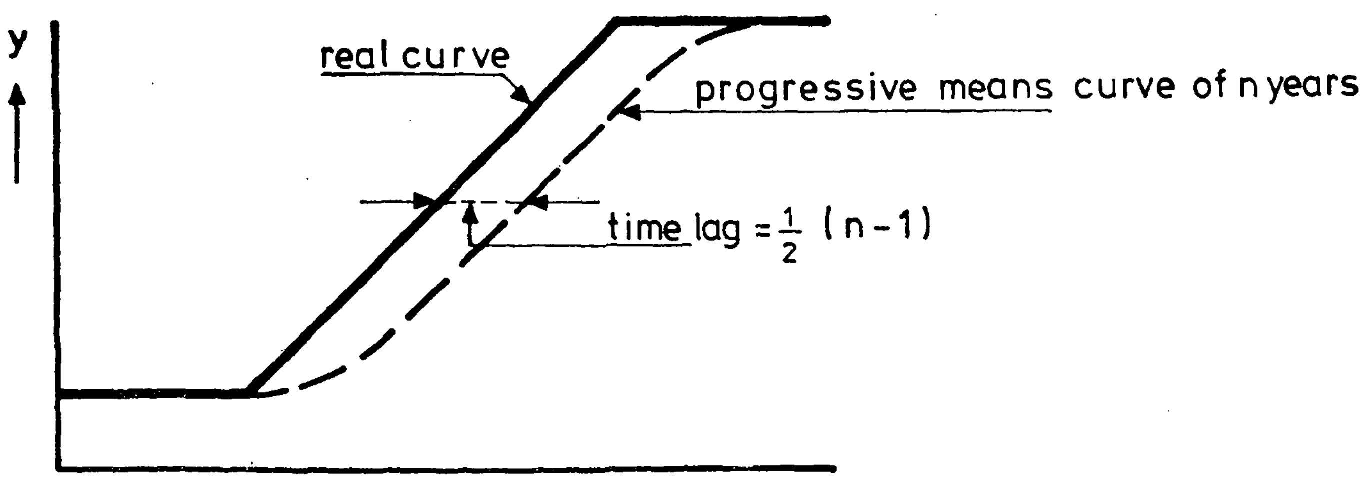
This remains to a large extent subject to judgement; the reality is mostly more complex than a simplified mathematical formulation. The next step after determination of the type of trend is the transformation of data into a linear relationship which can be represented by a straight line. Subsequently, the slope of the straight line is determined. This can be done by the technique of progressive means, a method where accidental deviations are eliminated to a large extent.

Let us assume for instance that we have to determine the progressive means of groups of n data (for example of four yearly water consumption).

This progressive mean lags behind the actual development. This lag is taken as $t_{lag} = 0{,}5\ (n-1)$ time intervals (Graph 5). One of the reasons that the progressive mean is not taken for too many subsequent data is to avoid changes in the trend only becoming visible when it is too late. On the other hand the number of data has to be large in order to provide for a sufficient elimination of incidental deviations. Finally the line which was determined by progressive means and whose slope is now known is shifted parallel by the interval of the time lag. This gives the trend line. In another method, the line of which the slope is now known is shifted parallel in such a way that the same number of actual data are above and below the line.

Now such a line can be extrapolated and the values which are found for the period of prognosis can be transformed back if transformed data were used initially.

The progressive means in this method are determined from n data which all carry the same weight of $1/n$ and data outside of the chosen period do not carry any weight at all. If desired, this transition can be made more gradually. The weights are then arranged in a decreasing sequence in such a way that the most recent data have the greatest weight. The sum of the total weights which can be arranged, for example, in a triangular or exponential pattern always has to be one.



Graph 5—The time lag in the determination of progressive means.

2.2.3 Advantages and disadvantages of conventional extrapolation methods

2.2.3.1 *Advantages.* Conventional methods of extrapolation have the following advantages:

- —They can be carried out in a very short time.
- —The requirements for specific knowledge in the person carrying out these calculations are low.
- —Consequently the costs are low.
- —Eventual changes in the approach or variations of the trend can be considered within a short time.
- —The requirements for the information necessary are quite low.

All these advantages carry quite a weight which is a good explanation for the fact that more advanced but considerably more complicated methods are only applied if it is obvious that conventional methods of extrapolation are bound to fail. They are then only used in situations where a more detailed approach becomes impossible due to the lack of useful information.

2.2.3.2 *Disadvantages.* The disadvantages are dealt with in more detail because they indicate in what direction work has to be done in order to arrive at better methods. A number of disadvantages come from the presuppositions on which conventional methods of trend extrapolation are based. These presuppositions are:

(a) historical data are available
(b) the data are real water demand values
(c) they are relevant for the future
(d) time is used as the variable factor for water consumption.

Historical data are generally available. For the most part, however, there are no water demand values available, only consumption data. The expression "demand" refers to the amount of water that would be consumed if the water were free of charge and available in unlimited quantities. The "consumption" is the actual amount of water consumed. The price of water and the extent to which water is made available is a matter of choice of policy. The extent to which the consumer wishes to fulfill his water demand under certain circumstances remains the consumer's choice. Generally, "demand" is higher than "consumption". It is difficult to say by how much. It is mostly assumed that under moderate circumstances "consumption" follows "demand" on a somewhat lower level. Changes in policy (big changes in water price, modifications in the tariff structure), legal measures (e.g. concerning the environment) and sometimes external influences (e.g. energy prices) can cause discontinuities in the trend of "consumption" which do not necessarily go together with a change in the "demand" trend.

If and to what extent historical data are relevant for the future remains a problem for all methods of prognosis. It is, however, hardly possible to come to a reliable statement about the relevance of consumption data because they are subject to a great number of influences. If aiming at an improvement one should therefore be concerned with analysis and the prediction of the development of the separate influencing factors.

In the extrapolation of trends time is used only as a variable. The influencing factors actually sometimes have a development pattern which is related to time. It should however be kept in mind that water consumption as such is by no means a function of time, but of a lot of influencing factors such as the number of the population, the number of dwellings, consumption habits, economic productivity etc. All of these factors show development in time but they are not themselves a function of time.

Further disadvantages can be mentioned: the fact that a trend is difficult to recognise from a series of consumption data. There are accidental factors, fluctuations in the patterns of the market etc. which can obscure the situation.

Furthermore, the result can be influenced by the length of the series of consumption data, the arbitrarily chosen end of data series etc.

These disadvantages are known. An improvement to these methods can be obtained by the introduction of statistical methods. By using statistical methods, trends can be better identified and it is not only possible to state an average expectation but to indicate the confidence range of a trend with a previously fixed confidence limit. This leads in consequence to a sort of maximum and minimum expectation.

2.3 Regression calculations

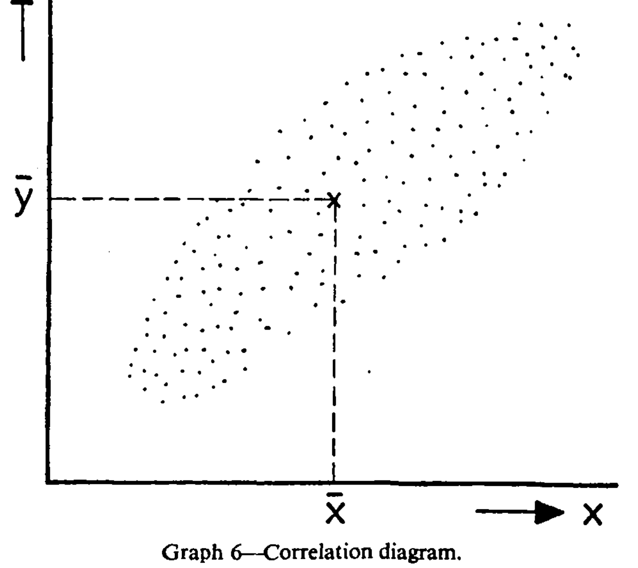
2.3.1 General

The use of statistical methods for setting up prognoses only recently became more commonly applied. By these methods expectations can be calculated far better and it is possible also to determine confidence limits ("the actual value will have a 90% probability between this limit and that limit").

Regression analyses, as these methods are called, can be used for finding out certain relationships between various parameters. This method as such is very important too for mathematical modelling which is demonstrated later.

The following considerations are based on a problem where the variable x is an independent variable of the variable y. The relationship is assumed to be a straight line and thus a linear relationship. In the case of curved-line or non-linear relationships it is sometimes a more or less linear relationship.

Examples of this have been given already. If such transformations are necessary, the calculations are done with the transformed data material, and the results can be transformed back later. The relationship between two variables is determined roughly by drawing out the variables in so-called correlation diagrams. An example of a positive correlation is given in Graph 6: the increase of x goes together with an increase of y (e.g. time and water consumption).



Graph 6—Correlation diagram.

2.3.2 Standard deviations, covariance and coefficients of correlation

The situation of a cluster of points can be indicated roughly by the point with the coordinates $\bar{x}=\sum x/n$ and $\bar{y}=\sum y/n$. This point is called the centre of gravity of the cluster of points. As measures of dispersion there are in the first place the standard deviations S_x and S_y which are expressed as:

$$S_x=\sqrt{\frac{\sum(x-\bar{x})^2}{n-1}} \qquad (12)$$

$$S_y=\sqrt{\frac{\sum(y-\bar{y})^2}{n-1}}. \qquad (13)$$

These two formulae do not express anything about the relationship between x and y.

The intuitive term "relationship" is numerically expressed in the covariance of x and y.

If the single points of the cluster of points are given as (x_1, y_1), $(x_2, y_2) \ldots (x_n, y_n)$ the covariance can be expressed as:

$$\text{cov.}\ (x, y)=\frac{1}{n-1}\sum_{i=1}^{n}(x_i-\bar{x}(y_i-\bar{y}). \qquad (14)$$

The fact that the covariance is a useful measure for the relationship of two variables can be explained as follows. If high values of x (higher than $\bar{x}$) go together with high values of y (higher than $\bar{y}$) each of these corresponding points will contribute positively to the total sum. The same holds good for the combination of low values of x with low values of y. The correlation in this case will result in a quite high positive value which means a positive covariance. An analogous reasoning can be applied to a negative covariance. The covariance does not change if all x_i's (or y_i's) are reduced by the same figure but the covariance is influenced by the units in which x and y are expressed.

This drawback does not exist for the correlation coefficient $r(x, y)$ which is formulated as:

$$r(x, y)=\frac{\text{cov.}\ (x\,.\,y)}{S_x\,.\,S_y}. \qquad (15)$$

The value of the correlation coefficient is always between -1 (negative correlation) and $+1$ (positive correlation).

The extreme values indicate that there is a complete and linear relationship. There is no relationship if all the points are on a horizontal line or if they form a cluster of points without any connection (the value of x does not influence the value of y); in this case the correlation coefficient is zero. A high correlation coefficient on its own gives little information.

In the case of two points, for instance, which are not on a horizontal line, the r is always either $+1$ or -1, independently of whether or not there is an actual relationship. The chance of obtaining an accidentally high correlation coefficient (without an actual relationship) is particularly high with a small number of data. In such a case the correlation coefficient has to be checked for this significance (the "being non-accidental" with a certain probability). The statement that there exists a significant relationship has to satisfy a certain reliability requirement. From this the risk can be determined that a certain statement is wrong. For this purpose in practice tables are used which indicate for a certain number of observations the value of r at which there is a certain probability of making a wrong statement.

Example: At $n=3$ there is a 5% chance of an apparent relationship at a value of $r=0{,}988$.

At $n=10$ there is a 5% chance of an apparent relationship at a value of $r=0{,}550$.

2.3.3 The method of least squares

If it is proved that data have a significant correlation, the next step is the determination of the straight line which best represents the cluster of points. In the least squares method the sum of the squares of the deviations is minimised. This method was developed by the statistician Galton who applied it to the relationship of the heights of fathers and their sons. He demonstrated that in spite of the fact that tall fathers generally have tall sons there is a regression towards the average length. This was how the name regression analysis was introduced.

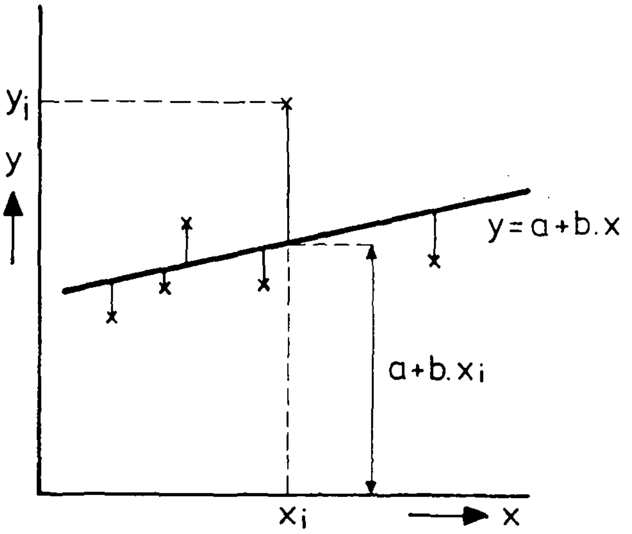
The line with the best fit has the general equation:

$$y=a+b\,.\,x \qquad (16)$$

and the characteristic that:

$$\sum_{i=1}^{n}=[y_i-(a+b\,.\,x_i)]^2 \qquad (17)$$

is a minimum. This is the least squares criterion (Graph 7).



Graph 7—The least squares method.

The expression:

$$y_i-(a+b\,.\,x_i) \qquad (18)$$

is the distance of the point (x_i, y_i) from the cluster of points to the line (Graph 7). The parameters a and b should be chosen in such a way that the expression (17) becomes a minimum. The final values are a^* and b^*. The result is found by means of differentiation.

$$b^*=\frac{\sum(x_i-\bar{x})(y_i-\bar{y})}{\sum(x_i-\bar{x})^2}=\frac{\sum x_i\,.\,y_i-\frac{1}{n}\,.\,(\sum x_i)(\sum y_i)}{\sum x_i^2-\frac{1}{n}\,.\,(\sum x_i)^2} \qquad (19)$$

$$a^*=\bar{y}-b^*\,.\,\bar{x}. \qquad (20)$$

The values a^* and b^* are called the regression coefficients. The line $y=a^*+b^*\,.\,x$ is called the regression line of y on x. The relationship between b^* and r is found from (15) and (19) and is expressed as:

$$r(x, y)=\frac{b^*\,.\,S_x}{S_y}. \qquad (21)$$

The differences $y_i=y^*_i$ are called residuals because they are the vertical distances between the points of the cluster and the obtained line. If the sum of the squares of these residuals is divided by $n-2$ the residual variance is obtained. It is considerably smaller than the variance (the square of the standard deviation referring to $\bar{y}$). The square root of this residual variance is the residual standard deviation:

$$S_{y\ es.}=S_y\,.\sqrt{(1-r^2)}=\sqrt{\sum(y_i-\bar{y})^2\,.\,\frac{(1-r^2)}{n-2}} \qquad (22)$$

The exactness of the regression line obtained is expressed in the value of this residual standard deviation.

A confidence interval can be defined symmetrically on both sides of the regression line. The total width of this interval is

$$2 \cdot t \cdot S_{y\,re} \tag{23}$$

and half of it is on each side of the line.

The values of t are given in tables on the basis of the one side exceeding probability and the so-called degrees of freedom φ. For this $\varphi = n-2$.

For instance for a one side exceeding probability of 5% (the confidence interval of 90%) we get for

$\varphi = 1$ (3 observations) $t = 6{,}31$

$\varphi = 8$ (10 observations) $t = 1{,}86$.

For a certain defined level of confidence, the width of the confidence limit becomes smaller as the number of observations increases.

2.3.4 Curved regression

During analysis for the relationship of two data series it often becomes obvious from the correlation diagram that a curved line gives a better fit than a straight line. In such a case the above mentioned cannot be applied. In some cases however it will be possible to linearise the relationship by the transformation of one or both variables as demonstrated in section 2.2. This means that the transformed data are used for a calculation. In the case of more complicated functions however it will not be possible mostly to apply simple transformations.

Sometimes it is possible to define the relationship as a linear function with various independent variables.

In such a way the parabolic function:

$$y = a \cdot x^2 + b \cdot x + c \tag{24}$$

can be defined as a linear function of two parameters:

$$x' = x^2 \tag{25}$$

and

$$x'' = x \tag{26}$$

This results in:

$$y = a \cdot x' + b \cdot x'' + c \tag{27}$$

A regression with more than one independent variable is called a multiple regression. In principle the determination of a multiple regression function is done in the same way as for a single regression function.

2.3.5 Some warnings

In setting up prognoses, identified relationships are extrapolated. These relationships were however determined for the time during which observations were made. Extrapolations can lead to wrong results. Statistical relationships as such do not give a decisive answer about an actually existing relationship.

It is for instance very possible that in reality there is a curved-line (or non-linear) relationship between two parameters which is not recognised because the data available up to now are fitted quite well by a straight line (for example the part of an S-curve around the point of inflexion).

2.3.6 Conclusions

If regression analysis is applied to data which are used for conventional extrapolation, the advantages mentioned in Section 2.2.3 remain for the most part. The use of a simple computer is however almost a necessity. Appropriate computer programs are generally available from the manufacturers. All of this certainly increases costs, but it can still be considered as an inexpensive and fast method. Part of the disadvantages mentioned in 2.2.3 are avoided by using statistical methods. However, the disadvantages which are inherent in the presuppositions remain. The detection of a trend becomes less of a problem. The correlation coefficients can be compared. If a simple transformation does not give good results it is still possible to go over to multiple regression. Furthermore it becomes possible to choose a certain confidence interval and to indicate the limits within which the trend line will be situated.

However the disadvantage of prognosis on the basis of the extrapolation principle still remains.

2.4 The systems analysis approach

2.4.1 General

There is at present a general tendency in different countries to apply systems analysis for prognoses. Systems analysis is a practice-orientated scientific approach in which a particular problem is represented by a mathematical model which is formulated on the basis of empirical knowledge. An example is the setting-up of prognoses for middle and long term water consumption which consider as far as possible the developments of population, urban growth, the legal background and the relevant technologies.

The complex reality is reduced to its simplified form, the mathematical model. For this, the reality is observed, the important influencing factors are identified and formulated as accurately as possible in mathematical terms. Assumptions are clearly stated and alternative developments are compared as to their possible consequences (Graph 8). This approach touches task-oriented prognosis. If the results of a prognosis which are determined on the basis of an "unchanged" or better "passive" policy are not desirable it is possible to devise measures for obtaining another development. The consequences of such a measure can be simulated on a model.

From what has been said so far it is obvious that it is scarcely possible to give a survey of the different types of models. Besides that, the author knows only of attempts in this direction being made in the Dutch water industry. This refers to investigations in the towns of Amsterdam and Rotterdam. They are dealt with in more detail in the Appendices.

2.4.2 Some additional characteristics of the systems analysis approach

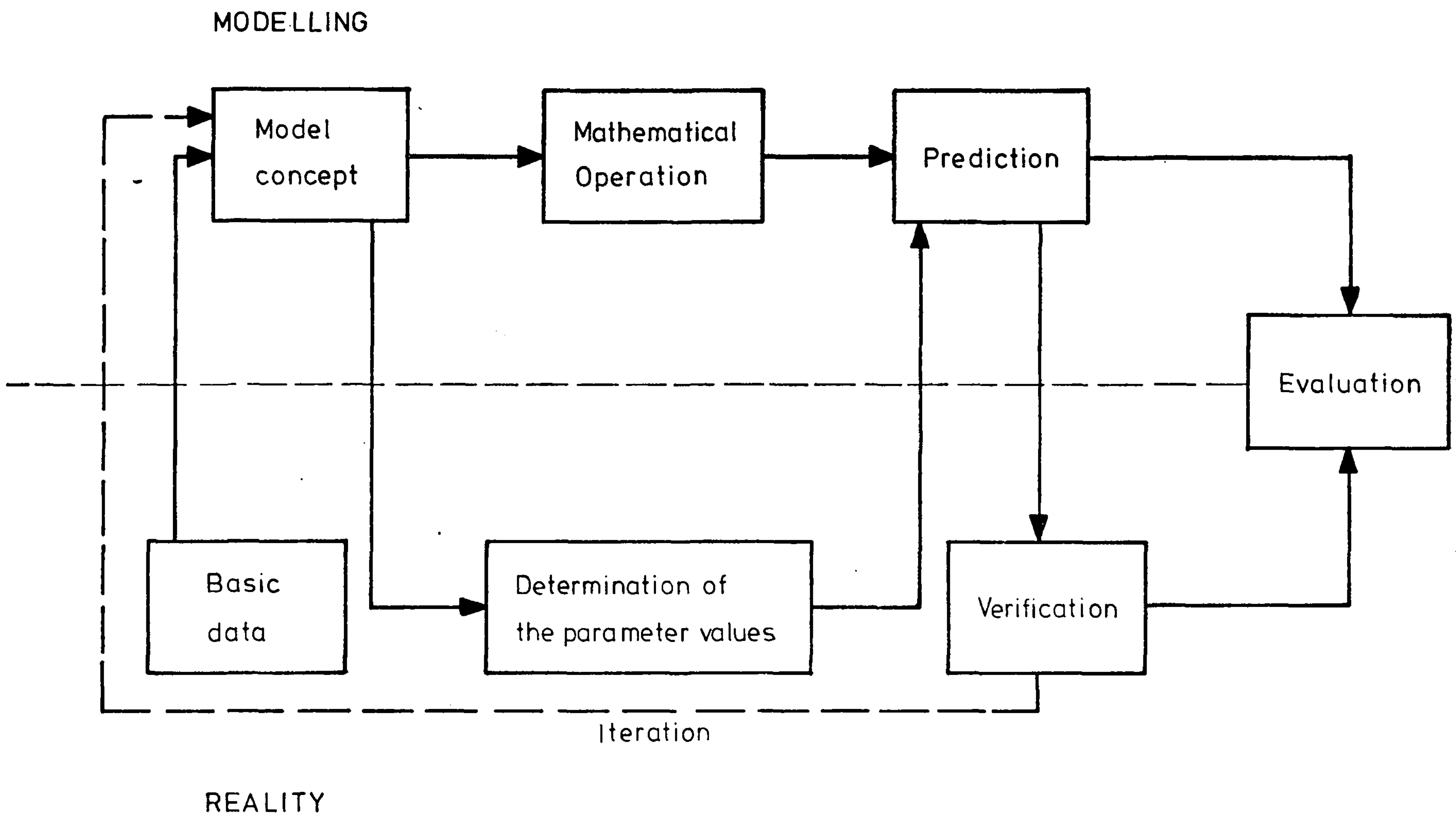
In addition to the characteristics which were mentioned in the previous paragraph, the following can be said:

(a) Systems analysis is used only for complex problems.

(b) The majority of problems and subjects to which systems analysis is applied are characterised by a high number of unknowns. These unknowns are identified, made explicit and finally used in modelling.

(c) In systems analysis, scientific methods are applied; it is a very flexible method and the results can be quantified.

(d) The judgement, intuition and knowledge of the expert are stimulated by this approach as well as directed in a systematic and efficient way. It enables people who have to take decisions to make their judgement on a much sounder basis.



Graph 8—Reality and modelling in the systems analysis approach.

2.4.3 Some limitations

(a) The analysis is necessarily incomplete. It is impossible to include all relevant factors in the model because the reality is much too complex.

(b) The development of some influencing factors can only be estimated.

(c) It is impossible to predict the future. Unknowns in future developments are—to a lesser extent than in conventional methods—a serious limitation in the application of systems analysis.

(d) The applicability of this method depends on the quality of the estimates. It is essential for modelling that the multitude, complexity and ramifications of all the factors and interrelationships in reality are replaced by a certain simplification. This necessitates a careful and considerate approach whose philosophy should be formulated as clearly as possible.

Systems analysis is more an art than a science. It is a method which is applied for complex large-scale problems and where specialists in cooperation with decision-makers apply scientific methods for finding the most suitable tools for solving certain problems.

2.4.4 General set-up of systems analysis projects (Graph 8)

It is quite obvious that systems analysis and modelling can be applied in many more fields other than for prognoses. The most widely known models are economic models, initially based on the work of Prof. J. Tinbergen. Other possible modelling applications are for instance investment models and pipe network optimization models. We will, however, restrict ourselves to modelling for prognoses.

(a) *Identification and formulation of the problem*
This begins with the reasons for the prognosis. The questions which should be answered by the application of a model are formulated. It must be considered at this stage how elaborate the model should be. Subsequently one tries to establish relationships between the important variables. In the case of little empirical data these relationships will be quite hypothetical but certainly they will be helpful in setting up the investigation and in visualising the logical structure of the analysis.

(b) *The investigation*
Basic data for the analysis are collected during this phase. Often this collection of information is the most time-consuming part of the whole investigation. It will be by definition impossible to get all data and facts complete. This is why the correct synthesis of collected data and theories formed becomes an absolutely decisive step. The information collected is the basis for identifying the relationships between the different variables and parameters. Subsequently the most suitable mathematical formulations are chosen. This phase can be called the descriptive phase.

The possible difficulties which can arise in this phase can be shown in the example of the investigation of the industrial and public water consumption in the town of Rotterdam. At the beginning of the project the consumption figures were only available in tariff groups. It took almost a full year of work before consumption figures were available in a useful classification and before the different uses of water were known.

(c) *Estimation of the parameters*
The parameters of the model are estimated by statistical methods, or if this is not possible, in some other way. At the beginning of an investigation it is sometimes impossible to collect enough data for a certain time interval. In such cases it becomes necessary to work with single data and the future development pattern has to be deduced from, for instance, discussions with experts; the application of statistical methods, however, is not possible in such cases during the first years.

(d) *Solving the model*
In the case of a simple model, the calculation can be done by hand or with a desk-calculator. If the model becomes somewhat more difficult the use of a computer is necessary.

(e) *Testing the model*
If possible the results of calculations are compared with reality for one or several cases where the results are already known. This mostly means to reverse a prognosis. If this is not possible it is necessary to wait until enough observations from reality are available. In addition to this the model has to be regularly modified if developments take place which were initially either not considered or were considered in some other way. Such checks can eventually lead to a change of the model and an iterative method of problem solution.

The sensitivity analysis is a frequently neglected part of the test phase (see 2.4.5.).

(f) *The introduction of a model*
During this phase the model is made ready for operation. This means in this case that the model is introduced as a support model. After its introduction, the model has to be reviewed at regular intervals. It can be expected that the model will have to be improved regularly during the first years e.g. because more detailed information becomes available.

2.4.5 Sensitivity analysis

This is one of the most neglected parts of modelling. For the sensitivity analysis, controlled changes are made in the parameters and input data and their effect on the results of the model is observed. These results will be more sensitive for changes in some parameters and input data than for others. Sometimes this can be predicted, in other cases the result of a sensitivity analysis can be rather unexpected.

In such a way this analysis indicates how precisely the parameters need to be determined. If in certain situations one parameter can be determined only approximately and has a big influence on the final results, the model becomes quite useless. In this case the model is mostly too simple and has to be improved. If a particular model is chosen the sensitivity analysis indicates the direction in which further work has to be done for the highest efficiency.

2.5 The practice of modelling

2.5.1 General

Total drinking water consumption is usually divided into several categories which are then considered separately. The two most important categories are domestic consumption and industrial consumption. Leakage losses, individual consumption, public consumption and industrial consumption smaller than 1000 m³ per year are in many cases added to domestic consumption.

The pattern of domestic consumption is quite a regular one. This makes a long term prognosis possible. For the majority of waterworks domestic consumption is the largest consumption category. Exceptions from this are only found in heavily industrialised areas.

The prognosis of industrial consumption is very difficult. These difficulties arise from the special characteristics of this consumption category which are given in 2.5.2.2. Practical experience has shown that due to the many unknowns involved it is generally not possible to make good prognoses for a period of longer than 10 years. An additional complication for prognoses of industrial consumption is the prediction of consumption for new industrial areas. If the type of future industry is known it is often possible to work with data from similar industries. If the type of industry is not known yet the total consumption can be estimated from the total area available, but such a rough estimate is mostly very unreliable.

In some regions consumption by agriculture and/or recreation plays an important role. They are sometimes considered as separate categories.

2.5.2 Characteristics of consumption

The characteristics of domestic consumption are of general validity. The characteristics of industrial consumption apply in particular to regions with big industrial consumers and where they have an important share in the total consumption of drinking water.

2.5.2.1 *Characteristics of domestic consumption*

(a) Domestic consumption is determined by a group of consumers which is large in number and where the water consumption of one individual consumer has no influence on total consumption.

(b) It can be subdivided only in a quite low number of different uses.

(c) On a yearly basis and in comparison with industrial consumption, domestic consumption is of a quite stable pattern.

2.5.2.2 *Characteristics of industrial consumption*

(a) The smaller number of consumers in this category enhances differences in consumption and water use.

(b) Often only a limited number of consumers is responsible for the majority of industrial consumption. This means that the basic data for prognoses for this group of water consumers have to consist of information about the particular consumers.

(c) These important water consumers are generally large and strong industries which are often concentrated in certain regions.

(d) In principle these industries can replace drinking water by substitutes such as air, surface or ground water and/or they can switch to partial or total reuse of process water.

(e) Changes in the production pattern of these individual industries can be a considerable influence on the total consumption of drinking water.

(f) The policy of the central or regional government authorities concerning industrialisation and environment can have important consequences for the consumption of drinking water.

2.5.3 The application of systems analysis

2.5.3.1 *General.*

The way systems analysis is applied in setting up prognoses depends strongly on local circumstances. In the following, a generally applicable model is presented as an example. It was first used in the town of Amsterdam and is known there as the determinant model.

The total water consumption in a certain reference year is split into influencing factors x_i.

Furthermore it is stated which fraction of the total consumption corresponds to a certain influencing factor. This is done with a weighting factor b_i.

The total consumption can then be expressed as:

$$W = \sum_{i=1}^{n} b_i \cdot x_i. \qquad (28)$$

If the water consumption is analysed again after a time t the influencing factors x_i which determine water consumption will have undergone a change Δx_i. This results in a change of the water consumption ΔW.

This can be formulated as:

$$\Delta W = \sum_{i=1}^{n} b_i \cdot (\Delta x_i) \qquad (29)$$

ΔW = change of the water consumption in comparison to the reference year.

Δx_i = change of the influencing factor.

Such a model makes a weighted difference between the various factors which contribute. By varying these factors their relative influences can be determined. This applies also to the weighting (sensitivity analysis). The model can be calibrated for today's circumstances and corrected with the past and subsequently used for a systematic determination of future water consumption by analysing and estimating the influencing factors, also by means of statistical methods. Such estimates have to be based as far as possible on future internal and external policy. They should consider also expected or planned development.

For the calculation, the indices d_i are used for labelling the influencing factors in relation to a certain reference year. The reference year is given the index 100. The water consumption in the year t can thus be expressed as:

$$W_t = W_r \cdot \sum_{i=1}^{n} b_i \frac{(d_i)_t}{100} \qquad (30)$$

W_t = water consumption in the year t after the reference year.

W_r = water consumption in the reference year

b_i = weighting factor for the influence factor of consumption x_i

$(d_i)_t$ = index for the factor x_i for the year t after the reference year.

From the actual water consumption, the theoretical water demand is calculated in consideration of extremes of price elasticity and tariff development. The mathematical formulation of the model is quite simple. The choice of the influencing factors considered is however very important. Below and in Appendix 1 they are dealt with in more detail.

2.5.3.2 *Domestic consumption.*

The most essential step in modelling is as already mentioned the analysis of reality which has to result in the determination of influencing factors. The factors which were used in a study for the town of Rotterdam are given below as an example. The different uses of water are used as a classification system.

The following influencing factors were identified:

(a) Factors related to the person:

—water toilet

—personal hygiene:—tap

—shower

—bath

These factors can be translated into consumption per dwelling by means of the average occupation figure of the dwellings (another influencing factor).

(b) Factors related to the dwelling

—laundry: —washing by hand

—washing machine

—dish washing: —by hand

—dishwasher

—miscellaneous: —car washing

—garden watering

—various other uses

In addition to these factors there are:

—the number of dwellings (new buildings—demolition—renovation)

—the population (occupation per dwelling)

In the case of Rotterdam price elasticity is hardly of importance because the majority of the population's consumption is not metered, which means that due to the flat rate there is no relationship between water consumption and the actual water tariff. This study is summarised in a much simplified form in Appendix 2.

Lack of information was once more a big problem. An accurate and detailed study can eventually solve this problem to a certain extent. The information which becomes available is not only relevant for the region in which the study is carried out because domestic consumption is not as specific for a certain region as is industrial consumption.

The fact that big differences still exist in domestic consumption data of different waterworks is most probably due rather to differences in definition, the chosen influencing factors and the corresponding weights than to real differences in the behaviour of people.

2.5.3.3 *Prognosis of industrial consumption.* The setting-up of the prognosis should be done with special care in regions with heavy industrialisation where a large percentage of the water is consumed by this group.

Again, Rotterdam can be taken as an example where $\frac{1}{3}$ of drinking water consumption goes to the population and $\frac{2}{3}$ are used by industry.

The earlier mentioned characteristics (see 2.5.2.2.) apply in particular to industrial areas. In such a case the study has to consider for instance:

(a) *The expected industrial production*

—The development of industrial production in the past and in the future (the word production is used in its widest sense; occupied hospital beds are also a form of production)

—The development of production capacity in the past and in the future.

—Expected changes in the present production pattern.

(b) *The quantity of drinking water*

—The quantity of drinking water to be consumed in the first years.

—The distribution of drinking water consumption over the consumption categories: boiler feed water (energy saving), process water (environment laws), cooling water (price of drinking water, legal measures), sanitary consumption (less production dependent), etc.

—The introduction of water saving or water using technologies as well as the motivation for this introduction.

—The effects of

—variations in the use of the actual capacity of installations.

—changes in the production pattern

—the legal background for conservation of the environment and for water consumption.

—The use of surface water and groundwater as well as the purpose for which it is used.

—The possibility of using another medium in the place of drinking water.

—The application of water re-use.

The practical case of a prognosis for industrial consumption is demonstrated in Appendix 3.

2.6 What can be expected countrywide and regionally from this methodology?

The big problem is mostly not the model but the availability of useful and sufficiently detailed data on the development of the influencing factors.

2.6.1 Domestic consumption

As already mentioned it can be expected that this group has countrywide as well as regionally a quite stable consumption pattern with a limited number of influencing factors. Carefully planned studies can yield a lot of useful information.

A prognosis term of 30 years is feasible if the increase of the unknowns of such a prognosis in the future is considered. Uniformity of terms and definitions is necessary for the analysis of differences in the results between various towns and regions. Furthermore it seems to be quite a problem to determine exactly the total amount of domestic consumption and the corresponding weights for the influencing factors.

It can, however, be concluded that a modelling approach for this consumption category is countrywide as well as regionally a very promising solution. There remains still a lot of research to be done for the improvement and refinement of models.

If desired, small industrial consumption, leakage losses etc. could be added to this category without substantially changing the above conclusions.

2.6.2 Industrial consumption

Meanwhile it is quite obvious that good data are much more difficult to obtain for this consumption category. Besides that there is a much larger number of influencing factors. A detailed study, however, is still feasible for local prognoses.

It remains a problem that such prognoses can be made only on a middle term basis and that the necessary data for regional economic development and expectations are rarely available.

If long term expectations have to be determined there is still the so-called Delphi-Method which is described in the following chapter. This method is probably more suitable for studies in larger regions (countrywide prognoses).

The problems for countrywide prognoses are entirely different. Economic data and expectations for the whole country are mostly available but the reliability of prognoses decreases with an increasing prognosis interval. The data are mostly given per industry or production branch. Industrial water consumption has to be considered and studied in the same categories. This distribution over the categories is more difficult for the whole country than for a certain region. The best solution is that, as much as possible, waterworks use the same industrial consumption categories for the whole country. This procedure can be completed with interviews in the most important industries and eventually be controlled to a certain extent with more detailed models of certain regions. It can be concluded generally that problems which are difficult to treat for the whole country are more easily solved on a regional scale and vice versa. Models for industrial consumption will include, in particular for regional studies, many more variables than for domestic consumption. Useful results can be expected from this method on a countrywide as well as on a regional basis, especially if the contacts with industries are intensified. There will be, however, necessarily a great deal of prolonged research.

2.7 The Delphi-Method for long-term prognoses of industrial consumption

It has been mentioned already that the detailed approach to the prognosis of industrial water consumption generally does not give satisfactory results for periods of longer than, for instance, about 10 years. One of the possibilities of arriving at a long-term prognosis is the so-called Delphi-Method.

The Delphi-Method is in fact a form of controlled brainstorming and thus a qualitative method. It was originally developed by the Rand Corporation (long-range forecasting study, Sept. 1964).

It is used generally only where few actual data for prognoses are available.

The Delphi-Method can be described as a number of consecutive brainstorming rounds where a group of qualified and selected experts are approached in a questionnaire and express their opinion concerning expected future developments and the time of their realisation.

The test persons are approached separately in written form (there is no contact between them) and the results of the information received are fed back—if possible according to a certain model. This procedure avoids a number of disadvantageous effects in brainstorming such as the impact of the spoken word in a discussion and the tendency in a discussion group to follow the most powerful arguments, or to accept the majority point of view.

This last factor, however, cannot be avoided completely because in the second and subsequent rounds the participants are confronted with the opinion of the majority. Another chance for this effect lies in the fact that people with strongly differing views are asked to explain their opinions.

This method aims at a consensus of expert opinion about future developments and special emphasis is given to the time when these developments are expected to take place. The Delphi-Method is a confrontation of opinions.

During this process the participants are put under impersonal pressure to review their opinion. The result of this method makes sense if a certain conformity is needed; it has the characteristic of well-based speculation.

The problem in applying this method to prognoses of drinking water consumption is that experts of the industries concerned have to participate in the study. This can be quite a problem in local studies because of the lack of experts. The possibilities for countrywide studies are considerably better. There are actually no cases known to the author where this method has been applied purposefully for prognoses of drinking water consumption.

2.8 Summary and conclusions

(a) There is obvious dissatisfaction with today's methods of prognosis. There is a strong tendency from "most-probable-line methods" to the application of dynamic models which give the possibility of identifying changes and different developments immediately and to calculate alternatives within a short time.

(b) The biggest problem in dynamic modelling is the identification of the influencing factors and the collection of relevant information about these factors. The time necessary to collect this information is often underestimated. It must be avoided to change a model at an advanced stage because certain information was not available in time.

(c) The person in charge of research and studies has to prevent modelling becoming a kind of a hobby for the persons concerned.

(d) Special emphasis in modelling has to be given to sensitivity analysis; on the one hand to avoid a study becoming too orientated towards factors and parameters which could be estimated roughly and on the other hand to check if a certain model is detailed enough.

(e) Modelling forces people to create for themselves a good picture of the reality and to define the variables and parameters properly. In this way econometrics for instance has caused a spectacular breakthrough in the economic sciences.

(f) The possibilities for modelling are probably better for domestic consumption than for industrial consumption because the increase of the number of factors in the latter case certainly increases the uncertainty.

(g) Prognoses for future drinking water consumption are up to now mostly not "task-oriented" and are therefore not influenced by the philosophy that future water consumption should be limited in one way or another. Modelling however opens up possibilities in this direction.

(h) Long term prognoses will always be a difficult problem. They are, however, absolutely indispensible because the increasing uncertainties in the modern society necessitate a sounder basis for general policy. Carefully compiled long-term prognoses are an indispensable part of providing such a basis.

3 Large scale planning of water resources and water supply systems

3.1 General

The survey of contributions from the various countries shows general agreement about what should be done in order to carry out large scale planning of water resources and water supply systems.

The measures which are considered to be essential and necessary for efficient and task-orientated large scale planning for water supply can be subdivided into three groups:

(a) Collection of data and information is essential in order to provide for a good decision basis. Some of the points mentioned were:
- —analysis of the present water consumption and prognosis of future water consumption
- —extensive hydrological and in particular hydrogeological surveys
- —extensive surveys of the present water pollution situation, in particular industrial water pollution
- —the expected industrial pollution in terms of amounts, kinds and location of the pollution
- —the present situation and the expected future situation of physical planning and its priorities at national, regional and local levels
- —the present and expected future development of technologies relevant to water supply
- —the present and expected future situation concerning procedures and laws relevant to planning of water supply.

(b) The establishment of regulations is considered essential for the formulation of the requirements of the water industry participating in a comprehensive large scale planning process. Some of the points mentioned were:
- —drinking water quality standards
- —surface water quality criteria
- —protection areas for high quality water resources
- —regulations against the use of certain chemical substances in certain areas, zones or river catchments
- —regulations about the use of water resources in terms of quantity.

(c) Reconsidering laws and regulations.

Existing administrative or statutory procedures and laws have to be changed or supplemented in such a way that regulations can be enforced efficiently and that a given large scale planning process remains practicable in spite of its complexity. If necessary, new procedures and/or laws have to be introduced.

3.2 Preparing a basis for discussion

Discussing large scale planning of water resources and water supply systems means discussing a topic whose aspects vary strongly with the accompanying circumstances and whose complexity makes a straightforward approach impossible. This leads either to extended discussions and a symposium on its own or to simplifications which are not acceptable.

Therefore, in preparing the basis for a discussion on such a topic, it is useful to choose a representative example as a kind of reference point for comparisons and as a background against which the different possible solutions to a problem can be made visible.

In the case of large scale planning of water resources and water supply systems, the planning of the Netherlands is chosen as the example. The reasons for this are: firstly, the Netherlands are a typical example of a densely populated and industrialised country with plenty of environmental problems; secondly, the author's participation in the water resources and water supply planning of the Netherlands makes a comprehensive presentation of the Dutch situation possible.

Before dealing with this particular example, some points which are common to large scale planning of water resources and water supply systems in all industrialised western countries should be mentioned briefly.

The participation of the public in planning and the large number of interested parties tends to result in rather complicated and long-winded procedures. There are certain dangers inherent in this.
- —A plan could be already out of date at the moment when it is finally ready for realisation.
- —Changes of the ends and criteria of a plan necessitate corrections and reviews of the chosen solutions. This reconsideration will possibly not take place because of the expected formal difficulties.

Depending on the legal background, statutory procedures and administration of planning, more specific aspects can come to the fore.
- —Planning procedures which aim at the maximisation of social and human welfare of a country can be abused by certain groups for carrying through their own interests. This can delay or even render impossible the realisation of a project.
- —The many bureaucratic levels which are involved in the realisation of a plan can alter a planning programme consciously or unconsciously.

3.3 Large scale planning of water resources and water supply systems in the Netherlands

3.3.1 Introduction

The Netherlands are generally known as a "water-country". Water supply for the population and industry is nevertheless a big problem. On the one hand there is a shortage of ground- and surface water of a sufficiently good quality, on the other hand the integration of water supply projects for the storage, transport and treatment of water into the national planning concept becomes increasingly difficult.

Long term planning is necessary in order to ensure the construction of the necessary water supply projects for the future. For this purpose a Structural Outline Plan for water supply was compiled in 1972 in the Netherlands for a planning period of about 30 years. The selection of separate water supply projects to be built will be based on so called Ten Year Plans.

Such a Ten Year Plan is in fact a worked-out action programme for the next ten years which is based on the planning concept as given in the Structural Outline Plan. Both the Structural Outline Plan for Water Supply and the Ten Year Plan will be revised periodically once every 5, and once every 2 years respectively. The preparation and adoption of the different plans are subject to procedures which ensure the participation of all interested parties.

Planning by means of the Structural Outline Plan and the Ten Year Plan will be explained in the next paragraphs. In order to facilitate understanding, these paragraphs are preceded by a short survey of the hydrology of the Netherlands.

3.3.2 The hydrology of the Netherlands

The north and west boundaries of the Netherlands are formed by the shoreline of the North Sea; the east and south borders with Germany and Belgium are crossed by several rivers and small streams. The most important of these are the Rivers Rhine and Meuse which form large river deltas. The surface of the Netherlands is about 37 000 km^2. A fifth of its land surface is below the average sea level and about half of the country is protected by dikes against floods.

The upper few hundred metres of the soil generally consist of sandy sediments with layers of clay and sometimes peat. The more elevated southern and eastern parts of the Netherlands, as well as the dune-belt along the sea coast, are quite suitable for the recovery of fresh groundwater. The land in the low-sited polders between the dune-belt and the higher situated inland sandy soil is entirely covered by a clay layer and drains the water from underground. Groundwater raised from the sand formations under this clay layer is mostly brackish or salt water and therefore unsuitable for water supply purposes.

The groundwater table in the Netherlands is generally only some decimetres below the ground surface. There are exceptions like the hilly areas in the centre of the country where the ground-water table is some ten metres below the ground surface. These areas as well as the dune-areas are very well suited for artificial groundwater recharge.

The following table gives the water balance of the Netherlands for an average year.

River Rhine	69×10^9 m^3 per year
River Meuse	8×10^9 m^3 per year
Smaller streams	3×10^9 m^3 per year
Precipitation minus evaporation	10×10^9 m^3 per year
Total available fresh water	90×10^9 m^3 per year

It should be considered that the availability of water in dry years is considerably lower and that the hydrological regimes of the rivers result in large flow differences during the year. Therefore the use of surface water for the preparation of drinking water necessitates storage both from the point of view of quantity (River Meuse) as well as from the point of view of quality (River Rhine).

3.3.3 The Structural Outline Plan for Drinking and Industrial Water Supply

At present such Structural Outline Plans are worked out for public utilities and communication systems in the Netherlands. There are already, and further will be prepared, Structural Outline Plans for drinking and industrial water supply, electricity supply, water management, airports, sea ports, refuse management etc. The total of these Structural Outline Plans gives a long-term projection of the development of the infrastructure of the Netherlands and they are the basis for Government policy on physical planning. The Structural Outline Plans are compiled by the different ministries but the overall coordination is done by the Ministry of Housing and Physical Planning. The Structural Outline Plan for Drinking and Industrial Water Supply, for example, is compiled by the Ministry of Public Health and Environmental Hygiene and the Government Institute for Drinking Water Supply (RID), in cooperation with the Dutch Waterworks Association (VEWIN).

Nearly 100% of the population of the Netherlands is connected to public water supplies. The main problems in water supply are to be found in the execution of projects for the storage, treatment and transport of water. The importance of the Structural Outline Plan for drinking and industrial water supply is that the defining of areas for projects of the Structural Outline Plan forms a framework in which the water industry can select the right projects and execute them in a responsible way.

The Structural Outline consists mainly of:

- —a prognosis of future water demands up to the year 2000
- —a survey of water resources
- —Government policy regarding public water supply
- —a survey of possible future water supply projects
- —a survey of the procedures leading to the preparation and adoption of plans on the various levels.

The estimated water consumption of population and industry (including self-production by industry) for the year 2000 is about 4000 million m^3 per year for an expected total population of 15,4 million.

For comparison it should be mentioned that the total water consumption in 1974 was about 1700 million m^3 per year, of which about 100 million m^3 were supplied by public water suppliers. These figures do not include the use of surface water for cooling water. At present the total population of the Netherlands is 13,4 million.

Partly owing to the sudden economic recession and partly to the charges on industrial effluents which were introduced some years ago, there is a tendency towards a reduced rate of industrial water consumption.

The prognosis of water consumption extends over a period of thirty years and is of a rather rough character. This is not a substantial obstacle to the purpose for which the Structural Outline Plan is compiled.

On this level of planning too-high estimates for water consumption can eventually lead to an increased and too-high reservation of possibilities for water supply projects. Regarding the actual realisation of a project this makes no difference because of the Ten Year Plan (see Ten Year Plan). It means in fact only that more alternatives than strictly necessary would have to be worked out at an early date.

The water resources for the water supply given in the Structural Outline Plan are mainly the fresh groundwater and surface water of the Rivers Rhine and Meuse. The total available freshwater is estimated, on the basis of regional studies, at about 1900 million m^3 per year. At the moment public water supplies use about 630 million m^3 groundwater per year. Industry uses about 500 million m^3 groundwater per year, half of which is used as cooling water. As far as surface water is concerned, for drinking and industrial water supply, the discharge characteristics of the Rivers Rhine and Meuse and the extent of their pollution determine the scale of works which are necessary for the storage, purification and transport of the water.

The essentials of the Structural Outline Plan for Drinking and Industrial Water Supply are summarised in a number of conclusions which form in their totality the policy guidelines of the Dutch Government regarding public water supply.

Some important conclusions deal with water management in general. It is stated for example that water management in the catchments of the Rivers Rhine and Meuse has to ensure the availability of water in terms of quantity and quality. In another important conclusion it is laid down that groundwater has to be used as far as possible only for domestic water supply and certain industries for which high quality water is an essential condition.

Other conclusions deal with the promotion of the economic use of water, the development of advanced water treatment techniques, the protection of groundwater against pollution, the distribution of different qualities of water for industry, water tariffs, the concentration of waterworks companies in larger units and the adoption of procedures for a good and timely consideration of all interests involved. Another part of the Structural Outline Plan consists of a survey of potential water supply projects. In this context the number of projects and their reservation from the planning point of view is rather important.

A surplus in the number of project alternatives is necessary in order to gain flexibility in the choice of projects for the Ten Year Plan which is dealt with later. All projects are subjected to close scrutiny at this early stage and the consequences of such projects have to be determined. The participation and comments of interested parties are welcome during this planning phase because it reduces the chance of conflicts with other interests at a later stage in the Ten Year Plan or during the realisation of the separate water supply projects.

The reservation of areas from the planning point of view was mentioned earlier. The purpose of the Structural Outline Plan is to ensure the water supply of the Netherlands in the future. This makes the planning protection of the areas concerned necessary in order to indicate, and if possible avoid in time, developments which may render the realisation of a particular water supply project impossible. The exact determination of the boundaries of an area is not so essential during this stage. Priorities are set in the Structural Outline Plan by making a difference between projects which have to be realised in the short term (up to 1980) and projects which will be realised in the longer term (1980–2000).

Finally the Structural Outline Plan deals with procedures for the preparation and adoption of different plans. The Structural Outline Plan is subject to the statutory procedures of the so-called key planning decision. This is an extensive procedure of advice and participation of interested parties. The Structural Outline Plan is then brought into Parliament for discussion.

The procedure for setting up the Ten Year Plan is also given in the Structural Outline Plan. The draft for the Ten Year Plan is worked out under the direction of the Dutch Waterworks Association, which has created a special planning department, and is offered to the Minister of Public Health and Environmental Hygiene. After having asked the advice of the different advisory committees, the Minister finally adopts the Ten Year Plan. Officially, the Ten Year Plan does not provide for procedures with hearings of interested parties.

Furthermore a bill concerning the adoption and realisation of specific water supply projects for the catchment, storage, infiltration and transport of water in the Ten Year Plan is announced in the Structural Outline Plan.

Another aim of this law is to avoid the realisation of other projects which are not in agreement with the Structural Outline Plan or the Ten Year Plan.

3.3.4 The Ten Year Plan

The Ten Year Plan is the realisation of the policy guidelines of the Structural Outline Plan for the next ten years. The Ten Year Plan makes a selection of projects to be actually built for catchment, storage, infiltration and transport of water. The structure of the Ten Year Plan can be compared with the Structural Outline Plan. Because of the shorter planning period the Ten Year Plan is, and has to be, formulated in a more concrete way. The prognoses for water consumption can be given more precisely and the survey of potential water supply projects in the Structural Outline Plan is replaced by a water catchment and distribution plan.

The Dutch Waterworks Association (VEWIN) has created five Regional Standing Committees for setting up the Ten Year Plan. These Regional Standing Committees include representatives of waterworks, the Dutch Waterworks Association, the Government Institute for Drinking Water Supply and the Provincial Planning and Water Management Authorities.

3.3.5 Participation in planning

The preparation of planning is organised in the Structural Outline Plan and the Ten Year Plan in order to ensure that the choice of the finally realised water supply projects is based on careful and responsible consideration of interests. The participation of interested parties in the preparation of plans and decisions is therefore welcome and necessary. However, there should be attention paid to the fact that unworkable and long procedures for such participation lead to unnecessary delays in the realisation of projects, and that efficient management becomes impossible. Too-long procedures become dangerous because a project could already be outdated by new developments at the time of its completion. The planning of public water supply can be subdivided into three stages; the Structural Outline Plan, the Ten Year Plan and the actual realisation of the different water supply projects. The participation of interested parties in one or other form is possible during each of these three stages but secondary plans and studies should be avoided as far as possible. The forms of participation in the Structural Outline Plan are regulated in the statutory procedures for key planning decisions. The participation of all interested parties (in public hearings etc.) is absolutely essential in this planning phase.

During the stage of the Structural Outline Plan, research and studies are carried out and the possibilities for water supply in the future should be discussed on a wide basis. In fact, possible alternatives are worked out and their merits are assessed. During this stage, planning cannot and should not be worked out in detail.

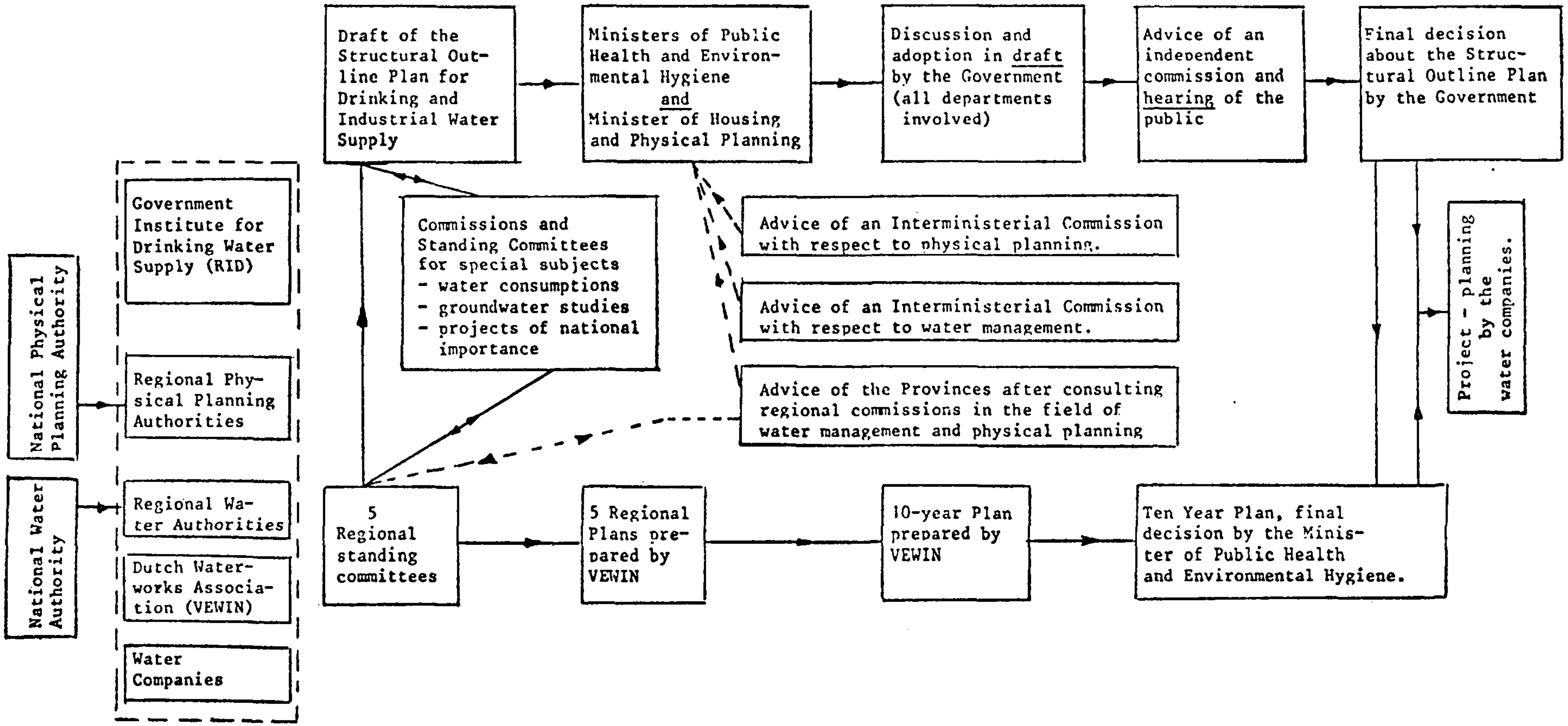
During the stage of the Ten Year Plan a selection is made from the alternatives which are given in the Structural Outline Plan and the points which were raised during participation in the Structural Outline Plan are taken into account. Participation at the stage of the Ten Year Plan is limited to the advice of expert commissions and government institutions. This procedure makes it possible to adapt a Ten Year Plan in due course to new developments.

For the realisation of separate projects a number of approvals have to be obtained which consider, for instance, structure planning and water management points of view. Each of these approvals, which are formulated finally in licences, has its own legal participation procedure where complaints can be raised against the choice which was made for projects earlier in the Ten Year Plan and the Structural Outline Plan.

3.3.6 The organisation of planning

Previous chapters indicated the importance of the Structural Outline Plan and the Ten Year Plan and they indicated the lines along which the different plans have to be prepared and adopted. The total of activities in the planning process is indicated schematically in Graph 9. It is obvious that whilst aspects of physical planning and water management determine policy guidelines for public water supply planning to a large extent, these aspects actually pervade the whole planning process. The schematic drawing in Graph 9 indicates the relationship between the engineers preparing the plans and the politicians and administrators who evaluate and adopt the plans. The left hand part of the scheme is mainly technically orientated, the right hand part of the scheme is purely administrative where the role of the engineer is limited to the presentation, the advocacy and finally the adaptation of the plans.

The origins of all activities are in fact the five Regional Standing Committees which are set up by the Dutch Waterworks Association (VEWIN) in cooperation with the Government Institute for Drinking Water Supply (RID). Changes and new developments are noticed here



Graph 9—Planning activities for public water supply in the Netherlands.

first and communicated to other levels. The Regional Standing Committees receive data and information mainly from water management and planning institutions, the Government Institute for Drinking Water (RID), waterworks and the Dutch Waterworks Association (VEWIN). These data are used by the Dutch Waterworks Association for the compilation of the Ten Year Plan and by the Government Institute for Drinking Water for setting up the Structural Outline Plan. In addition to that, both institutions make use of results obtained from different commissions and standing committees which are concerned with specialist studies. The drafts of both the Structural Outline Plan and the Ten Year Plan then follow the procedures described.

3.3.7 Summary

The Structural Outline Plan and the Ten Year Plan form a coordinated controlling unit for the implementation of policy guidelines regarding public water supply. The Structural Outline Plan extends over a planning period of thirty years; the Ten Year Plan covers a planning period of ten years. Certain statutory procedures have to be followed for their adoption.

For the realisation of water supply projects the waterworks use, in the first instance, approvals within the framework of national and regional legislation. There are sometimes separate statutory procedures for participation in these approvals. Both the Structural Outline Plan and the Ten Year Plan aim at good coordination of future drinking and industrial water supply where separate water supply projects are considered adequately and in a responsible way. The very fact that a water supply project is part of the Structural Outline Plan and the Ten Year Plan should result in a simplification of the various procedures of approval. This should be possible because such a project is discussed and examined at a quite early date and because the national and regional importance of the execution of such a project is indicated in time. The Structural Outline Plan and the Ten Year Plan contain decisions at Government level which are based on total planning of the water industry. This is the firm basis for measures which have to be taken at national and regional level in order to ensure public water supply in the future.

4 The role of the waterworks engineer in planning and its limitation in relation to the role of other specialists

4.1 The waterworks engineer as a profession

The waterworks engineer is a person employed by a water undertaking who has an academic or comparable engineering training.

—The term waterworks engineer actually does not express anything of the necessary training background or the kind of job such a person is supposed to do; it merely indicates the kind of employer an engineer has chosen.

—Depending on the size of the water undertaking and on the diversity of engineering and management tasks which have to be fulfilled, the waterworks engineer has to adapt himself completely to the sometimes rather specific requirements of a given task.

—A survey of the reports of the member countries however, gives a somewhat uniform impression of the general training background and activities of waterworks engineers.

—The majority of waterworks engineers at all levels have a civil engineering background with early specialisation in the "water business". There are a number of mechanical and chemical engineers too, in particular where the activities of a water undertaking include extensive pumping and the operation of large water treatment plants. Due to the increasing diversity of activities there is, especially in larger water undertakings, a strong tendency to employ specialists such as electrical and electronic engineers, microbiologists, limnologists etc.

4.2 The role of the waterworks engineer in planning

The role of the waterworks engineer in planning is determined by the following factors:

(a) The level of planning (town, region or country).

(b) The kind and size of the project to be planned.

(c) If the planning is done on municipal level:
—the size and structure of the water undertaking
—the presence of specialists.

(d) If the planning is done at regional or country level:
—the status and capacity of regional or national waterworks associations
—the presence of (and cooperation with) other planning authorities and specialists.

The multitude of possible planning patterns and technical and administrative levels make it extremely difficult to characterise adequately the role of the waterworks engineer in planning.

It is, however, possible to identify some generally valid points which apply to most of the waterworks engineers involved in planning:

(a) The waterworks engineer works from his mainly technical background.

(b) In particular in industrialised countries, the waterworks engineer faces in his work almost continuously economic, administrative, legal and other special problems.

(c) The waterworks engineer becomes increasingly aware of the limits of his mainly technical background. He has to acquire additional knowledge in order to be able to carry out his work successfully. There is however a danger inherent in this positive approach: the waterworks engineer can easily develop into a jack-of-all-trades and master of none.

(d) In particular in larger organisations or projects the efforts of the waterworks engineer concentrate on becoming able to communicate and cooperate in a team with other specialists, and to use his technical background to contribute to the achievement of the determined end of a plan.

(e) In a team with other specialists, the technical know-how of the waterworks engineer is extremely valuable and almost irreplaceable. It is often the waterworks engineer who reduces problems to their correct proportions and who ensures that all practical aspects of planning are considered.

It goes without saying that this development of a distinct professional profile resulted in certain organisational structures:

(i) National and international congresses and meetings promote cooperation and the exchange of experience and help to coordinate research activities.

(ii) National (and partly regional) waterworks associations form their own advisory and planning groups which consist of full-time professionals. On the one hand, they assist water undertakings in solving their specific problems, on the other hand they represent the water industry completely in government planning authorities, give advice to policy makers and sometimes take care of the compilation of structural outline schemes for long term planning and plans for middle term planning.

(iii) National and regional standing committees and commissions on technical, administrative and planning matters coordinate the policies of the water undertakings, standardise procedures and stimulate the exchange of experience.

(iv) There is increasing cooperation between water undertakings. Larger undertakings with a bigger engineering capacity cooperate with and sometimes sell their services to smaller water undertakings.

4.3 Conclusions and outlook

What is said above of the role of the waterworks engineer in planning is in principle valid for industrialised countries and developing countries.

The only essential differences are:

—In developing countries waterworks engineers generally can work as planners and engineers at the same time. Their field of activity is considerably wider and their judgement is not subject to as many constraints as their colleagues in industrialised countries.

—Due to this single and sometimes rather isolated position the average waterworks engineer in developing countries carries a higher personal responsibility than his colleague in an industrialised country who mostly works in a team of specialists.

—In planning, waterworks engineers in developing countries have to face mainly technical and organisational problems; their colleagues in industrialised countries are considerably more concerned with other problems such as administration, statutory procedures, laws, industrial pollution, public participation in planning etc.

The role of the waterworks engineer involved in planning will very likely undergo further changes in future.

In industrialised countries a further increase in the complexity of the planning process for water supply and its procedures can be expected. The reason for this will be mainly growing public participation, more stringent environmental criteria and the increasing number of colliding interests.

This development will not change so much the professional life of the average waterworks engineer but it will create the need for a new type of waterworks engineer who considers planning as an essential and challenging management task. Besides his technical background such an engineer should have some training in economics and/or planning. Because a large percentage of his work will be dedicated to teamwork and to the advocation of programme points on various levels, he must be able to communicate actively and he must have a fair knowledge of group dynamics and dialectics.

In developing countries, the waterworks engineer involved in planning will have to face in the future more and more of the non-engineering problems which make presently more difficult the professional life of his colleagues in industrialised countries.

It remains to be hoped that the experiences and efforts of industrialised countries in the planning of water supply will help waterworks engineers in developing countries to avoid some of the problems discussed here.

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Appendix 1

Influencing factors on the prognosis of drinking water consumption in Amsterdam (1970)

The total water consumption was divided into the following categories:

(1) Domestic consumption
(2) Public consumption
(3) Trade consumption
(4) Industrial consumption

The year 1966 was chosen as reference year and drinking water consumption during this year in the four categories was 35,5; 7,8; 3,0 and 15,8 million m³.

1 Domestic consumption

The following determinants were used:

(a) number of dwellings dating from before 1900[1]
(b) number of dwellings dating from after 1900[1]
(c) average occupation figure of the dwellings
(d) sanitary installations
(e) water using equipment
(f) number of connections to warm water boilers
(g) number of cars per 1000 dwellings
(h) number of gardens per 1000 dwellings
(i) water tariff
(j) price elasticity

1. *Especially important for Amsterdam*

The determinants are based on consumption analyses and completed with data from national reports of 1965 and 1970.

2 Public consumption

This is the consumption in hospitals, Government buildings, schools, buildings serving cultural or sporting events, public consumption points, shops and laundries. The following determinants were used:

(a) number of inhabitants in the distribution area
(b) consumption per inhabitant[2]

2. *Based on consumption analyses, German data from DVGW and research of the Johns Hopkins University, Baltimore.*

3 Trade consumption

This is drinking water consumption in wholesale trading firms, banks, insurance, printers, hotels and restaurants, transportation and freelance professions.
The prognosis uses the following determinants:

(a) number of people working in this category[3]
(b) potential consumption per individual worker in this category (including air conditioning etc.)
(c) the water tariff
(d) the price elasticity

3. *Based on figures from reports of the Town Council of Amsterdam, the Government Planning Institute and the Ministry of Economy.*

4 Industrial consumption of drinking water

The following determinants were used for the consumption of drinking water:

(a) total production of the petrochemical industry
(b) total production of the metal industry
(c) total production of the paper industry
(d) total production of the printing industry
(e) total production of the food industry
(f) total production of gas- and electricity works
(g) total production of other industries
(h) development of the technologies of water use in industry
(i) the change in connections of industries to the public water distribution system
(j) water tariffs
(k) price elasticity
(l) use of raw water.

The data for production are based on reports of the Town Council of Amsterdam, the Government Planning Institute and the Ministry of Economy.

The data for technologies are based on reports of the Standing Committee on Regional Planning, the Central Commission on Drinking Water Supply and publications in "Water in Industry" (1965); "GWF" Heft 48 (1969); "Wasser, Luft und Betrieb" Heft 1 (1970).

There was no direct investigation and collection of data in the industries. An inquiry was, however, held in 1970 in order to improve the prognosis but it did not yield much useful information.

Summary: The available information for this prognosis was restricted to quite general and non-detailed regional and country-wide publications. Because of this it was often necessary to use data from foreign literature. The lack of information was in this case the crucial point.

Appendix 2

A simplified calculation of domestic water consumption in the town of Rotterdam 1973/1975

1 Determinants and weighting factors

The determinants and weighting factors were chosen after studying the following publications:

- —Netherlands: Regional Planning of the Standing Committee on Water Consumption.
- —Germany: Prognose des Wasserbedarfs in der Bundesrepublik Deutschland bis zum Jahre 2000.
- —Great Britain: Journal of the Institution of Water Engineers, vol. 21 nr. 3.

Finally the determinants and their weighting factors for the year 1970 were taken as:

(1) Water-toilets 33% (varying from 30% to 42%)
(2) Personal hygiene 29% („ „ 20% to 35%)
(3) Laundry 12% („ „ 10% to 17%)
(4) Dish washing 8% („ „ 6% to 11%)
(5) Cooking 8% („ „ 4% to 18%)
(6) Gardening, car-washing etc. 8% („ „ 4% to 14%)

2 Development of the determinants

2.1 Water-toilets

The average water consumption in 1970 was about 7,8 litres per toilet for flushing. This figure is expected to increase to about 8,4 litres for the year 2000. In addition to this it can be expected that the average number of flushings of 5 per toilet per day in 1970 will increase to 5½ in the year 2000. This will result in a consumption increase of about 18%. If a linear pattern is assumed the following index values can be used:

year	index value
1970	100
1980	106
1990	112
2000	118
2010	124

These data are from a report of the Standing Committee on Water Consumption.

2.2 Personal hygiene

The development was estimated on the basis of a national inquiry as follows:

1970
1,3 showers or baths per week and person Index value 100
1980
1,5 „ „ „ „ 115
1990
1,7 „ „ „ „ 130
2000
1,9 „ „ „ „ 145
2010
2,1 „ „ „ „ 160

2.3 Laundry

It is expected that the total number of washing machines will not increase any more. A slight increase in the frequency of use will be compensated by the trend towards smaller families. It is assumed that all of this will result in a constant index value.

2.4 Dish washing

The water consumption of an automatic dishwasher is about double that of dish washing by hand but the frequency of use of a dishwasher is lower. It is estimated that the use of dishwashers results in an average increase of water consumption of about 50%.

At present about 6% of the families in Rotterdam own a dishwasher. The Standing Committee on Water Consumption expects that in the year 2000 about 60% of the families in big cities will have dishwashers.

From these data, the development of the index values for dish washing is estimated as follows:

	Big cities	
	percentage of dishwashers	index value
1970	5	100
1980	10	200
1990	32	640
2000	50	1000
2010	60	1200

The weighting factor for dish washing (0,08) is divided into one weighting factor for dish washing by hand and one for dishwashers. The index values for dishwashing by hand are calculated from the dishwasher index values. The resulting index values are:

	Big cities	
	dish washing by hand	dish washer
Weighting factor	0,07415	0,00585
year	index value	
1970	100	100
1980	95	200
1990	72	640
2000	53	1000
2010	42	1200

2.5 Cooking

The expected development of index values is as follows:

year	index value
1970	100
1980	104
1990	108
2000	112
2010	116

2.6 Gardening, car washing etc.

The average total value for this determinant for the year 1970 was 9 litres per capita per day.

According to local enquiries a car is washed on average every 3 weeks and about ¾ of the families in Rotterdam owned a car in 1971. If about 90 litres of water are used for one car wash the consumption will be:

$(90 \times \frac{3}{4})/21 = 3$ l per family per day.

Water consumption for gardening was about 2 litres per capita per day.

The index values were determined as follows:

	1970	1980	1990	2000	2010	Weighting factor (relative)
Garden	100	167	235	300	367	22
Car	100	152	167	167	167	11
Various other uses	100	89	78	66	55	67
Weighted index	100	113	122	129	136	

3 Determination of the weighted indices

In the calculation a difference is made between "person-related consumption" and "dwelling-related consumption". Water consumption in different categories is calcûlated by means of the weighted indices for these two consumption classes.

"Person-related consumption"
= water toilet
+ personal hygiene
weighting factor = 0,62

"Dwelling-related consumption"
= rest of the consumption
weighting factor = 0,38

4 Prognosis of the number of dwellings and the population number in Rotterdam

Year	Number of dwellings	Number of inhabitants	Dwelling occupation figure
1970	235 000	686 586	2,92
1973	235 000	654 024	2,77
1980	243 600	575 000	2,36
1985	244 500	534 500	2,19
1990	245 500	530 280	2,16
2000	247 500	519 750	2,1
2010	249 500	523 950	2,1

5 Calculation of water consumption

The calculation method is given in simplified form in Tables 1 and 2.

TABLE 1

INDICES FOR DOMESTIC WATER CONSUMPTION IN ROTTERDAM

	1970	1980	1990	2000	2010	Absolute weighting factor	Relative weighting factor
Water-toilets	100	106	112	118	124	0,33	0,53
Personal hygiene	100	112	124	136	147	0,29	0,47
Final index for person-related consumption	100	109	118	126	135	0,62	—
Dishwashing by hand	100	95	72	52,5	42	0,07415	0,204
Dishwasher	100	200	640	1000	1200	0,00585	0,016
Laundry	100	100	100	100	100	0,12	0,32
Cooking	100	104	108	112	116	0,08	0,22
Garden, car etc.	100	113	122	129	136	0,09	0,24
Final index for dwelling-related consumption	100	105	110	114	118	0,38	

TABLE 2

SIMPLIFIED CALCULATION OF DOMESTIC DRINKING WATER CONSUMPTION IN ROTTERDAM

	1970	1980	1990	2000	2010	Weighting factor
Final person-related consumption index (Table 1)	100	109	118	126	135	
Occupation index of the dwellings	100	81	74	72	72	
Corrected final person-related consumption index	100	88	87	91	97	0,62
Final index for dwelling-related consumption (Table 1)	100	105	110	114	118	0,38
Consumption index per dwelling	100	94	96	100	105	
Consumption in m³/dwelling per year	110	103	106	110	116	
Expected number of dwellings	235 000	243 600	245 500	247 500	249 500	
Expected number of population *	686 586	575 000	530 280	519 750	523 950	
Expected occupation figure	2,92	2,36	2,16	2,1	2,1	
Consumption in litres per capita per day	103	120	134	144	151	
Consumption in million m³ per year	25,8	25,1	26,0	27,2	28,9	

Comments:

—Correction of the final indices for "person-related" consumption with the average occupation of a dwelling:

Corrected final index for person-related consumption =

$$\frac{\text{final index} \times \text{dwelling occupation figure}}{100}$$

—Calculation of the consumption index per dwelling:

Consumption index =

final index for "person-related" consumption × weighting factor + final index for "dwelling-related" consumption × weighting factor.

—Calculation of the consumption per dwelling:

Consumption per dwelling =

$$\frac{\text{consumption per dwelling in 1970} \times \text{consumption index}}{100}$$

—Calculation of the consumption in litres per capita per day:

$$= \frac{\text{consumption per dwelling } (m^3/\text{year}) \times 1000}{365 \times \text{dwelling occupation figure}}$$

—Calculation of the total consumption in m^3 per year:

total consumption: number of dwellings × consumption per dwelling.

6 Evaluation

This is another example where the lack of relevant information is the biggest problem for a prognosis. Sometimes even foreign sources of information have to be used.

It was already known in 1973 that the prognoses were too high. This was mostly due to other expectations for the number of the population, number of dwellings, dwelling occupation figures and the general reliability of available information.

It will be useful to identify some of the reasons for the unexpected differences in 1973.

—It is not certain whether or not the "dwelling-related" indices remain correct if the occupation figure of the dwellings decreases.

—The population in the prognosis area decreased more and faster than expected. The increase in the number of dwellings did not come up to expectations either.

—It seems that the number of showers and baths installed in 1973 was higher than expected. The frequency of their use however was overestimated in 1973.

—The frequency of use of different water consuming equipment was estimated independently in 1973. In the meantime an extensive investigation has been carried out into this problem.

—The specific water consumption of certain water consuming equipment, such as for example washing machines, was wrongly estimated. For example the German "Merkblatt W410" gives a value of 250–300 litres for 4 kg of laundry if it is washed by hand; a washing machine uses only 100–180 l water for the same amount of laundry because the rinsing is more efficient.

An additional problem in the case of Rotterdam is that domestic consumption is generally not metered and leakage in the distribution system is not known. It is therefore impossible to determine exactly the domestic consumption. In the near future an attempt will be made to improve the data which are relevant to the model. In addition to this there are plans to extend the approach to include:

—the influence of the family structure

—the relationship between income and the numbers of water-consuming devices

—the influence of the renovation of dwellings on the numbers of water consuming devices

—the influence of the quality of dwellings on the numbers of water consuming devices.

Appendix 3

Prognosis of the industrial consumption of drinking water in the town of Rotterdam

There are no numerical calculations given in the following example. It is rather a report summarising the difficulties which have arisen in a study which is taking place at present in Rotterdam.

At the beginning only total water consumption figures in the various bulk consumption categories were known. These bulk consumption categories are the basis for industrial water tariffs. The situation after one year of work is now as follows:

From 1962 onwards water consumption by large-scale industrial consumers was split up into a number of branches (26 with a water consumption exceeding 100 000 m³ per year). Small-scale industrial consumption is distributed over these branches.

The municipalities to which water is sold in bulk are asked for data on industrial consumers whose consumption exceeds 10 000 m³ per year; these consumptions are also split up into a number of branches. The industrial consumption in these municipalities which are smaller than 10 000 m³ per year are distributed over these branches in the same fashion as the small industrial consumptions in Rotterdam. This is done in order to look for relationships with production, number of workers etc.

The whole classification is set up in a way that it can be fully integrated into an existing regional economic model.

An enquiry was held in 1974 amongst the consumers whose water consumption exceeded 100 000 m³ per year. This enquiry was repeated in 1975 in a more extensive way and with more detailed questions. In addition to that, the enquiry was extended to industries with a water consumption between 20 000 and 100 000 m³ per year provided the data were relevant for the characterisation of a process-dependent consumption category. The questions refer to the use of the water, expected water consumption up to 1978, the extent of recycling at present and in the future and the direct use of ground- and surface water.

Participation of the industries approached was very high: 98% in 1974 and 90% in 1975 at a much increased number of approached industries.

A start was made by interviewing the most important water consumers (with a consumption exceeding 100 000 m³ per year or with a high consumption growth rate). The purpose of these interviews is the collection of additional data on the expected development of production and the economic and technological situation. The detailed form of the model that will be used is not yet determined, but the following can be stated already:

(a) Separate prognoses will be made for the industries interviewed and they will be integrated into the total prognosis.

(b) For other industries with a water consumption higher than 100 000 m³ per year (they are mostly situated in municipalities to which drinking water is sold in bulk) consumption is only known per branch category. It seems that these categories are not yet fully homogeneous. The available information will be completed with official Government data for the expected economic development.

(c) Industrial branch categories will be used also for industries with a water consumption between 20 000 and 100 000 m³ per year. An enquiry with a questionnaire can yield additional data for the development of specific water consumption. In some cases, however, it is advantageous to use the number of workers instead of production for estimating water consumption.

It is expected that industrial branch categories are more useful for this size of industry than for large-scale industries. The reason is that this group of consumers has less opportunity to introduce processes where water is substituted by another medium. This leaves economic development as the most determining factor. The development of specific water consumption will therefore fluctuate less in this case than for industries mentioned under point (a).

(d) Industrial consumption smaller than 20 000 m³ per year is not considered in the enquiry because in this case the industrial branch categories are quite inhomogeneous.

It can be expected that the first result of the prognosis will be available, after about two years of work, in July 1976.

Planification à long terme pour l' alimentation en eau

par M. le Professeur P. L. Knoppert

Résumé

Le rapport s'articule autour de quatre chapitres dont les titres sont les suivants:

1. Considérations générales sur la planification à long terme de la distribution d'eau
2. Méthodes de prévisions de la consommation d'eau
3. Planification à grande échelle du système d'approvisionnement en eau et de distribution de l'eau
4. Le rôle des ingénieurs hydrauliciens dans la planification et sa limitation eu égard à l'existance d'autres spécialistes.

En annexe figurent des exemples numériques illustrant les méthodes de calculs et de conception proposées dans le corps du rapport.

1 Considérations générales sur la planification à long terme de la distribution d'eau

—Planifier c'est prévoir et mettre en oeuvre dans un cadre de possibilité techniques et réglementaires une suite de décisions. Il faut déterminer les objectifs et les critères de choix, établir un ensemble de possibilités et trouver la solution optimale et en conduire la réalisation.

—La nécessité de planifier s'est révélée pendant les années 60. Il est alors apparu évident que, dans les dix prochaines années, les eaux souterraines seraient complètement utilisées, et que les eaux de surface, de plus en plus sollicitées, verraient leur qualité baisser grandement. Enfin, l'accroissement de l'infrastructure des services publics, les oppositions d'intérêts et la sensibilisation de l'opinion publique ont mis en lumière la nécessité d'un plan à long terme négocié.

—Celui-ci a pour objectif d'assurer un approvisionnement en eau correspondant, tant sur le plan de la qualité que de la quantité, aux besoins humains et industriels.

2 Méthodes de prévisions de la consommation d'eau

Le principe général consiste à extrapoler la consommation future sur la base des consommations observées. Connaissant l'évolution de la consommation au cours des années précédentes, on est en mesure de prolonger le trend* pour trouver les valeurs recherchées.

Lors de l'application d'une telle méthode, on est amené à faire certains choix.

—Dans une extrapolation linéaire du type $V_t = a + bt$, l'accroissement par unité de temps est considéré comme constant ($= b$). La constante b peut être soit estimée soit calculée.

—Dans une extrapolation exponentielle du type $V_t = a.b^t$, c'est l'accroissement relatif par unité de temps qui est considéré comme constant.

—Dans une extrapolation logistique de type

$$V_t = \frac{\hat{V}}{1 + 10^{ab-t}}$$

le niveau de saturation futur $\hat{V}$ est estimé

Les méthodes sont relativement intuitives, dans la mesure où la détermination du trend passe dans de nombreux cas par une méthode graphique. L'analyse peut être affinée par exemple, en pondérant les différentes valeurs des consommations en fonction de leur ancienneté, on arrive ainsi à diminuer l'impact des valeurs, relativement anciennes, valeurs issues de situations en partie dépassées, et à augmenter le poids des informations les plus récentes, reflet d'une réalité plus proche.

Les avantages des méthodes classiques de prévision résident dans la rapidité d'exécution et la faiblesse des coûts, la simplicité des calculs à effectuer et des informations à regrouper.

Par contre, leurs inconvénients résident dans les hypothèses qu'il faut accepter pour les appliquer.

—Les chiffres reflètent la véritable demande en eau.

—Les séries du passé sont significatives pour le futur.

—La consommation en eau est considérée comme une fonction du temps.

L'analyse des hypothèses est un domaine très délicat et l'on doit garder à l'esprit qu'il peut y avoir une différence entre consommation effective et demande effective. La distortion entre ces deux valeurs peut provenir de nombreuses sources peu négligables, prix, réglementation, disponibilité etc. . . .

Il ressort de cela que l'on peut analyser la consommation d'eau comme se déroulant dans le temps mais n'étant pas uniquement une fonction du temps.

La prise en compte d'un certain nombre de facteurs exige, avant toutes études générales, que l'on détermine d'abord leur niveau de dépendance ou d'indépendance. On y parvient par l'étude des covariances et des coefficients de corrélations qui permet de déterminer si la relation entre plusieurs facteurs est fortuite ou réelle.

Une fois la corrélation de plusieurs séries statistiques reconnue, on synthétisera ces informations en déterminant par la méthode des moindres carrés une droite de régression.

La régression peut être simple ou multiple, s'il y a plus d'une variable indépendante. L'ajustement peut être également fait sur une courbe.

Les méthodes d'analyses s'inspirant de la régression, c'est à dire de la prise en compte de plusieurs variables explicatives, pour le phénomène consommation d'eau, bien que permettant de travailler avec une plus grande certitude au niveau de la détermination du trend et de la prévision, comportent un certain nombre de désavantages; ce sont ceux inhérents à tout travail dont les matériaux de base sont des séries statistiques (problème de l'exactitude, de la signification pour le futur, du choix de la période que recouvre les séries etc. . . .).

Pour pallier les difficultés statistiques, la tendance générale consiste à adopter l'analyse de système, qui vise à élaborer un modèle mathématique devant refléter le comportement de la demande en eau.

Alors que précédemment la collation des séries statistiques était le point essentiel, dans cette méthode c'est le choix des variables significatives et l'établissement d'équation les reliant qui constituent le travail le plus délicat.

La modélisation est un Art qui se fonde en partie sur des connaissances empiriques et dont la difficulté réside dans le fait de devoir transcrire en relations mathématiques relativement simples une réalité très complexe.

Démarche générale dans l'élaboration d'un modèle.

1. Détermination des questions auxquelles doit répondre le modèle.
2. Elaboration des relations mathématiques.
3. Recherche de données statistiques pour confirmation et amélioration des relations mathématiques.
4. Test du modèle—par comparaison des résultats obtenus et de résultats connus.
5. Vérification de la sensibilité du modèle.

L'analyse de système est utilisée pour les problèmes complexes où il demeure un grand nombre d'incertitudes. Par le biais de cette méthode, les incertitudes sur l'avenir sont moins présentes que dans les méthodes conventionnelles. Les performances d'un modèle dépendent pour une large part de la qualité des estimations et des simplifications.

Dans le cadre d'un modèle portant sur la consommation d'eau, eu égard aux caractéristiques spécifiques de ce problème, de bonnes prévisions ne peuvent couvrir plus qu'une période de dix ans.

3 Planification à grande échelle du système d'approvisionnement et de distribution d'eau

—Pour toute étude de planification générale certains éléments de base sont habituellement considérés comme indispensables.
Trois catégories se dégagent:

(a) le recueil d'informations numériques ou théoriques:
- —demande présente et future en eau
- —analyse du système hydraulique
- —nature et volume des pollutions présents et futurs

(b) une base réglementaire
- —normes de qualité de l'eau
- —critères de qualité de l'eau de surface
- —réglementation sur l'utilisation des produits chimiques, des captages etc...

(c) l'existence de systèmes administratifs et juridiques ayant les moyens de faire appliquer leurs décisions et de sanctionner les contrevenants.

—Dans l'édification des plans à long terme, le nombre des parties prenantes crée des dangers inhérents à tous travaux collectifs.
Conscients du fait que nous ne pouvons tous les énumérer, deux nous paraîssent être d'importance majeure.

1. Le nombre des parties prenantes crée des lourdeurs dans l'avancement des travaux et risque de rendre inadaptées certaines décisions prises dans les phases précédentes.
   Dans certains cas, même, les objectifs risquent d'être dépassés, de ne plus correspondre aux besoins présents.
2. L'intéraction de nombreux intérêts peut désorienter le plan au profit d'intérets spécifiques et compromettre l'atteinte de l'objectif global qui doit demeurer la maximisation du bien-être de la communauté humaine.

—Un exemple de planification concertée:
La planification à grande échelle des systèmes d'approvisionnement et de distribution en eau des Pays-Bas.
Deux types de planning se supperposent.

(a) Un plan directeur.
L'objectif de ce plan est de déterminer le canevas dans lequel les choix peuvent être effectués. Il se compose d'études traitant entre autre des sujets suivants:
- —Prévision de la demande en eau jusqu'à l'an 2 000
- —Etude générale des ressources en eau
- —La politique gouvernementale face à la distribution d'eau
- —Détermination des besoins à court et long terme
- —Etude des procédures conduisant aux différents niveaux à la préparation et à l'adoption des plans

Une fois fixé le plan directeur est débattu au Parlement.

(b) Un plan décénal
Le plan conserve l'adoption de certains projets. Il a une structure comparable à celle du plan directeur mais présente un aspect moins théorique. Aussi bien pour l'élaboration des plans que dans les phases de réalisations, les différentes parties prenantes sont amenées à collaborer activement.

4 Le rôle des ingénieurs hydrologues dans la planification et sa limitation eu égard à l'existence d'autres spécialistes

—Le rôle est fonction:
du niveau auquel se fait la planification (ville, région, pays)
du type et de l'ampleur du projet

—Le rôle est majeur sur le plan technique

—Dans les pays en voie de développement le rôle est encore élargi et peut alors englober la planification et l'exécution.

—Dans l'avenir le rôle de l'ingénieur hydrologue subira certainement des modifications étant donné l'intérêt croissant que porte le public aux problèmes d'eau, l'accentuation de la rigueur des critiques se rapportant à l'environnement et l'augmentation du nombre des parties prenantes.

General Report 2

Control of water supply demand

by J. Anthony Young, C.Eng., F.I.C.E., F.I.W.E., M.I.Struct.E., M.B.I.M.

Director of Operations, Wessex Water Authority

1 Introduction

"Water, Water Everywhere
The very boards did shrink.
Water, Water Everywhere
Nor any drop to drink".

Rhyme of the Ancient Mariner
Samuel Taylor Coleridge

1.1 Why Control Water Demand at All?

1.1.1 After the air we breathe, the unique substance water is the second essential of life. Surely everyone should enjoy the unlimited use of this commodity which represents 71% of the earth's surface, ample for all conceivable needs and unlike space ship earth's other resources which are finite, is automatically re-cycled for re-use by the hydrological cycle?

1.1.2 True although all these statements are, such a naive approach does not withstand examination. Among the essential reasons for the control of demand are the following:—

(a) Fresh water is unevenly distributed over the planet including large and increasing areas of desert where rain seldom falls, glacial areas where water is locked up in permafrost and even in countries where overall resources are ample, e.g. the United Kingdom, most rain tends to fall in mountainous sparsely populated areas and least in the densely populated south east.

(b) While the purification capacity of rivers and the sea, together with the operation of the hydrological cycle, evaporation, etc., in former times guaranteed a reasonable resource, in recent times man, the sorcerer's apprentice, has created complex materials, e.g. nuclear wastes, persistent organo-chlorides, etc., which linger on in some cases for thousands of years after their usefulness is spent as a danger to succeeding generations and often needing sustained and complex treatment to render them safe. The discovery of D.D.T. in Antarctic penguins betrays the seriousness of the problem.

(c) The purification, distribution and reclamation of water needs a large investment in capital and energy terms. We are heavily dependent on finite resources of fossil fuels for energy and these resources must be conserved.

(d) The water in any catchment area, be it surface or underground, is needed for many purposes; water supply, agriculture, industry, fisheries, ecology, amenity, sport, leisure, hydro-electric production, effluent dilution, irrigation, milling, etc. Too much needs flood alleviation works. The list of uses is endless and to manage such a system to attempt to satisfy all users must require control of demand.

(e) All resources are limited and if more than a proper proportion is spent on satisfying water demand, less will be available for some other basic need.

(f) Whilst infinite resources of saline waters are available in the sea, its conversion to potable water is extremely energy intensive and its use, except in specialised situations such as islands or arid areas contiguous to cheap fuel sources, is unlikely to extend unless a cheap and renewable source of energy is found or harnessed, e.g. solar energy.

1.1.3 There is a very real danger that unless greater awareness of these problems is displayed by man, the dire words of the ancient mariner may come true for us or for our children.

2 Control of demand for potable water

2.1 Primary Resource Planning

2.1.1 Control of demand, of course, commences at the resource planning stage of river basin management. For example, to relate river needs to low summer minimum flows, river regulations may be necessary via storage reservoirs used to augment river flows in summer filled with winter high flows, e.g. Clywedog Reservoir in Wales for flow in the River Severn, on the Seine for Paris, etc.

2.1.2 It is considered that control of demand in this sense is beyond the scope of this Report and is, in fact, covered by General Report No. 1—Long Term Planning of Water Supply.

2.2 Methods of Control of Potable Water Demand

2.2.1 There appear to be three main methods of control possible:
(a) Technical
(b) Financial and legal
(c) Educational

These are obviously inter-related and are not meant to be listed in order of importance.

2.3 Technical Methods of Control

2.3.1 SYSTEM ENGINEERING. It is a blinding glimpse of the obvious that to control demand a system must be well designed, well constructed using first-class materials, well-run and maintained with adequate finance to renew worn out plant etc., in good time before large losses occur.

2.3.2 Few, if any, water supply undertakings are in this condition. Early engineers built well and durably but many systems are now well over 100 years old. Because of its nature, most water supply works mains, services and service reservoirs are underground and their physical condition is difficult to observe, either internally or externally. It is also difficult, even with quite comprehensive systems of metering, to arrive at true figures for the loss rates from reticulation systems.

2.3.3 TYPES OF LOSSES. Water starts to be wasted almost as soon as the process starts. Pump gland cooling water often runs to waste where mechanical seals could be fitted; waste water from the operation of chlorinators is not recovered; filter back washing water runs to waste instead of being settled and re-used. Control of demand must start at source works. Once treated water enters the distribution system, leakage becomes the dominant factor, occurring due to excessive supply pressures, old mains, corrosion both external and internal, ground movement, settlement, earth tremors, traffic vibrations, disturbance due to other utilities' operations, and to surge and water hammer resulting from the operation of valves, hydrants, ball valves, pumps, etc.

2.3.4 COMBINED ATTACK REQUIRED. As with most problems, there is no simple answer: a combined attack is necessary on the various factors. Replacement of old and defective mains will not solve the problem unless excessive pressures are reduced, more suitable materials are used for the replacement resistant to corrosion, vibration, etc., and the causes of surge removed or minimised.

2.3.5 MAINS AND SERVICE RENEWALS. The heavy nature of the product and the continuous 24 hours a day, 365 days a year nature of the service provided requires a very heavy capital expenditure in underground mains and services, the age and state of which must give grounds for concern in many countries.

P. Schulhof, in the French report states that while in a well-managed undertaking water waste should not exceed 7–8% (in the author's opinion a very low figure which few undertakings achieve in practice), it may be large when systems are more than 100 years old and have reached a critical stage, and the problem of their renewal over the next few years is acute. The new British Water Authorities now managing the whole water cycle (see Appendix 1) and having to assess priorities between water supply, sewerage, sewage disposal, flood alleviation, etc., in a period of high inflation, are concerned that with the expenditure possible, the renewal of water mains (and sewers) is falling far behind their rate of decay. One medium size undertaking a few years ago estimated 13% of its mains system of 1136 km needed renewal on grounds of corrosion, inadequate size or both. Its programme to remedy this situation has had, due to the effect on charges, to be repeatedly postponed.

From Japan, Kiyoharu Taniguchi reports that the rapid urbanisation taking place around major cities and the provision of new networks which this entails is retarding the replacement of old mains, the leakage rates from which are increasing due to frequent underground construction works in their vicinity and heavy traffic loads.

Differential settlement is a major cause of leakage, damaging pipe joints in Bulgaria, as reported by Hristo Hadjev. The energy crisis has also affected this problem by increasing the price of petroleum derived uPVC mains which have important advantages in flexibility and corrosion resistance when carrying out mains renewals.

2.3.6 OPTIMUM SUPPLY PRESSURE. It is obvious that the pressure of water supplied to the consumer should be adequate for all reasonable uses and these uses may need definition. It is questionable, for example, whether the pressure should be sufficient to drive water driven potato peelers when water is becoming used as a source of energy rather than a proper use of water.

The pressure, if adequate, should not, however, be too high as this is likely to lead to excessive usage, especially in gardens, and increased leakage.

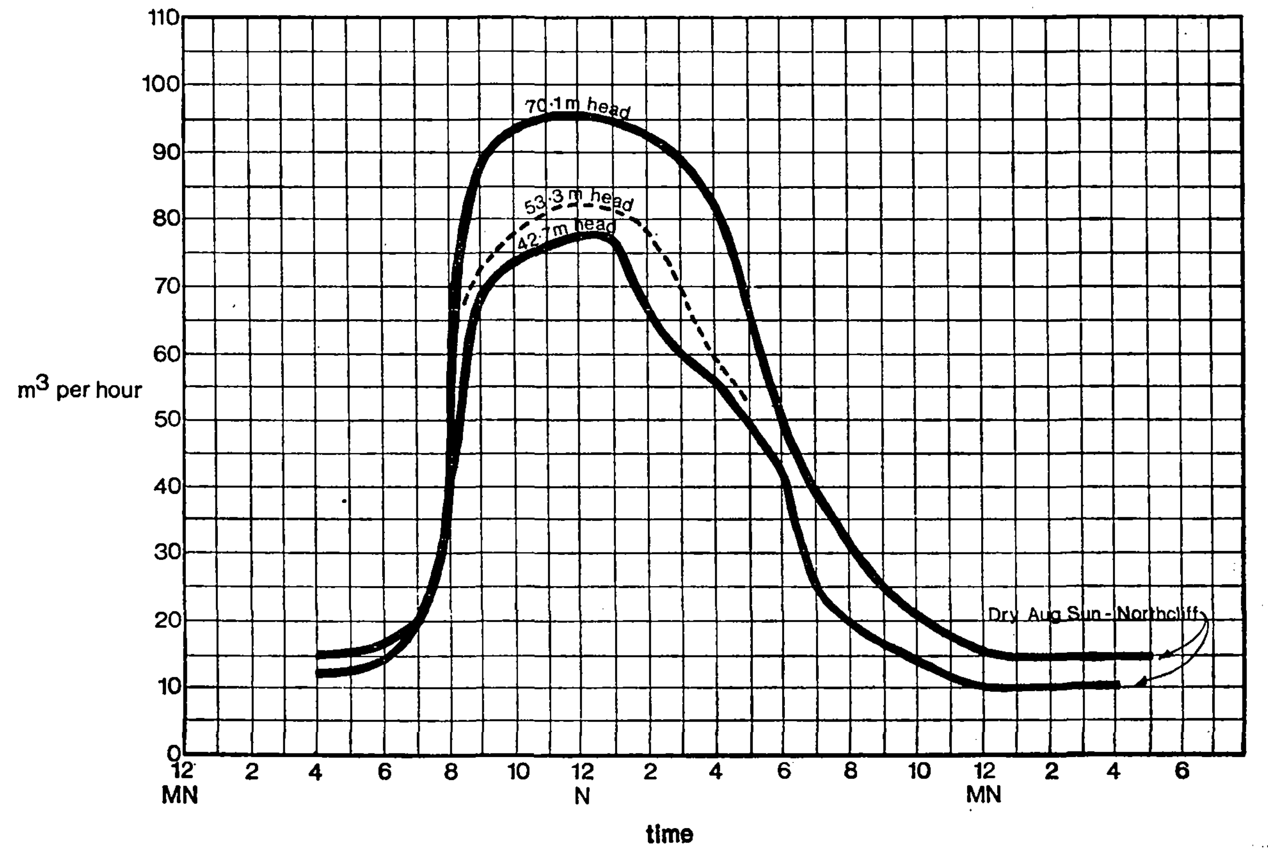


Figure 1—Water demand curves in August at various pressures: Johannesburg, 1974.

National standards on operation supply pressures are reasonably consistant viz:

TABLE 1

United Kingdom	30 metres head
France	30 metres head minimum
Ireland	30,6 metres head (3 bars)
Italy	Pressure Regulation if pressure exceeds 61,2 metres head (6 bars)
Netherlands	20 metres head minimum
South Africa (Transvaal)	25 metres head minimum 100 metres head maximum

It must be recognised that it is simple to maintain reasonable average pressures in flat terrain but much more difficult in undulating areas where successive pressure boosting and pressure reductions may be necessary.

2.3.7 PRESSURE REDUCTION AS A MEANS OF REDUCING DEMAND. Most rapporteurs agree that pressure reduction is a useful means of controlling demand with the exception of the Netherlands where W. C. Wijntjes states that Dutch water

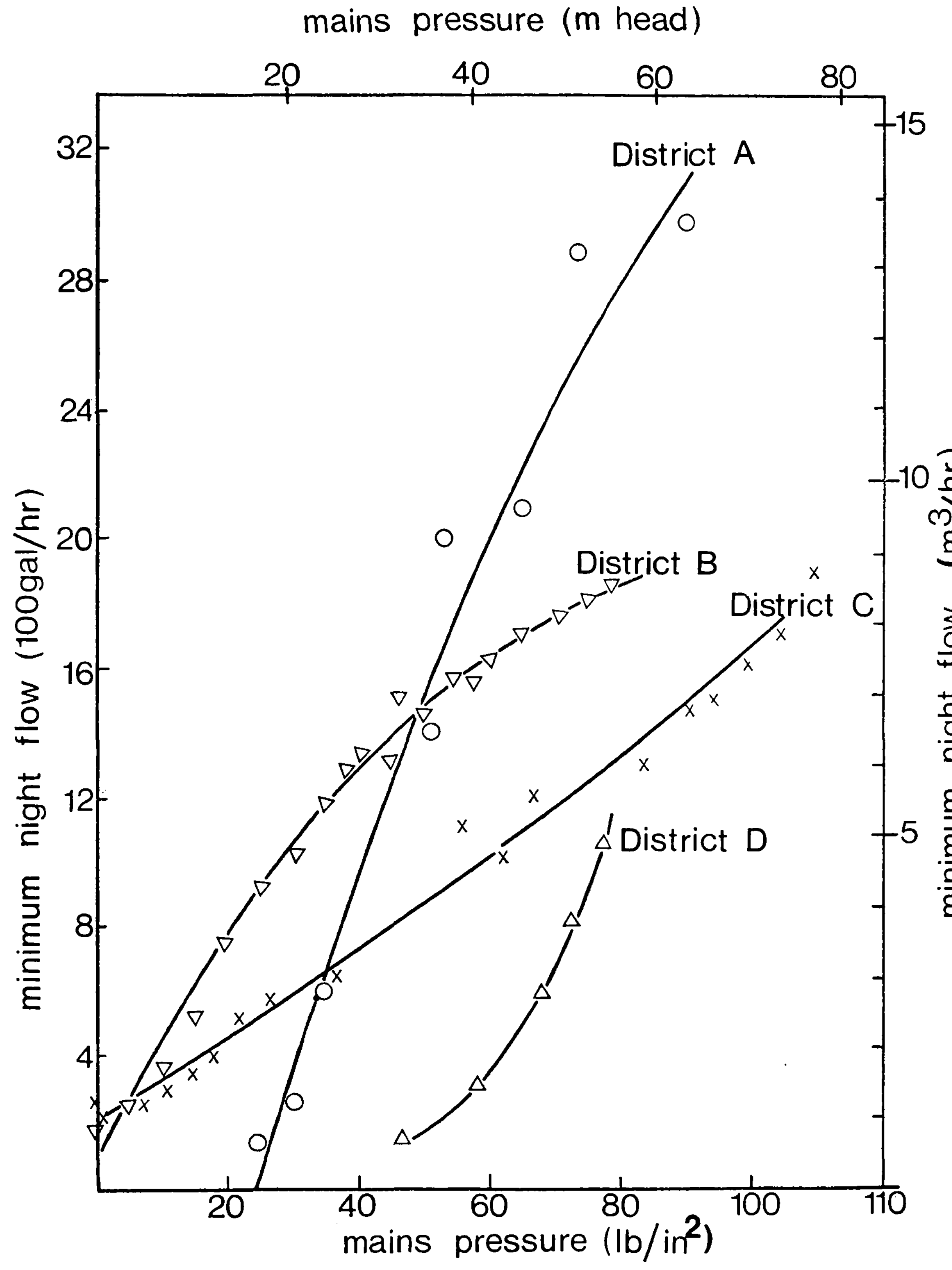


Figure 2—Curves of minimum night flow v. pressure at waste meter. WRC (Great Britain)

undertakings have differing views and a KIWA committee has carried out an investigation in which the networks of mains in some smaller towns were isolated and water consumption at varying pressures monitored. The tentative conclusion is that pressure has little effect on consumption. Bearing in mind the flat nature of the Netherlands, it is likely that pressures are lower to start with, giving smaller opportunities for reduction. In a report from South Africa covering experiments in Johannesburg, a very good relationship between pressure and consumption has been observed (Figure 1). In this case, three successive weeks supply pressure to a zone was varied for a week and the resulting consumption curves plotted. Demand increased or decreased significantly with the pressure and followed the relationship postulated by Ledochowski in 1956.

$Q = kh^x$
Q = flow in m^3/hour
k = a constant depending on the characteristics of orifice
h = pressure head in metres
x = a constant for the particular orifice.

Pressure was demonstrated to follow this curve and it was shown that a rise of 60% in pressure caused a 30% increase in consumption. Tests were also carried out by restricting garden watering to a limited number of hours morning and evening. This saved 20% on consumption at peak demand. A 33% reduction in pressure could have produced the same result. (Reference 1).

Work carried out at the Water Research Centre in Great Britain also shows a definite cause and effect. (Figure 2) (Reference 2). In Denmark, the supply pressure in Copenhagen is regularly adjusted, failure to make the adjustment results in a 5% increase in the 300 000 m^3/day consumption.

Whilst pressure reducing valves are not used in domestic premises they are required for swimming baths and industrial premises.

No formal studies have been made in Finland but random increases in pressure have increased consumption and leakages to such an extent that metered consumers have challenged their bills on the basis that their meters were over-registering. This year a study is proposed where a reduction of circa 200 kPa (25,5 m head) will be made in parts of the network where it is high and the effect on consumption noted.

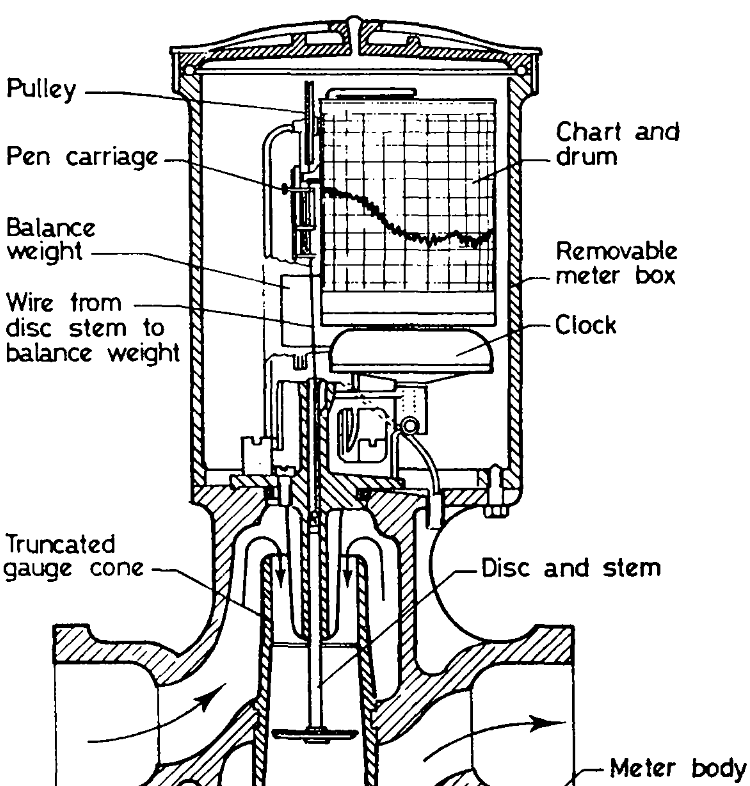
Network pressure in Bulgaria is regulated by systems of pressure reducing valves and also by the throttling of main valves by remote control at certain times.

In Great Britain, in undulating areas where variations in pressure are high, distribution pressure reducing valves of both the dead weight and spring loaded types are used. On mains where there are few consumers and pressures in the main must be retained to feed an adjoining higher zone, fixed ratio pressure reducing valves are fitted on individual consumers' services.

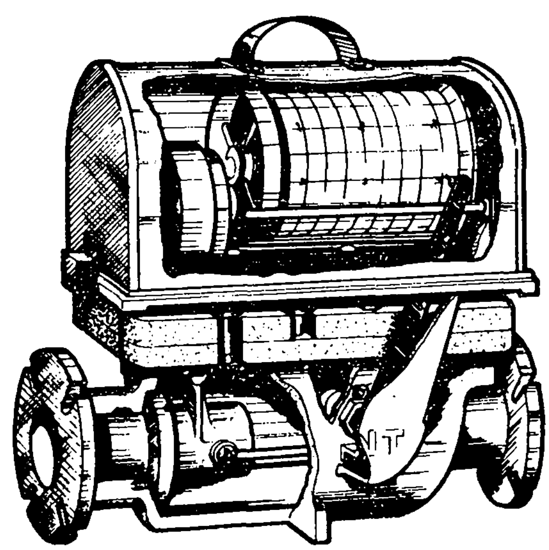
Czech practice is to divide systems into pressure zones with surge protection to prevent pipework being overstressed.

In U.S.S.R., restrictions on water supply to communities are permitted only in emergency cases. Water supply systems are subdivided into three categories with regard to their reliability, as follows:

Category of Reliability	Population Served	Permissible Time Interval for:	
		Delivery lowered by 30% of design flow	Interruption of delivery
1	50 000	3 days	—
2	500–50 000	1 month	5 hr
3	500	1 month	1 day



DEACON WASTE WATER METER



GATE-TYPE WASTE WATER METER

Figure 3—Waste water meters in use in U.K.

To sum up on pressure reduction, this should be introduced wherever possible and whenever mains are renewed the opportunity should be taken to reduce supply pressures if these are higher than those necessary to provide a reasonable supply of water.

2.3.8 LEAK DETECTION AND CONTROL. However well designed and maintained a mains system may be, the detection and repair of leaks will remain the recurrent task facing its managers. The reports submitted indicate that leak detection and control is not generally carried out on a thoroughly scientific or systematic basis. Reports speak of "occasional soundings of mains" and detailed investigations only when waste becomes of major proportions.

Two countries, however, reported waste control systems which could well be considered for adoption on a wider scale.

2.3.9 UNITED KINGDOM WASTE METERING SYSTEMS. Although metering of domestic consumers is not practised at present in the United Kingdom, many authorities and companies have districts controlled by 'waste' meters. These are of the disc or gate type (Figure 3). These meters are either installed in reservoir outlet chambers, in underground chambers or on trailers. In the latter case, they are used by means of hoses attached to hydrants on each side of a control valve which is closed and flow taken through the meter when in operation. These give a 6 hour, 24 hour or weekly record of rate of flow. By valving off districts, the 'nightline', i.e. flow of approx. 3.00 a.m., which is when legitimate consumption is at a minimum, enables the waste in a district to be evaluated. By further valving known as 'shuts', lengths between valves having leaks can be found. The leak is then pinpointed by stethoscope or electronic leak detectors. If the leak is in the public system it is repaired; if private, a notice is served for the consumer to repair. If this is not done in a reasonable period, powers exist for the authoriy to repair and charge the consumer with the cost.

2.3.10 SWEDISH WASTE CONTROL METHOD. An interesting variation of the U.K. method is reported by Sweden (see Figure 4). In this system, instead of waste meters, a precision level meter is installed in the service reservoir supplying the zone. The sensor is about 1 m in length suspended in the water surface of the reservoir and connected by cable to a display instrument located in a radio vehicle adjoining the reservoir. The instrument can detect changes in water levels of 0,2 mm. It is possible to determine the rate of outflow in about 5 minutes and thereafter the system of 'shuts' is similar to the U.K. system. While more limited in its scope than the U.K. system, it is an economical method of introducing systematic waste detection. However, only a few instruments have been manufactured to date and these are used exclusively for waste detection surveys carried out by a Swedish firm of consultants for their client communities.

2.3.11 FUTURE TRENDS. It is obvious that ageing mains systems, rising costs, increasing interference with traffic over mains, are increasing the water industry's awareness of the need for systematic waste detection, but there is still scope for considerable improvement in methods.

One promising development in the United Kingdom is the acoustic correlation leak detector pioneered by the Water Research Centre, Medmenham. The Centre has placed a contract with a leading electronics firm to produce a simplified version of the original apparatus incorporating major developments in electronic circuitry which have taken place over the last few years. The

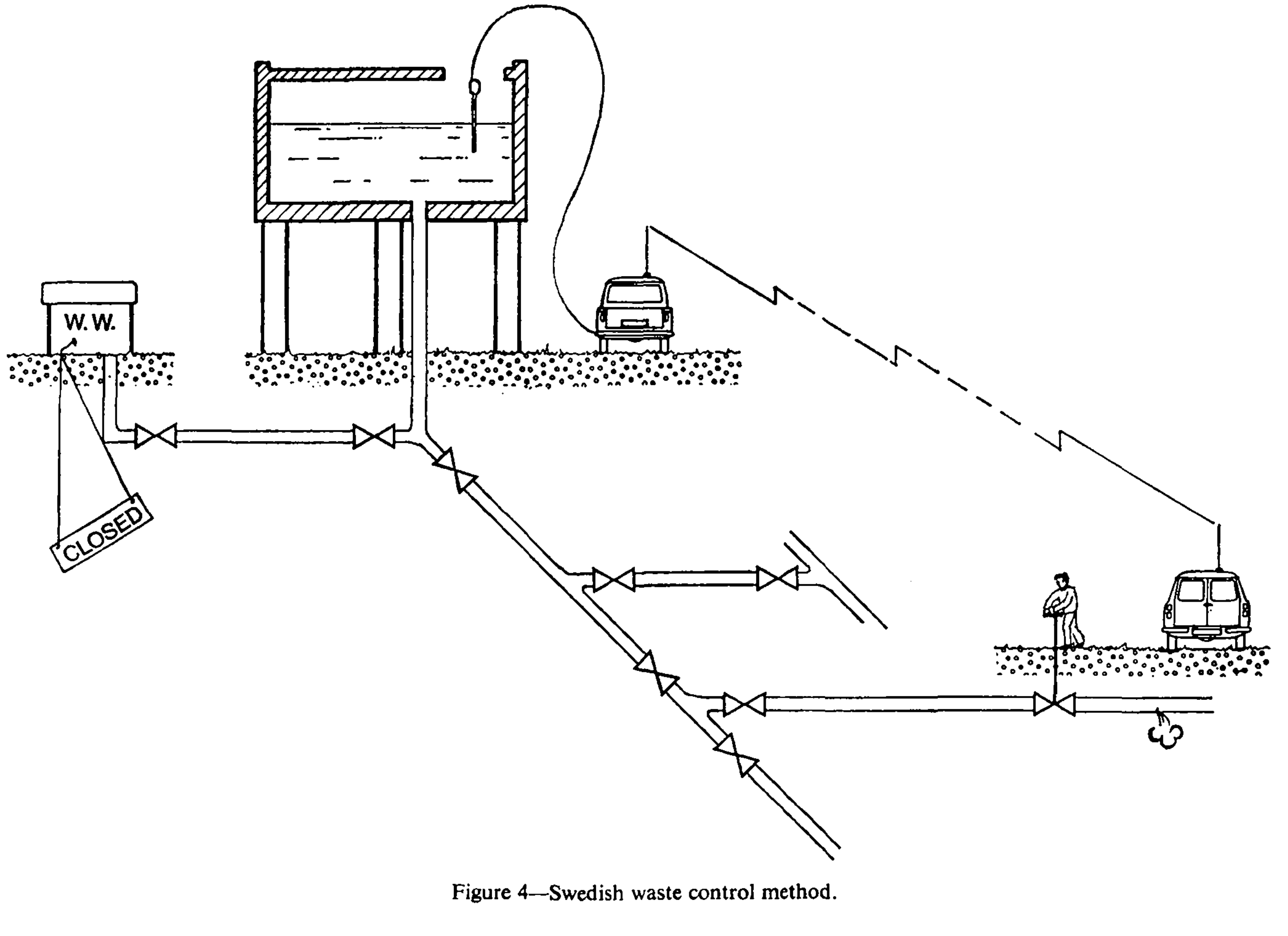


Figure 4—Swedish waste control method.

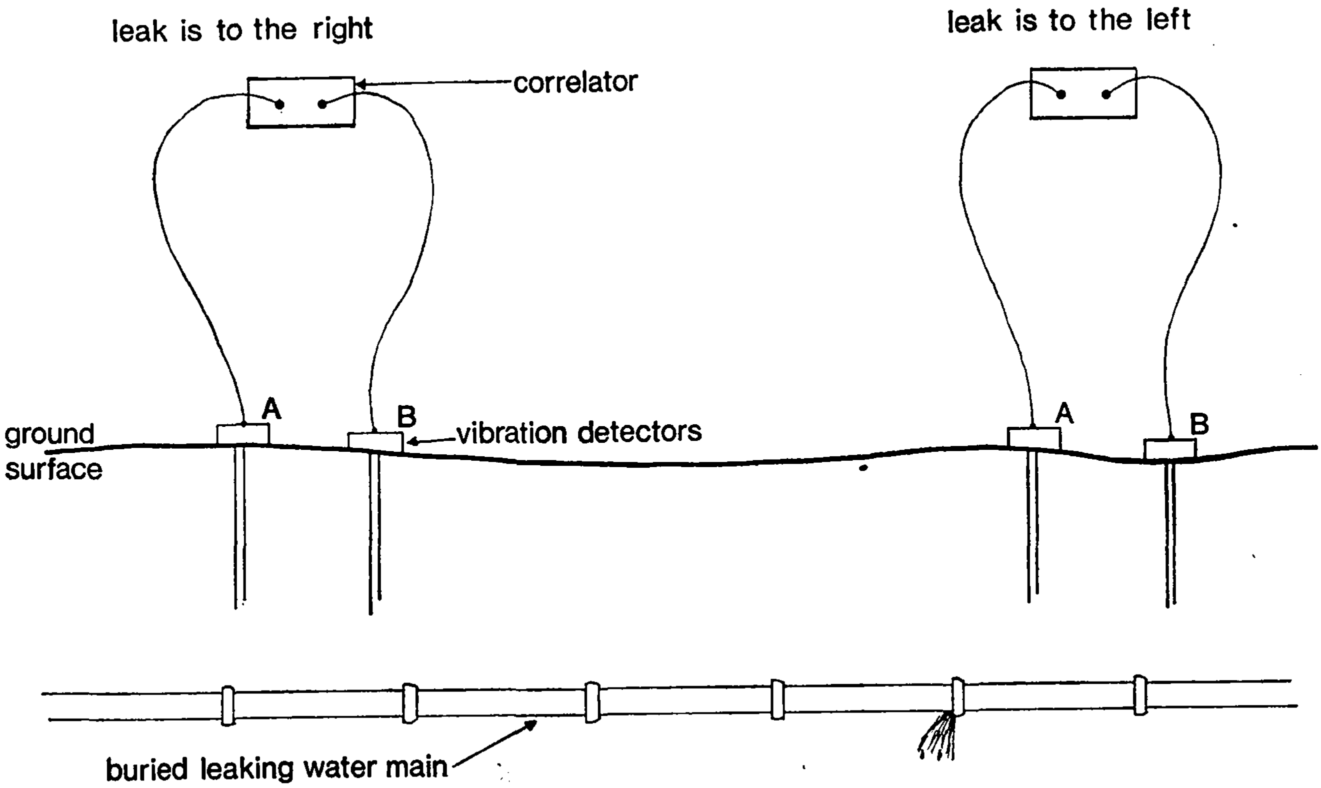


Figure 5—WRC/Plessey acoustic correlation leakage detector.

proposed apparatus (Figure 5) will compare the outputs from two acoustic vibration detectors placed a few metres apart along the line of a main. It will give a correlation trace which indicates that a signal is coming along the main from either the 'right' or the 'left'. The detectors will then be moved to different positions and the same correlation analyses undertaken until the position of the source of vibration is accurately defined. The apparatus will be small enough to be mounted in the back of a car and should be tested in Autumn 1975 using a controlled leak set up at the Medmenham Laboratory where vibration detectors can be readily mounted both on the main and into uniform gravelly soil above the main. The controlled leak will be either free-draining or will play below the local water table. Following these tests it is intended that the firm will be awarded a further contract to develop a "Mark II" equipment for use and evaluation under field conditions by Regional Water Authorities.

2.3.12 REDUCTION OF WASTE OF DOMESTIC CONSUMERS. Waste on domestic consumer's premises may be due to several causes:

(i) excessive supply pressure (see paras. 2.3.6, 2.3.7).

(ii) excessive usage. Difficult to control except by domestic metering and demand reducing tariffs (see paras. 2.4.1, 2.4.2).

(iii) waste or leakage due to defective fittings, e.g. dripping taps, defective flushing valves, defective WC flushes, ball valves, etc.

(iv) poorly designed domestic appliances, e.g. washing machines, dish washers, garbage disposal units using excessive quantities of water.

A study has recently been approved by the Severn Trent Water Authority in the United Kingdom which will involve the detailed analysis of water consumption in 1178 selected households in Mansfield and Malvern. These households will be metered and an attempt will be made to relate the main components of useage (car washing, garden watering, lawn sprinkling, baths, showers, toilet flushing, etc.), to socio-economic and other factors such as family size and age structure, occupation group, social class, appliance ownership, car ownership, garden size, house type, rateable value, tenure of property, house occupancy during day, climate, etc. Householders will receive a small financial incentive to make regular meter readings and to maintain diary records of the use of the main items of water-using equipment. This study will provide, amongst other things, information on waste within the house, from the local distribution system, and—by deduction—on leakage from the mains system. (Reference 9).

2.3.13 CONTROL OF WASTE FROM DEFECTIVE FITTINGS. Control is necessary at the manufacture stage of the fittings and at the installation stage.

Low waste and per capita consumption is attributed in the Netherlands, at least in part, to the testing of meters and other devices by the Central Institution KIWA over the last 25 years, and the fact that only licenced plumbers may carry out installations.

In Denmark too, a high standard of materials and workmanship is reported to be secured by laws and regulations.

A new Building Act is to be enacted this year in Finland to regulate water and sewer fittings with an aim of reducing consumption. A domestic demand study is also in hand. Control is also exercised by throttling stop valves on consumer's services.

Bulgaria reports that the principal causes of loss are water taps and defective flushing valves. These are being replaced by more effective home produced designs which are also designed to give a better use of kinetic energy for washing purposes.

Basic byelaws covering the manufacture and installation of water fittings have existed in the U.K. for many years but in the absence of domestic metering and the

TABLE 2—SUMMARY OF WASTE DETECTION METHODS

Country and estimated losses	Waste meters systems	Sounding	Remarks
BULGARIA 15%	Automatic monitoring systems in use	Electronic acoustic apparatus in use	Domestic and trade meters deducted from station outputs to find waste levels
CZECHOSLOVAKIA 18,7% plus 5,9% used for backwashing, fire, sewer flushing, etc.	Waste meters installed in large systems. Bi-annual tests	1 electronic fault detector to each 1 km of water main. (Total mains system 31 314 km)	
DENMARK 5–50% (20–25% common)	Some large undertakings	Some large undertakings	Smaller use private firms for leak detection
FINLAND % loss not stated		When fairly large leakage noted or where systems known to be bad or old. Annually in Helsinki	
FRANCE Well-run systems 7–8%			Research into detection methods needed
ITALY 5–30%		Electronic acoustic methods increasingly used due to traffic noise, etc.	Computer remote control systems being installed, e.g. Turin 1974, Rome 1975
JAPAN High		Now being increased	
SWEDEN	By remote system sensors and deduction of minimum night flow. Mobile electromagnetic flow meters	For final pin-pointing	Radio-active tracers used in difficult cases
UNITED KINGDOM (also some Commonwealth and ex-colonies) 15–30% (U.K. Only)	Waste meter systems common	Programmed soundings carried out	Lack of domestic meters hampers waste control

divorcement of water byelaws from general building regulations, enforcement has presented difficulties. At the present time the inclusion of water regulations in general building regulations is being considered as part of a comprehensive Act known as the Health and Safety at Work, etc. Act.

There appear to be few regulations controlling fittings in Italy, and water for flushing WC's still often flows continuously!

Sweden reports efforts to develop installations that will use water more effectively including further research into the pattern of domestic demand.

2.3.14 CONTROL OF WASTE BY IMPROVED DESIGN OF DOMESTIC FITTINGS AND APPLIANCES. A considerable saving (up to 10%) could be achieved by the widespread adoption of 'dual flush' WC cisterns. In this cistern (see Figure 6), by a simple differential use of the handle, it can be made to flush either 4,5 litres for minor purposes or 9 litres for major use. Although costing very little more than a conventional cistern, present British charging methods do not encourage its use. However, adoption in countries where domestic metering is employed, bearing in mind the long-term unreliability of flushing valves, could offer appreciable savings.

Spray taps for hand washing also offer worthwhile economies. In Finland water use in air conditioning plant is being studied as a worthwhile area for economy.

Because of rapid urbanisation in Japan, pressure on water resources is causing water economy to be studied.

A disc for regulating flow from ordinary domestic taps has been developed, more efficient WC's and automatic urinal flushes controlled by the electro-conductivity of urine or by a push-button instead of intermittent cistern rinsing, are in use. Washing machines are being improved by savings on rinsing water of up to 50%.

Most U.K. water engineers still believe that domestic storage cisterns also considerably reduce demand.

Such cisterns are generally discouraged on the Continent for fear of contamination of the stored water. A study recently carried out, however, reveals the following: (Reference 3).

> From 426 properties water samples were taken and tested for E.coli and coliform bacteria. Coliform bacteria were only found in the mains supply of 5 installations and in the storage cistern water of 11 others. E.coli were not detected in these 16 installations. In one other installation, coliform and E.coli were detected in both the mains supply and in the storage cistern water.

The survey concluded:

> "Water drawn from domestic storage cisterns has not been found to deteriorate to any significant extent in respect of bacteriological quality.
> However, uncovered cisterns have shown some diminution in quality. Therefore, we believe that the results indicate that where a storage cistern is adequately covered and does not receive water other than that supplied by the water undertaker, back siphonage of stored water would not be harmful to health and would not present a hazard to the mains system."

In spite of this, recent legislation and reports in the United Kingdom are tending to favour, regrettably in the rapporteur's opinion, the use of pressurised hot water systems.

Satisfactory domestic storage systems offer the following advantages:

(i) low and constant pressure head on fittings and pipework, leading to less wear and wastage.

(ii) less wastage when water used from splashing, overfillings etc.

(iii) lower instantaneous demand on the distribution system than directly connected fittings as peak demand is 'cushioned' by the cistern. This applies to groups of up to 300 properties. Over this number the averaging effect of demand cancels out the effect.

(iv) smaller main systems or rather existing mains reticulation systems especially in rural areas, lasting longer without replacement or pressure boosting.

(v) reserve in the event of mains failure

Where cisterns are installed, and they are by no means universal, it is usual to take the tap in the kitchen from the mains supply before the cistern to ensure 'fresh' water for dietetic purposes.

The adoption of pressurised systems in the United Kingdom is largely to economise in the first cost of plumbing installations. It is likely that the overall effect, when the long-term aspects of waste and main sizing are considered, could well be more expensive and energy consuming.

Sweden is also reducing the quantity of water used in WC flushing cisterns and fixing feed devices to regulate excessive pressure to them. Hot and cold feed pipes should be insulated from each other to prevent waste and dish washers and washing machines approved. It is clear that in the past domestic water using appliances have been designed with little or no attention to water usage but here, as elsewhere, overdue attention is now being given to this factor.

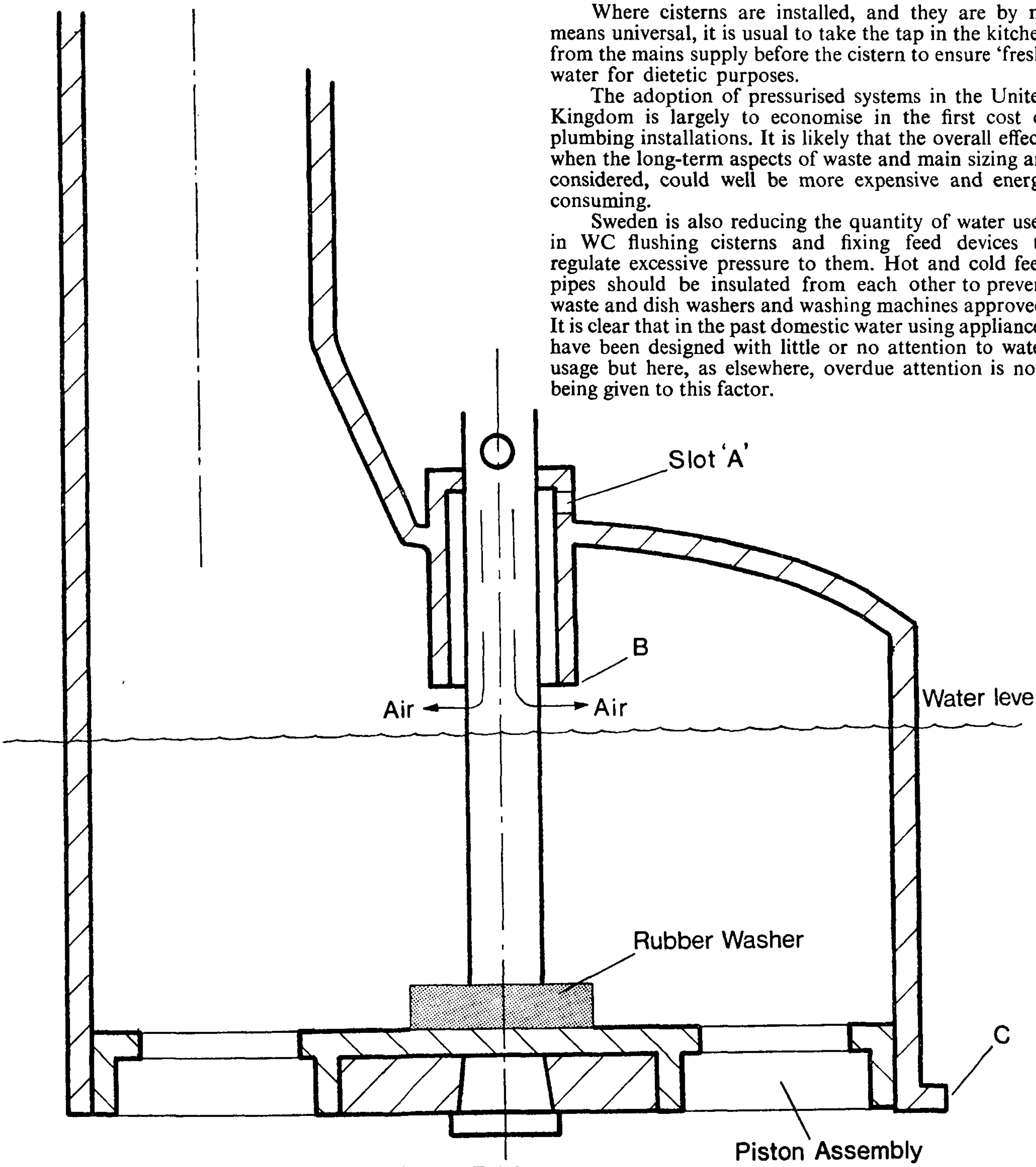


Figure 6—Twinflow syphon.

Syphon is provided having Slot 'A'.

FOR SHORT FLUSH (i.e. 4,5 litres).
Operating lever depressed and let go. Piston assembly returns to position shown and syphonage of water commences. Removal of water continues until water level falls below Point 'B'. At this point air enters system through Slot 'A' and cuts out syphonic action.

FOR LONG FLUSH (i.e. 9 litres).
Operating lever depressed and held. This maintains rubber washer of piston assembly against Face 'B', giving airtight seal. Syphon functions discharging until water level reaches Point 'C'.

2.4 Financial and Legislative Methods of Control

2.4.1 GENERAL CONSIDERATIONS. Whilst it is a truism that a supply of water is a basic human need, it can, nevertheless, especially if it is metered, be controlled via the price mechanism and, admitting as well that a basic supply for hygenic and culinary purposes should be provided at a reasonable price, further 'luxury' use such as garden watering, car washing, etc., should be supplied at a realistic cost reflecting its true economic value, especially where resources are scarce.

2.4.2 WATER METERS AS A METHOD OF CONTROL OF DEMAND. Industrial consumers are normally metered and charged for volume of water consumed; domestic consumers are increasingly being charged by meter, the United Kingdom and Ireland being the only countries reporting little or no domestic metering. Even in the United Kingdom, where domestic metering was not possible legally, this constraint has been removed by the Water Act 1973 S.32, which now confers the power to meter domestic consumers.

However, metering each consumer will not, in itself, restrain water demand unless the tariff structure is well designed and realistic. For example, if the price per cubic metre is very low, restraint will be small; also if there is some entitlement to a free or very cheap initial quantity, demand control will be blunted. An example of this is Italy where, until recently, although most consumers were metered, unit costs were very low based on revaluation of charges current in 1942! In 1974 this was changed by new legislation requiring charges to be related to economic costs but setting very low charges for essential domestic consumption (250 litres per family per day) with higher charges for consumption in excess of this.

It is also essential to meter all separate units of accommodation and not to bulk meter blocks of flats, etc.

The Netherlands report savings of up to 40% when consumers are metered for the first time, but not if metered communally (Figure 7). France also reports that in Paris, useful economies have been obtained where individual meters have been installed in blocks of flats, formerly bulk metered. Also in the past it was often the case that industrial consumers were placed on reducing block tariffs, i.e. the higher their consumption the lower the unit charge for water. It is now recognised that with the increasing cost of developing resources, these concessions should be removed and even reversed.

The adoption of increasing block tariffs has been considered in the Netherlands but they report the following difficulties:

(a) there is no relation between high consumption and undue consumption. An industry doing its utmost to economise but still having a high demand due to the nature of its product, should not be penalised.

(b) a *merger of two factories would increase* theoretical consumption and hence charges.

(c) charging a price to act as a brake on consumption is not necessarily a means of achieving efficient consumption. Water prices should cover costs for each category of consumer.

Nevertheless increasing block tariffs are being increasingly used, viz:

Tucson, Arizona, U.S.A. 1974
Israel since early 1960's
Cyprus, Jamaica, Mauritius, Nicaragua and proposed for Iran (Reference 4).

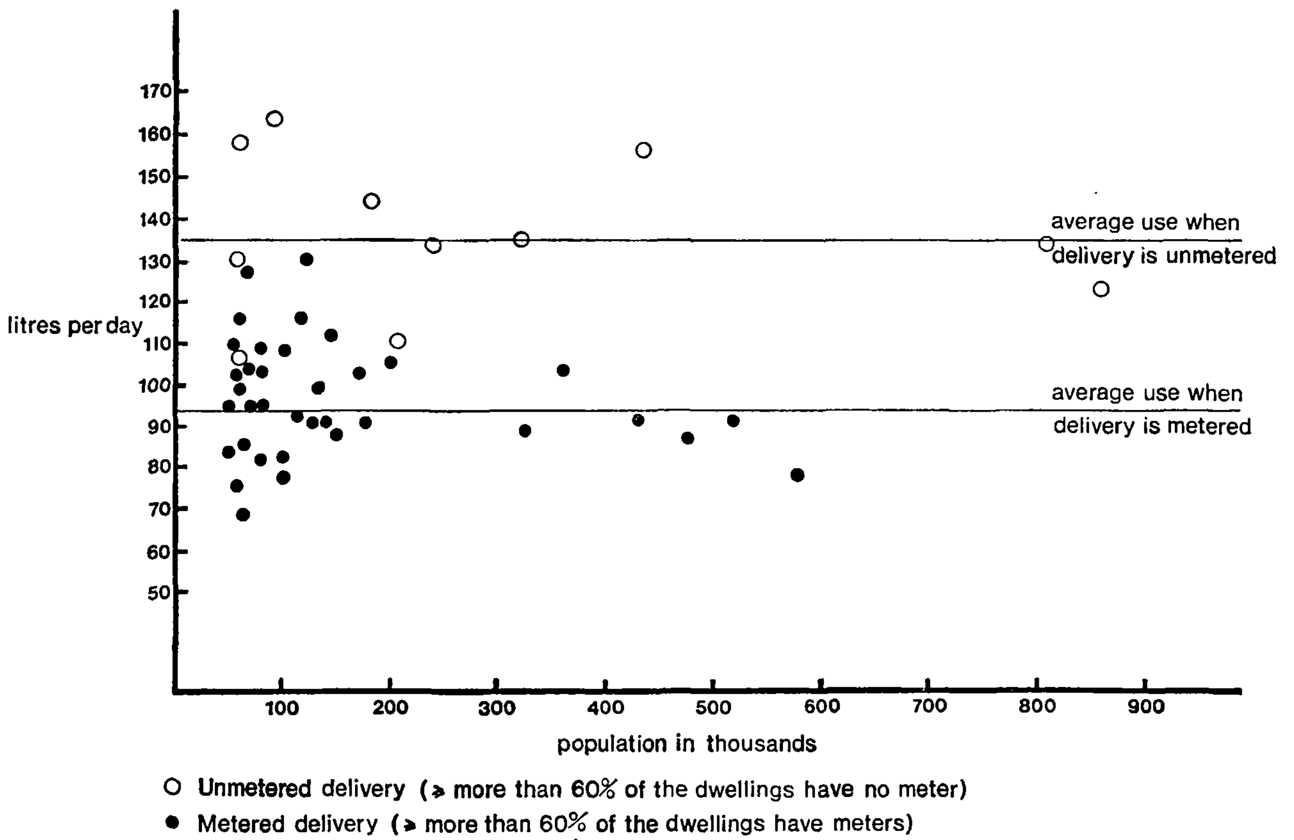


Figure 7—Water consumption per capita in areas of supply with a population above 50 000. Netherlands 1967.

An interesting and logical charging system is that adopted by Fairfax County Authority, U.S.A., reflecting the seasonal nature of costs. A standard rate per 1000 gallons is charged but in addition a surcharge of 3 times the standard charge is imposed for the summer quarter for use in excess of 1,3 times the previous winter quarter's use. This raises an economic charge for the heavy garden watering demand imposed when sources are approaching minimum yield.

In the U.S.S.R., meters are installed at all industrial plants, in public buildings and in individual or groups of residential houses. Water metering in every apartment is not practised. Water rates for domestic use are very low and subject to no increase. The average rate is about 0,04 rouble (2,5p) per m^3. A major part of the urban population, living in state-owned houses, pay their water bills according to fixed per capita consumption standards. If the actual consumption exceeds that calculated on the basis of these standards, then the difference is paid for by public service offices responsible for housing management or, as is the case in some large cities, by those responsible for maintenance of plumbing systems. This encourages these offices to keep plumbing fixtures in good repair and take steps to reduce leakage and undue flow of water. It is believed that installation of meters brings about a decrease in per capita consumption of water but there is no reliable data at hand to evaluate this decrease. Water rates in the U.S.S.R. are 2–5 times higher for industrial consumers than for domestic use.

It is stated in several reports that, in the past, price elasticity to metering has been low in the domestic field, i.e. increases in price have had little effect in reducing consumption but the effect of inflation, together with the increasing tendency to charge also for waste disposal through the same meter, is increasing the effect, especially in the industrial field. This is noted in the Netherlands, France and in Finland where the addition of the drainage charge has changed an annual 7% increase in consumption to a decrease.

Bulgaria reports savings of 30% when consumers are metered for the first time. In Sweden the annual increase in consumption has also been halted in newly metered areas and old wells re-opened for garden watering.

The disadvantages of metering must also be stated:

The cost of installing, reading, billing and servicing meters represents an appreciable capital and revenue cost, increasing charges without making more resources available. Reading difficulties occur if meters are installed internally due to consumers being out or unwilling to admit callers; they are more expensive to install externally and may freeze. Methods for remote reading exist via telephone lines, power cables, helicopters, etc., but the use of scarce resources required to install and maintain such systems must represent an unacceptable proportion of the cost of the service as a whole.

A final advantage of metering, however, is that it facilitates waste detection and the prompt repair of leaks on the consumer's side of the meter.

Water supply systems are generally designed with regard to three parameters: average annual demand, maximum day demand, and peak low demand. The annual, maximum-day and peak-hour pricing clearly affect each of these variables and the various parts of the system are designed with regard to the given temporal parameters. Table 3 (Reference 17) shows the importance of each of these system parts in relation to costs.

TABLE 3—ALLOCATION OF PLANT FACILITIES BY COSTS AND DESIGN HORIZON

Facility	Design Horizon	Part of Total Cost %
Distribution Main	Peak hour	6,9
Distribution storage and booster pumping	Peak hour	27,1
Transmission Mains	Maximum Day	31,1
Pumping	Maximum Day	14,3
Treatment	Maximum Day	3,9
Source (reservoir)	Annual Use (based on safe yield)	7,6
Unassignable	—	9,1

TABLE 4—OUTLINE OF BENEFITS AND ITEMS OF COST UNDER PEAK-LOAD PRICING

Category Affected	Benefits	Items of Cost
Municipal Utilities	Flexibility in planning investment horizon Reduction in plant-design parameters Reduction in physical inefficiency of overburdened facilities by increased use of what was formerly idle off-peak capacity Implicit interest savings due to deferment of investment Forestalling of the transaction cost of bond issues Less sewage plant capacity needed Less system waste due to consumers willingness to repair household leakages	Demand meters or adaptors
Domestic Consumers	Lower average water rates Greater choice in private use pattern and savings from off-peak prices Lower average sewage disposal rates Gain in consumers' available surplus during off-peak use Stable water pressure	Repairing of household leaks Reduction in surplus during peak use
Industrial and commercial consumers	Stable pressures and more reliable supply Lower average rates Consumers' surplus gain	Capacity needed for storing of water in off-peak period
Community (municipal or otherwise)	Overall welfare gains Less investment needed to be channelled into water resources Less of financial and inflationary burdens when interest rates are high	

It is seen that the design parameters for approximately 85% of system costs are sensitive to maximum-day and peak-hour consumption.

A recent paper has shown that the ratios of the average summer, maximum-day and peak-hour demands to the average annual demands were 2,49; 8,27 and 30,1 respectively for 41 study areas throughout the U.S.A.

An analysis of the costs and benefit of implementing demand metering (with a rate schedule related to the amplitude of the variables discussed above) may be viewed in relation to supplier and consumer. Table 4 (Reference 17) shows some non-quantifiable and quantifiable benefits and costs.

Feldman (Reference 17) makes a case for adapting municipal water supplies to a responsive rate variation. Two recent patents, at present at the R & D stage, describe such a system. It includes a meter-measuring quantity, a pressure-measuring device, and a register. The outputs of the pressure meter and quantity meter are correlated to give a numerical readout at the register that is inversely proportioned to the pressure of the water in the service line and directly proportioned to the quantity of flow. Thus, the demand on the water system is reflected in the quantity registered. A demand-metering principle based on pressure-adjustment factors could efficiently keep track of the demands placed upon the system by consumers. The readout could reflect the current short-run marginal costs of the system, whether off-peak or on-peak.

2.4.3 METHODS OF CONTROL BY LEGISLATION. These may be divided into two categories:

(i) control of fittings and installations by regulations dealt with in para. 2.3.14.

(ii) restrictions on usage in drought, accident, etc.

Most countries, although aiming to meet demands, are prepared to impose restrictions in times of drought, frost or major pollution. In the Netherlands, consumers who ignore restrictions on hosepipes in drought may be cut off. In France, voluntary restrictions are not considered very effective. They are also unpopular in Denmark and large undertakings try to avoid them; nevertheless they are quite common on garden watering and car washing in the summer.

In Bulgaria they are also unpopular but necessary in hot weather and sometimes involve cutting off zones for periods of time.

The severe drought of 1975 in the United Kingdom has resulted in restrictions in some areas where development of new resources tends to lag behind demand due to environmental opposition to new resource schemes.

1972 saw a severe drought in Japan with widespread restrictions. The price of water was increased and large consumers were severely affected but were very co-operative.

Annual restrictions are necessary in some parts of Italy but always cause controversy.

In Ireland restraints are imposed in some summers but consumer reaction is becoming increasingly adverse.

It is becoming clear that with the rising cost of water, restrictions are being increasingly resented. This creates a difficult problem as to meet demand in any drought would impose unacceptable costs to the consumer in each year and restrictions in those rare years are the best economic solution but this needs to be demonstrated by publicity.

It is not possible to lay down hard and fast rules as each country has different climates, resource and demand problems but, in temperate climates and developed countries, restrictions once in say 10 years would seem to to be a reasonable imposition on the consumer to prevent over investment in rarely used resources.

2.5 Education and Publicity as Methods of Control

2.5.1 GENERAL CONSIDERATIONS. In the past, water supply undertakings have shunned publicity, considering themselves as benefactors of mankind and content to be judged by the service they give. They suddenly discovered, as was pointed out by Abel Wohlman in his inaugural address at the IWSA New York Congress in 1972, that in developing water resources they were suddenly being considered as predators.

Clearly the public needed education and publicity in the essential nature of the service and the need to economise in what is becoming in some areas a scarce resource.

2.5.2 EDUCATION. The recently developed public awareness of the harm done to the environment by uncontrolled exploitation has had the effect of mobilising public opposition to the development of new water resources in spite of their basic nature. It is desirable to channel this awareness into economy of the use of water to minimise its demand on natural resources and energy. It must begin in the schools with a co-ordinated programme through a child's school career, not forgetting the 'career in water' aspects, and go on to adult visits to Works, advertising and the use of the mass media, as well as great care in the development and operations of works to include public participation in the planning stages.

Water recreation is an aid in the public relations field and public opinion as to the price they are prepared to pay for absolute reliability in rare droughts needs to be canvassed.

However, it must be borne in mind that an opinion received just after a water bill has been paid may be quite different to that obtained when water is restricted in hot weather during the rare drought!

In the United Kingdom, for example, there is a statutory requirement to give a constant supply at a reasonable pressure, and less than absolute reliability would be difficult to plan for in the face of this statutory obligation, even if it is not always achieved in practice.

Whilst the public, in general, respond well to appeals for economy in drought situations, where rain is a frequent occurrence it is difficult to persuade the public that an ample supply cannot readily be made available.

The very much higher charges now made for water also mitigate against the more tolerant attitude adopted in the past when water was cheap and locally administered.

Countries with temperate climates without extremes of heat or cold have an obstacle to efficient operation as authorities tend to be unprepared for the rare drought or severe winter and it is difficult to justify resources to deal with these extremes when they happen so rarely and their influence on reliability is therefore small. For example, although in the United Kingdom there were considerable water losses in the very severe winter of 1962/63, since then winters have been exceptionally mild and these losses are, therefore, over a period quite negligible.

2.5.3 PUBLICITY. Education and publicity obviously overlap but there has been a tendency to overlook the long-term benefits of education but to institute and have publicity campaigns when there is an exceptional drought or failure to keep pace with demand. Examples of public relations campaigns are those carried out in Japan, Ireland and in the east of the United Kingdom in recent years.

As described in para. 2.4.3, Japan suffered a serious drought in 1972. As well as imposing restraints and increasing prices, a large-scale publicity campaign was mounted on the streets by pamphlet distribution and through newspapers, radio, TV and weekly magazines.

Primary school teachers were provided with textbooks on water to seek their co-operation. So successful was the campaign that subsequently water demand has not increased but has tended to decline.

In 1974 a similar campaign was started in Ireland by one authority and run by an advertising agency. The primary target was domestic consumers in Dublin city and county, and in those areas which draw water from the four reservoirs supplying this area; the secondary target was industrial consumers. The response was excellent; normal growth of consumption was actually reversed and although sponsored by one authority it proved of benefit to others as the major media have national coverage. The initial budget was £30 000 and has been repeated this year. Industry responded so well that a considerable decrease in water revenue resulted.

The industrial response was partly due to an increased awareness by people in industry of the water shortage resulting both from the consumer advertising campaign and direct contact with corporation officials.

Following a series of dry winters in which replenishment of the predominantly underground resources was reduced by 50% of normal, a group of undertakings in the east of the United Kingdom collaborated in a publicity campaign run by an agency, the shared cost of which was £27 000 and which lasted for 4 months. (See Figure 8). It proved difficult to quantify the exact savings made; one undertaking estimated savings at 3–5%;

Figure 8—Water Economies Campaign Stickers.

another in excess of this, but the two others involved still had to impose restrictions so that they incurred a loss of revenue as well as the campaign expenses. However, the public appreciated positive advice on ways of saving water rather than just the imposition of restrictions.

In Denmark, the Ministry of the Environment is about to start a campaign to educate the public in the problems of water collection, storage, treatment and distribution, to effect an economy in consumption. A similar study is being considered in Finland.

It appears that well-planned campaigns can be very successful and more should be done in this direction to 'sell' water economy via the mass media now available.

2.6 Climatic Conditions and their Effect on Demand

2.6.1 *Cold Climates.* This topic was dealt with at the 1972 IWSA Conference at New York and it is not proposed to duplicate the information here. (Reference 11).

2.6.2 *Arid Climates.* Author's Apology: It has been possible to obtain only one report from an arid country (Ghana), therefore, this section has had, perforce, to be prepared mainly from what current literature is available.

2.6.3 *General Considerations.* Needless to say, most of the factors affecting water demand in temperate climates are present in arid areas but are modified by climatic conditions and the wealth, or lack of it, of the communities served.

2.6.4 Water demand in developed countries having arid zones, e.g. the United States, Australia, parts of South Africa, etc., show very heavy per capita consumption to deal with high garden use, increased hygienic use and water for air conditioning units, as well as increased use for irrigation.

In 1965, for example, in the United States (Reference 12), 85% of the water consumed took place in the relatively arid seventeen Western States, where only 25% of the country's water resources are to be found. This accentuates the problem of meeting demand and has led to conservation measures, particularly in thermal power stations. Here, re-cycling through cooling towers and ponds has led to repeated re-use.

The supply and demand relationship in the 17 semi-arid States of the U.S.A. approaches the critical. (See Figure 9). Fresh water withdrawals in 1965 in the arid states amounted to 590 million m^3/d, very close to the 680 million m^3/d estimated dependable supply. To sustain withdrawals in excess of dependable supply, an increasing amount of re-use is necessary and careful monitoring of the standards of re-treatment are necessary. Industries using cooling water with re-use will concentrate dissolved minerals in the plants' effluent and raise the temperature of the discharged water leading to possible eutrophication problems. In some areas, notably the Upper Missouri and Upper Arkansas, supply is already inadequate and chronic water shortages are experienced.

A clear relationship between precipitation and per capita demand is shown in Figure 10 relating to the U.S.A. (Reference 13).

In the urban areas of Australia, water usage for industry, garden watering, etc., seems to follow the U.S. pattern with the following exceptions (Reference 14).

Per capita use in Australia is 80% of the U.S.A.'s but rising at a faster rate, and is expected to exceed that of the U.S.A. by the end of the century; garden watering equates to potential sub-soil need compared with only 60% in the U.S.A.

Demands in South Africa, an arid region, led Windhoek, the capital of South-West Africa, to consider the reclamation of sewage effluents for domestic use a decade ago. A waste water reclamation plant has been constructed to meet certain needs, especially to tide over the period while a new dam was under construction. Available for reclamation was a water source from nine maturation ponds giving tertiary treatment to the conventionally treated sewage from the Gammams sewage works. Ammonia concentration is reduced in summer months due to effective algal action in the ponds; the algae are subsequently removed by flotation in the reclamation plant. There is also settlement, sand and carbon filtration, followed by break-point chlorination. Due to cost, break-point chlorination was only accomplished in the summer months and the plant was shutdown in the winter. Although the reclaimed water eventually mixed with impounded water, the two systems were kept separate properly to control and monitor the reclamation plant (Figure 11).

One of the inherent problems of such a process is the resistance of entero viruses and the infectious hepatitis virus to some normal water and sterilization processes, and special studies were made (References 15A, B & C).

The plant ran from 1968 to February 1971 during each summer, until the heavy rains and the completion of the Von Bach dam enabled the plant to be stood down.

2.6.5 Water demand in the developing countries is often unknown as the potential demand is suppressed by poverty, lack of resources, etc.

The size of the problem can be judged by the modest goals of the United Nations Second Development Decade (1971–80) programme (Reference 16).

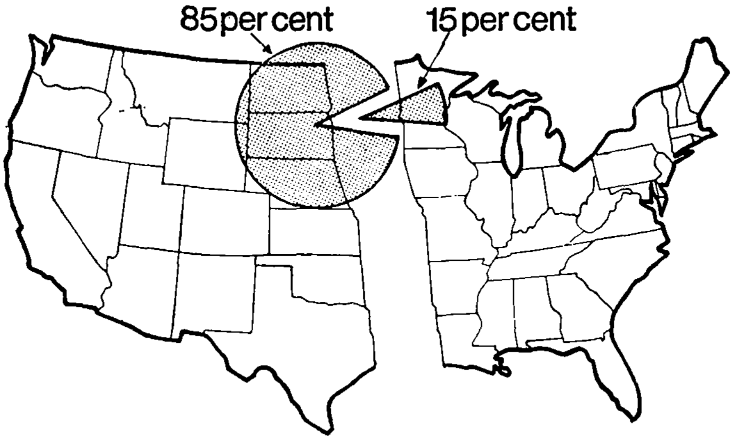
In 1970, it was proposed that a determined effort should be made during the decade to increase the percentage of urban dwellers with piped water into their homes from 25–40%; of townspeople receiving public standpipe supplies from 26–60%; and rural inhabitants supplied with safe water from less than 10% to at least 20%. This was expected in 1970 to cost $9{,}1 \times 10^3$ million U.S. dollars, one-sixth of which would be needed for the rural development. Inflation has by now probably doubled this sum!

It is clear that no sum approaching this is likely to be made available. However, everyone has a water supply of some kind, however insufficient or polluted, and the normal problem is to purify, store and distribute water that already exists at or near the area to be served, and to ensure its controlled use and maintenance thereafter, for a community supply, if neglected, can increase or introduce hazards that did not exist before. Lack of waste disposal facilities and lack of knowledge of the hazards of contamination therefrom also increases the danger when larger supplies of water become available.

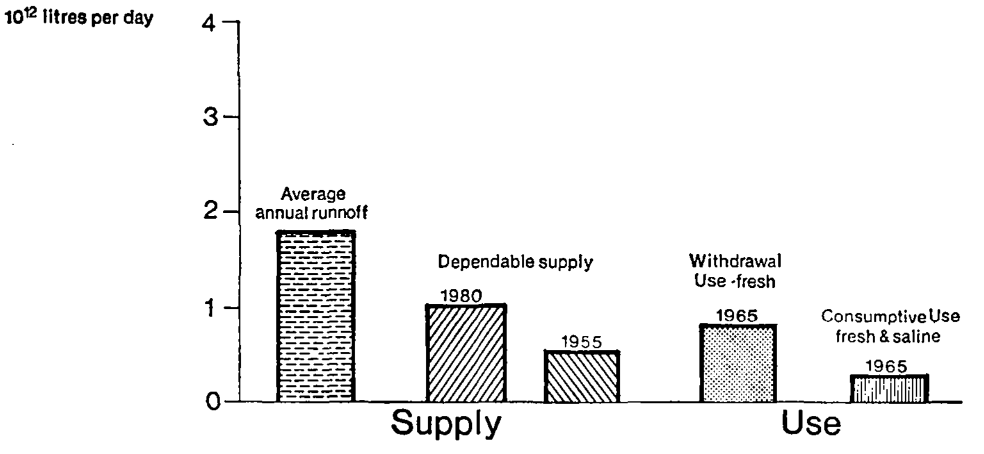
Water must be acceptable to the consumer and not too far away, otherwise there may be a reversion to the nearer, more acceptable, polluted source.

The provision of an enlarged, safe, water supply to a community is the 'break-through' point for community development, as so much depends on water. Primitive communities will derive benefit from animal and vegetable plot watering; it may provide a potential for the establishment of local crafts such as dyeing, pottery, etc. It will enable clinics, schools, laundries and public baths to be provided. The list is endless and the demand starting with what the wife or wives can carry home per day on their heads will end only when the villager is washing the second family car!

Whilst the provision of a safe water supply is the first step in eliminating water-borne diseases such as cholera, increasing the quantity of domestic water may,



a. Water consumption in 1965 by the 17 Western States and 31 Eastern States



b. Water supply and demand in the 17 Western States

c. Supply compared with demand in Western States

Region	Estimated dependable supply, 1980[1]	Total withdrawal 1965	Water consumed 1965	Fresh surface water withdrawn 1965[2]
Missouri	150	95	46	75
Arkensas	91	47	26	23
Western Gulf	91	163[3]	64	45
Colorado	68	77	38	55
Great Basin	41	31	17	24
South Pacific	127	173[4]	68	64
Pacific Northwest	318	132	45	109

1 Woodward (1957) p 49
2 Including some minor inter-regional diversions
3 Includes 21×10^9 litres per day saline water
4 Includes 50×10^9 litres per day saline water

Figure 9—Water Consumption in drier Western U.S.A.

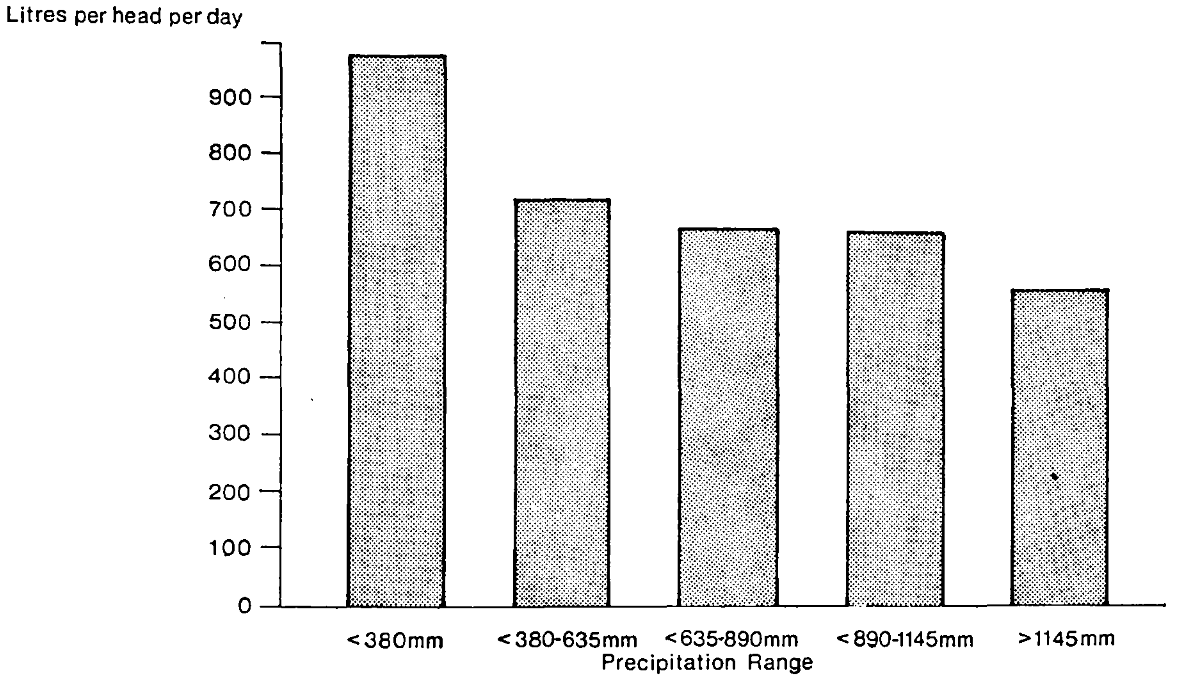


Figure 10—Per capita water consumption as related to precipitation in U.S.A.

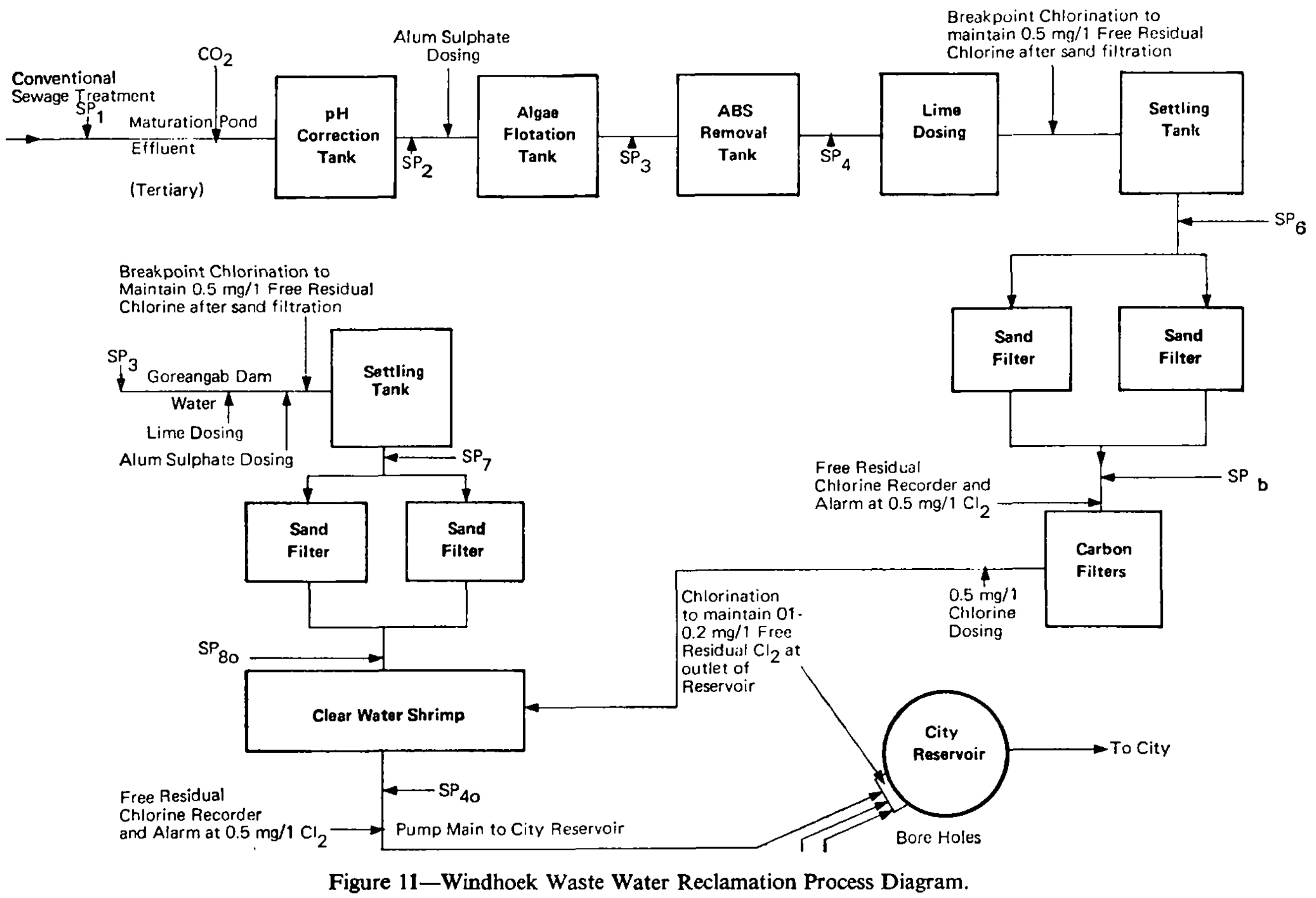


Figure 11—Windhoek Waste Water Reclamation Process Diagram.

in fact, create health problems which were not present before. Bilharziasis may breed in undrained pools of waste water; a muddy area around a public standpipe may help to spread hookworm. This indicates that the mere provision of the supply is not enough—arrangements for its maintenance and the education of the community in its hygienic use are also essential.

The overcrowded shanty developments on the fringes of large towns in developing countries often overload the local supply systems and epidemics are caused by reversion to unsafe sources. Another problem is the lack of data when developing supplies, leading to over or under estimates for potential yields. There is also a tendency for priority to be given with the limited funds available to urban areas, even though these may not represent the great part of the population. Means must be found to focus attention on the less interesting, more repetitive rural schemes, to ensure that they receive a fair allocation of funds.

In the Middle East where very arid areas are now related to large resources of energy in the form of oil and great wealth from the oil revenues, it is to be expected that de-salination schemes will increase in importance as will new conjunctive use schemes with de-salination for topping up.

Like most towns in developing countries, El Obeid in Western Sudan charges for water, and this must lead to reduction of undue consumption. Unfortunately, the poorest people who have no house connection pay most for water: in the summer months, vendors are able to charge 2 piastres for a 4 gallon tinfull, equivalent to £7,63 per thousand gallons or £1,68 per m^3. In villages, the price is still higher.

It is very clear that demand drops substantially during and after the short rainy season. For example, at Khor Teggat water yard, where there are 4 metered standpipes supplied from a borehole, the daily sales during and after last year's rain averaged only $24 m^3/day$ compared with $168 m^3/day$ during the rest of the year. The demand for water appears to be more influenced by availability of water from other sources than by ambient temperatures, although highest temperatures (above 50°C) come just before the rains when most other sources have dried up.

Because most of the distribution system is old and inadequate, some areas have no flow at some time each day throughout the year. In hot months, before the rains, some outlets only deliver water during the night. People grumble but at least they are better off than in the villages where there may be virtually no water at all.

3 Re-use of Waste Water

3.1 General Considerations

Economy in the use of water for industry can be affected in several ways:

(a) water supplied to a potable standard for the public supply can be re-circulated and re-treated for less high quality uses.

(b) dual supply systems can be constructed so that water of a lower quality is made available at a lower cost.

(c) sewage effluent can be used with or without further treatment for industrial purposes or for artificial re-charge of aquifers.

These uses are being pressed by lack of fresh water resources in such countries as Israel and by environmental considerations in other areas.

The increase in charging schemes where waste water, often not charged for at all previously, is now being charged for its re-purification through the water supply meter has brought a rapid increase in industrial re-use. The waste water treatment charge is sometimes greater than the water supply charge already, and likely to equal it in the Netherlands by 1976.

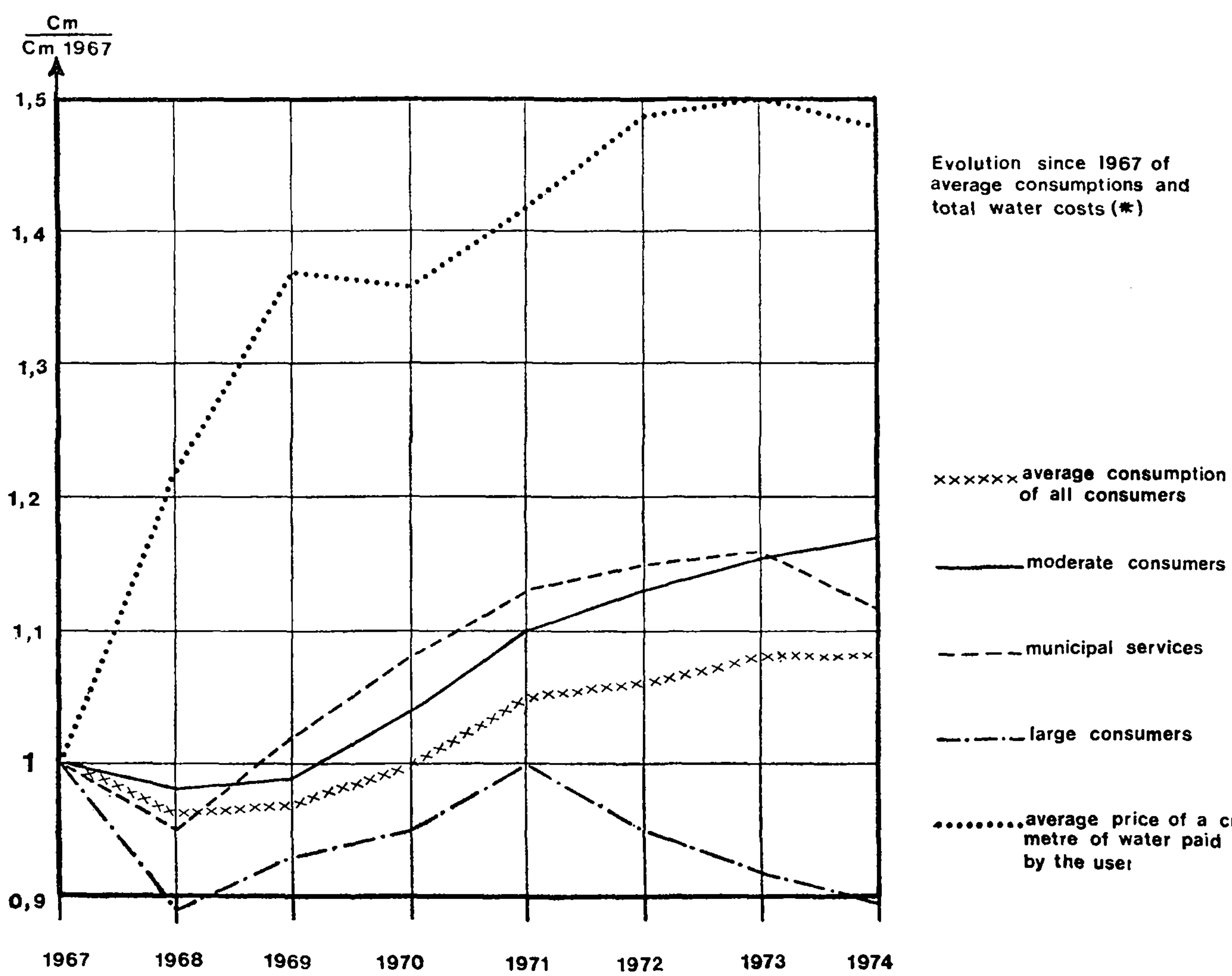


Figure 12—The effect of water cost on water demand.

3.2 Re-use of Potable Quality Water Supplied to Industry

In France the increasing cost of potable water supplied to industry shows a co-relation to a reduction in industrial demand after a time-lag which appears to be the time taken to install re-circulation and re-use equipment (Figure 12). Numerous examples of such re-use have been quoted and within the limits of the paper, only a few may be given, as follows:

> In Denmark it is forbidden to use potable water directly for cooling or mechanical purposes, so cooling water is always re-cycled. In Copenhagen, all large water consumers have been surveyed and advised on how re-cycling can best be carried out.
>
> As a contract, in the United Kingdom only a few years ago, due to low charges for potable water and little or no trade effluent charges, water was often used only once to cool a product a few degrees. With the advent of higher charges this mis-use no-longer occurs. However, in the Paris region of France, where water is relatively plentiful, in spite of its economic advantages, re-use is not encouraged on hygienic grounds.

In countries with inadequate rainfall, use is made of sewage effluents for irrigation of crops. In South and West Africa about 25% of sewage effluents are used for industrial purposes and at least 50% for crop irrigation. In Israel sewage effluents from the Tel-Aviv area is used to irrigate the Negev, and in Israel water re-use is as near 100% as is physically possible. The main problems reported are the public health aspects, the presence of boron (and other elements) in the effluent and the possibility of accumulation of soluble solids in the soil.

Japan is re-cycling increasing quantities of waste water—from 28% in 1961, to 58% in 1972, and it is planned to reach 70% by 1975. Research is aimed at zero discharge plants and some are stated to be under construction—a considerable achievement. In Bulgaria, supplies of potable water are scarce and efforts to re-cycle water are increasing all the time.

Typical percentages of re-cycling are:

> The oil industry 89%, using oil traps, trickling cooling towers and stabilisation with natrium metaphosphate acid and chlorine diluted with 5% of fresh water. In cellulose based paper mills 40% is re-used by means of vertical sedimentation tanks and filtration. At one foundry, re-use now reaches 78,5%.

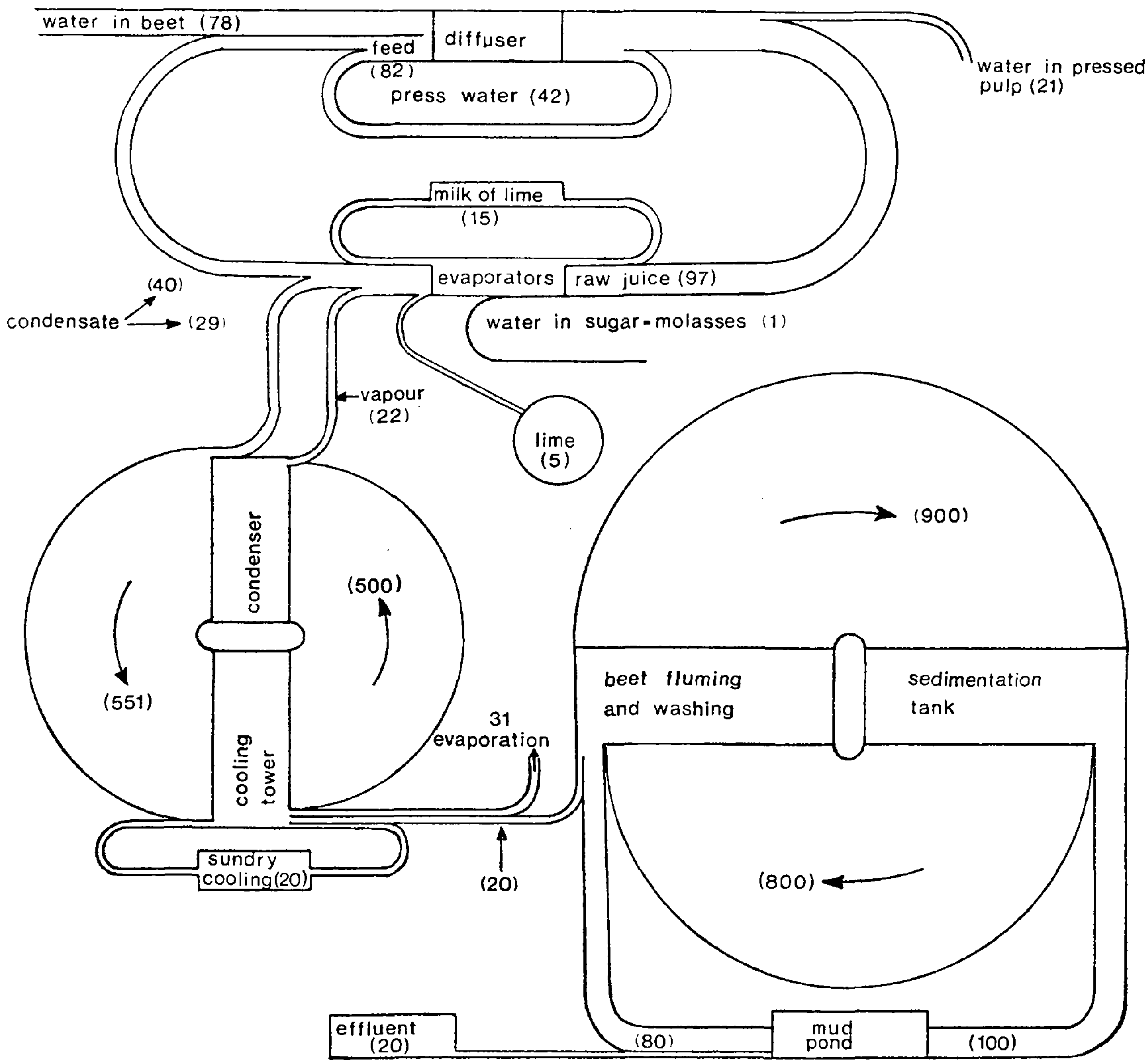


Figure 13—Water flow with complete re-use — Sugar Beet processing.

Saving in cost on re-use compared with potable water, varies from 30% to 70% less.

The industries of the United Kingdom, as elsewhere, under the twin pressures of higher charges for potable water and for trade effluent treatment charges, are increasingly studying re-use.

A striking example is in sugar production from sugar beet, where a total usage of 1170 tonnes of water per 100 tonnes of beet has been reduced to 20 tonnes of water, a saving of over 27 300 m^3 of water per day, in a factory slicing 2440 tonnes of beet. Figure 13 shows the flow diagram for water flow in a sugar beet plant for complete re-use (Reference 5).

In Essex, in the United Kingdom, some direct re-use of sewage effluents occurs for spray irrigation but only of crops intended to be cooked before consumption.

Attention is now being paid to re-use in Italy where apart from the chemical and oil industries, little re-use was made in the past. Once again, it is the impact of higher charges that is forcing the issue.

In the Netherlands the situation is similar and reductions so far achieved include the halving of the consumption of aluminium works from 1×10^6 to 5×10^5 m^3 by re-circulation and re-purification. Computer control of a brewery process has saved 3×10^6 m^3/day and an even greater proportional reduction by a canning factory of from 3,7 to only $1{,}5 \times 10^6$ m^3/day.

Water undertakers themselves are not above criticism. At Eindhoven, backwashing of filters can be reduced from 2,2% to 0,2% of throughput by re-treatment and at Midden Nederland, air conditioning use has been reduced from 200 000 m^3/year to 1500 m^3/year and by the installation of a cooling tower.

One defect of Dutch legislation is that while the use of ground water by water undertakers is controlled, industrial consumers can use such sources as they wish. This results in surface water having to be used for potable supply while industry uses good quality ground water for cooling purposes.

It appears that potable water is, at last, due to its increasing cost, being recognised as the valuable resource that it is and real efforts made to re-use it in industry. This trend can be expected to continue especially as more countries decide to charge for clean and waste water through the same meter.

In the U.S.S.R., the Academy of Municipal Economy and other organisations have recently carried out research work on the use of treated municipal waste water for industrial water supply. Conventional secondary treatment, rapid filtration and disinfection were employed. As a result of this work a number of industrial water supply systems have been constructed and are now in operation using tertiary treated municipal waste water. One such system has been constructed in Moscow to supply non-potable water to a power station and a group of district plants. The present capacity of the system is 200 000 m^3/d while its ultimate capacity will be 600 000 m^3/d.

3.3.1 DUAL SUPPLY SYSTEMS—GENERAL CONSIDERATIONS. To the layman, the use of dual water supply systems where a lower quality water is available at a cheaper price for say garden watering and WC flushing, seems to be a simple solution to resource problems but there are major drawbacks.

To duplicate a large mains reticulation system would be completely uneconomic as the installation, maintenance and renewal of mains and services as the major cost of any waterworks, and the savings in treatment costs would never service the capital required for a duplicated mains system.

A second major problem is the ever present likelihood of cross connection between the potable and the secondary quality supply. This can occur either by mistakes by the water undertaking—in the author's experience attempts to tap gas mains and cast iron sewers is not unknown—but more likely inside factories where both supplies are in use, to which must be added the normal hazards of back siphonage from processes not correctly isolated from the incoming supply. However, subject to safeguards, such systems can be used where factories, etc., are situated near to sources of supply or are prepared to finance the necessary mains lines.

In some areas the secondary supply may even be of higher quality than the potable supply, e.g. de-mineralised where advanced technology processes are involved.

3.3.2 EXAMPLES OF DUAL SUPPLY USE. In Denmark, the economic aspects mentioned above limit dual supply systems to large industrial firms and electrical power stations in coastal areas.

France reports the installation of a dual system to supply a concentrated industrial zone in the Basse-Seine, and another where low quality water is used to transport cinder hydraulically from a power station.

Because of its special problems, Japan is pressing ahead with dual supplies. Already 13 systems exist, supplying 54 000 m^3/day. For office buildings, urban re-developments and for municipal housing, plans are in hand for a dual supply to be installed for WC flushing from which it is hoped to save potable water consumption of 40–60% in the case of offices, and 20–30% in housing areas.

Such systems are also used in Bulgaria, where the potable supply is for domestic use and fire fighting, and the lower quality supply for industry.

In the United Kingdom, dual supplies are in use to isolated industrial complexes, e.g. to the Fawley oil refinery from the West Hampshire Water Company; hard spring supplies to breweries, etc., but on hygienic grounds they have not been widely used in cities as duplicate systems. An exception is Warrington in Lancashire, where a separate industrial water supply is provided. The source is the heavily industrialised and polluted River Mersey. Some 34 Ml/d, generally of poorer quality than many sewage work's effluents, are treated and sold to a number of adjoining industrial users for use in wire drawing, condensor cooling and board making. The price is considerably less than that of the potable supply and it has been used for over 40 years without corrosion problems related to ferrous metals, but the high ammonia content has been held responsible for corrosion of brass and copper fittings.

A useful study of dual supplies was made as part of the extensive River Trent Research Programme designed to investigate methods of clearing up the grossly polluted river to make it acceptable as a source of water supply for various purposes. The study involved a survey of the potential for non-potable supplies in the Trent region and an outline design and economic appraisal of a dual supply system (Reference 6). The conclusions of the study were that potentially a dual supply could substitute for 600 000 m^3/day of present potable demand representing about 30% of the potential potable supply deficiency in the year 2001. Cost savings might be of the order of 18% in favourable cases. To reduce pollution risks via cross connection, the use of a secondary supply to augment a canal was considered, from which industrial supplies will be drawn.

In the Netherlands, dual supply systems are treated with caution for the reasons given in paragraph 3.3.1.

However, a special case exists with Rhine water where the Water Company, Rijn-Kennemerland, partially treats Rhine water and then transports it to other companies, such as Amsterdam and North Holland, who complete its treatment before distribution as potable water.

In 1973 the Rhine Company supplied 45 million m^3 of the partially treated water to steel works and a paper mill; 2 million m^3 was also supplied to Amsterdam Docks. In the Rotterdam Dock Area a secondary supply of very high quality (demineralised) water amounting to 2 million m^3/year is supplied.

To sum up, it appears that most Western European countries regard dual supplies on an extensive scale with caution because of their hygienic aspects, and the heavy costs of the second distribution system, but they are being actively promoted in Japan and Bulgaria to meet resource problems.

3.4 Use of Sewage Effluent (with or without further treatment) for Industrial use or Irrigation

3.4.1 GENERAL CONSIDERATIONS. Sewage effluent without further treatment can often be used as a conveyance medium, for cooling and spray irrigation and with further treatment, either for industrial use or for human consumption. There are obvious problems for potable use, especially the unknown long-term effects of complex chemical residues now present in nearly all effluents, and attempts to use effluent without considerable dilution with 'clean' river or underground water, must be viewed with concern given the present state of knowledge.

3.4.2 EXAMPLES OF EFFLUENT RE-USE. In Denmark such usage occurs by the discharge of effluents into lakes which are also used for water supply.

In Le Havre in France, a study is being carried out into the use of effluent in a district heating scheme.

In Tokyo, a sewage treatment works is under construction using ozone and activated carbon after coagulation and rapid filtration to promote the complete recovery of effluent to potable standard. However, the reservations implied in 3.4.1 apply to the effluent from the plant if intended for potable supply. The capacity of the plant will be 50 000 m^3/day.

Much investigation is carried out in Bulgaria using sewage effluents diluted with river water and after treatment its organic constituents have been found to be valuable in increasing the yield per hectare of maize, beetroot, alfalfa, poplars, etc. The purifying effect of such irrigation is also significant, reducing suspended solids by 67%, for example.

In the United Kingdom many examples can be quoted. A sewage treatment works at Bristol serving a population of 510 000, gives partial treatment to the whole 174 Ml/d input, and 32 Ml/d is further processed for industrial re-use and in an adjoining refuse destructor. Sludge digestion is used to produce methane which generates electricity in a 5000 kilowatt station powering the works and the refuse destructor, as well as cooking meals for 200 old people a day.

Sewage effluent is widely used for transporting chalk slurries at cement works and in the Severn Trent region, 300 000 m^3/annum is re-used for industrial use, cooling and spray irrigation.

Digested sewage sludge in solid or liquid form is widely used in the United Kingdom as a soil conditioner and is widely sought after and paid for by farmers in some areas.

An interesting irrigation scheme in an arid climate is that employed in Libya for the towns of Tripoli, Benghazi, Tobruk, Derna, Miswata and Sebha, where direct re-use is employed. After conventional biological treatment followed by gravity sand filters to a 10:10 standard and chlorination, the final effluent is pumped to open reservoirs of 81 000 m^3 total capacity at the irrigation area. Irrigation is delivered to 100 farms of 6 hectares area with overhead spray nozzles. The area totals 750 hectares, to be expanded to 3000 hectares when sufficient effluent becomes available. The soil was barren—85–90% sand, 3% clay with some loam and silt with a very low organic content, and for the first 1½–2 years soil improving crops (beans, clover, etc.) were ploughed in with dried sludge. The farms are now producing valuable crops of potatoes, onions, tomatoes, lettuce, carrots, beans, egg plants, melons, peaches, plums, pears, wheat, barley and alfalfa (Reference 8). It would appear that in such cases, where living standards are fairly low, the principal hazards will be the risk of re-cycling endemic disease rather than the long-term effects of complex chemicals. The endemic ring-worm in Tristan Da Cunha, due to the use of human untreated excrement to grow potatoes, is well-known, although in Libya steps have been taken to minimise such likely effects.

In Singapore, an industrial estate is supplied with water reclaimed from sewage effluent after sand filtration.

At Texel, on the north coast of the Netherlands, a central sewage plant is being considered to re-use the effluent as drinking water after hyper-filtration or after artificial infiltration, and admixing with dune water and de-salinated water. In the author's opinion the palatability of the resulting water may leave something to be desired but no doubt it can be disguised with plenty of excellent Dutch gin! Such a solution can only be considered where there is such a large increase in population during the 3 months' tourist season.

3.5 Use of Waste Water for Artificial Re-Charge

3.5.1 GENERAL CONSIDERATIONS. This use of waste water already exists in various places and as the pressure on resources increases and techniques of purification and re-charge improve, it can be expected to increase still further.

Reservations must, however, once again be expressed concerning particular chemicals which will not be removed by passage through underground strata and the possibility of fissures being blocked by the deposition of solids from the effluents during their passage.

3.5.2 EXISTING AND PROPOSED RE-CHARGE USES. The growth of population in Nassau County on Long Island, New York, over the past 20 years has greatly increased the demand for water and, at the same time, urbanisation has reduced the recharge capacity of the local aquifer, and this problem is accentuated by the threat of salt water encroachment into the aquifer.

A series of artificial recharge experiments has been carried out to examine the feasibility of injecting reclaimed water into a network of wells. 2,7 Ml/d of recharge water is obtained from tertiary treated effluent from an activated sludge sewage treatment plant. The tertiary treatment involves coagulation and sedimentation, two-state filtration through a dual-media sand-anthracite filter, and a series of four activated carbon columns, and final chlorination. This is injected through a gravel-packed well and 14 observation wells were arranged, four to lap the injection stratum, the rest to tap beds either above or below the recharge zone. The experiments showed that the chemistries of the injected reclaimed water and the native aquifers were, in general, compatible. A significant water-quality problem was the loss of specific capacity of the recharge well during prolonged injection (Reference 10).

The use of the Rhine to re-charge depleted fresh water in the dunes of Amsterdam, the Hague, etc., in Holland, is a notable example of re-charge on a large scale.

In the United Kingdom, cooling of 275 KV power cables is carried out by abstracting water from chalk boreholes, cooling each cable at the rate of 68,3 m^3 hour; the water having passed through cooling jackets adjacent to the cables is returned to the chalk via re-charge boreholes.

Chalk water is similarly re-charged at Welwyn Garden City and Bromley after use for cooling purposes.

As a further part of the Trent Research Programme (referred to in 3.3.2), a study was carried out on the re-charge of the bunter sandstone using sewage effluent (Reference 7). Both basin and irrigation re-charge was studied, the effluent being settled and aerated prior to re-charge and activated carbon adsorption and disinfection after re-abstraction. Costs for a scheme yielding 3,4 m^3/sec. were calculated but the process appeared to be practicable.

A similar project is currently in hand to study the re-charge characteristics of superficial deposits in improving the quality of re-charged water. Alluvial deposits in hydraulic continuity with a river are to be used and water quality monitored. Initial studies in the Taff Valley in Wales will be followed by others in Hertfordshire and Nottinghamshire before the end of 1975.

At the Rye Meads Sewage Treatment Works of the Thames Water Authority, a proportion of the effluent enters percolation lagoons from which about 4,5 Ml/d returns to replenish the groundwater system.

At Whittier Narrows, U.S.A., about 45 Ml/d of reclaimed water are infiltrated into the ground to replenish natural waters.

At Santee, California, highly purified sewage effluent is used without dilution to supply a series of recreational lakes.

4 Effects of Rising Energy Costs on Waterworks Operation

4.1 General Considerations

As I write, a further increase of 10% in the cost of oil has been announced, underlining the "new facts of life" for water treatment operations. Designs of recent years replacing labour intensive processes with those relying on chemicals, themselves energy intensive, and power, must now be reconsidered and a return to traditional methods contemplated. The world-wide recession caused, in parts, by the inflation produced from energy costs is, itself, reducing water consumption and making demand prediction difficult.

4.2 Examples of Energy Constraints

In Denmark, consumption of water is lower in cities than before the oil crisis in November 1973, and old pumping plant is being replaced to reduce energy usage.

Finland, because of its cold climate, has always paid attention to thermal insulation which has proved useful in the planning of sewage disposal works. For pumping, pressures are studied and electricity used at night to fill storage and even out electricity usage. Economies are at individual plants—no national directives have been issued. The high cost of petroleum derivatives has reduced the use of plastic pipes for water mains.

Pumping and chemical usage are both being studied in France as both are directly related to the cost of energy. This points to more extensive use of biological, as opposed to chemical treatment and pressure filters rather than gravity types.

In source development, full development of local underground sources is considered preferable to importing water over long distances.

Diesel pumps should be bought for standby use only and service maintained with the largest possible pumps, these being more efficient. This appears to clash with the desire for the use of local sources which are likely to be small. However, the various factors will need to be weighed for each scheme to arrive at the best energy balance.

To save energy, more water storage will be required, especially high storage which in pumped storage scheme is a source of electrical energy to meet peak demands.

In the preliminary calculations for a waterworks producing 100 000 m^3/day absence of high storage could necessitate the use of 2 MW of electric power. Pressure problems in undulating areas need careful study to keep pumping heads as low as possible.

In designing conduits, a new factor enters classic design formulas which must in future include:

—the cost and likely future costs of electricity

—the likely interest rates on capital

—the trend for public works costs at the given moment

—the likely course of inflation

Such design studies are particularly difficult at the present time. In France a programme called VERSEAU (Verification Economique des Renforcements des Services D'Eau) has been developed to assist this process.

Ireland, low in natural energy resources, is looking increasingly to gravity rather than pumped schemes, and where water is imported for both hydro-electric and water supply purposes, it may now be necessary to maximise the valuable energy output at the expense of the most convenient arrangement for water supply. An example of this is to be found in the United Kingdom Kielder Water Scheme which involves a large 188×10^6 m^3 impounding reservoir in the north east. In the early years of operation, the water required for river regulation purposes (up to 1230 Ml/d releases expected eventually) will be minimal and, during this period therefore, the reservoir will be kept as high as possible for extended periods throughout the year. This clearly improves its power generation capacity and three turbines coupled to 1,6 MW synchronous generators, plus a smaller 200 kW turbo-alternator, will feed energy via a transmission line into the National Grid at the Spadeadam sub-station, the energy being purchased by the North Eastern Electricity Board. In order to conserve water at a maximum head without overflow loss, and to help alleviate downstream flooding, it is intended to maintain the reservoir level at 1,6 m below top water level. As and when the flow into the reservoir increases, then the output from the turbo-alternators will be increased also to maintain the 1,6 m level. When the turbo-alternators are running at maximum capacity, the level in the reservoir will rise when the inflow is in excess of this quantity, but calculations indicate that overflow losses will be minimal if the 1,6 m rule is adopted. Once the level drops below the 1,6 m below TWL, output flows through the turbines will be reduced until eventually the quantity flowing through the turbines will be equal to the quantity required for regulation purposes. The level in the reservoir will then fall in accordance with system demand, until such time as the inflow is adequate to raise it to the 1,6 m below TWL conditions once more. The top 1,6 m freeboard in the reservoir represents a stock value of 18 200 Ml.

It is interesting to note that whereas calculations made in 1971 indicate that it would be uneconomic to generate power at Kielder, the enormous increases in fuel costs since that time have resulted in this power generation scheme now becoming a viable proposition when the data used for the 1971 study is updated. In time it will be necessary to raise the level in the reservoir towards the end of the winter to ensure that the reservoir is virtually full by spring of any year to cater for a possible dry summer to follow, thereby ensuring that a maximum quantity of water is available for regulation purposes. This condition is unlikely to occur during the first 15 years of operation. As the demand for water source abstraction increases, the time will come when the 1,6 m rule cannot be maintained throughout the year. The first feasibility study performed by Consultants in early 1971 concluded that "a generation scheme of even modest proportions would be out of the realms of practicability". However, when the data used were revised in 1974, the Consultants found that annual profits from a hydro-electric scheme could be as high as £100 000 in a wet year and £30 000 in an average year. The main cause of the change is the price per unit offered by the Electricity Board, based on fuel prices. The estimated capital investment required had increased by a factor of 3 in 1974, while the estimated annual income from sale of electricity had increased by a factor of 6 to 7, reflecting the astronomical increases in fuel prices during the energy crisis period.

The cost effect of the crisis is shown in figures supplied by Japan. In 1973 water supply used 0,35 kWh per cubic meter of water. Costs increased as follows:

	Water Supply (mil. m³/ year)	Energy (mil. kWh/ year)	Former Energy Price (%)	New Price (%)	Increased ratio (%)
Tokyo	1 660	565	5,2	11	212
Kyoto	218	51	2,8	5,9	211
Osaka	398	267	16,5	35,7	215

Where fire-fighting needs are likely to exceed those needed for domestic supply in Bulgaria, an independent fire system is provided to reduce excessive energy costs for daily domestic use. Asbestos cement pipes are also preferred because of their low friction losses.

The production of potable water must now be considered as a continuous cycle including the treatment of waste water for re-use further down the river system, and the tertiary treatment of sewage to a high standard tends to use water supply technology, e.g. micro straining.

Advanced technology tends to be energy intensive both in the construction and in subsequent running, and does not always give comparable results to the systems replaced. For example, in water treatment the slow sand filter with better methods of cleaning is regaining favour against the rapid gravity filter, the slow filter giving a better filtrate from the bacteriological point of view and needing less energy to operate.

In sewage disposal, activated sludge, oxygenation ditches, etc., all need more energy than the traditional rotary biological filters they replaced which can be operated by gravity. For sewerage, gravity sewers to local works have been replaced by the small bore pumping of sewage long distances to large works with high pumping costs, and in some cases the use of oxygen injection to avoid the septicity induced by the long pumping runs.

In coastal areas, disposal to sea can be an efficient method and energy economical as it uses the vast energy resources of the sea and sun to effect purification.

The design of aqueducts will also be affected as overall energy balances will tend to favour large diameter and high first cost pipelines against smaller pipelines with constantly increasing pumping costs, even if discounted cash flow techniques tend to under-value the long-term benefits in cash terms. In the U.K. Government Schemes of assistance for the provision of energy saving schemes have been announced although details are awaited at the time of writing. It is to be hoped that such assistance would help to promote such schemes among Water Authorities. It may be possible to give some examples to the Amsterdam Conference.

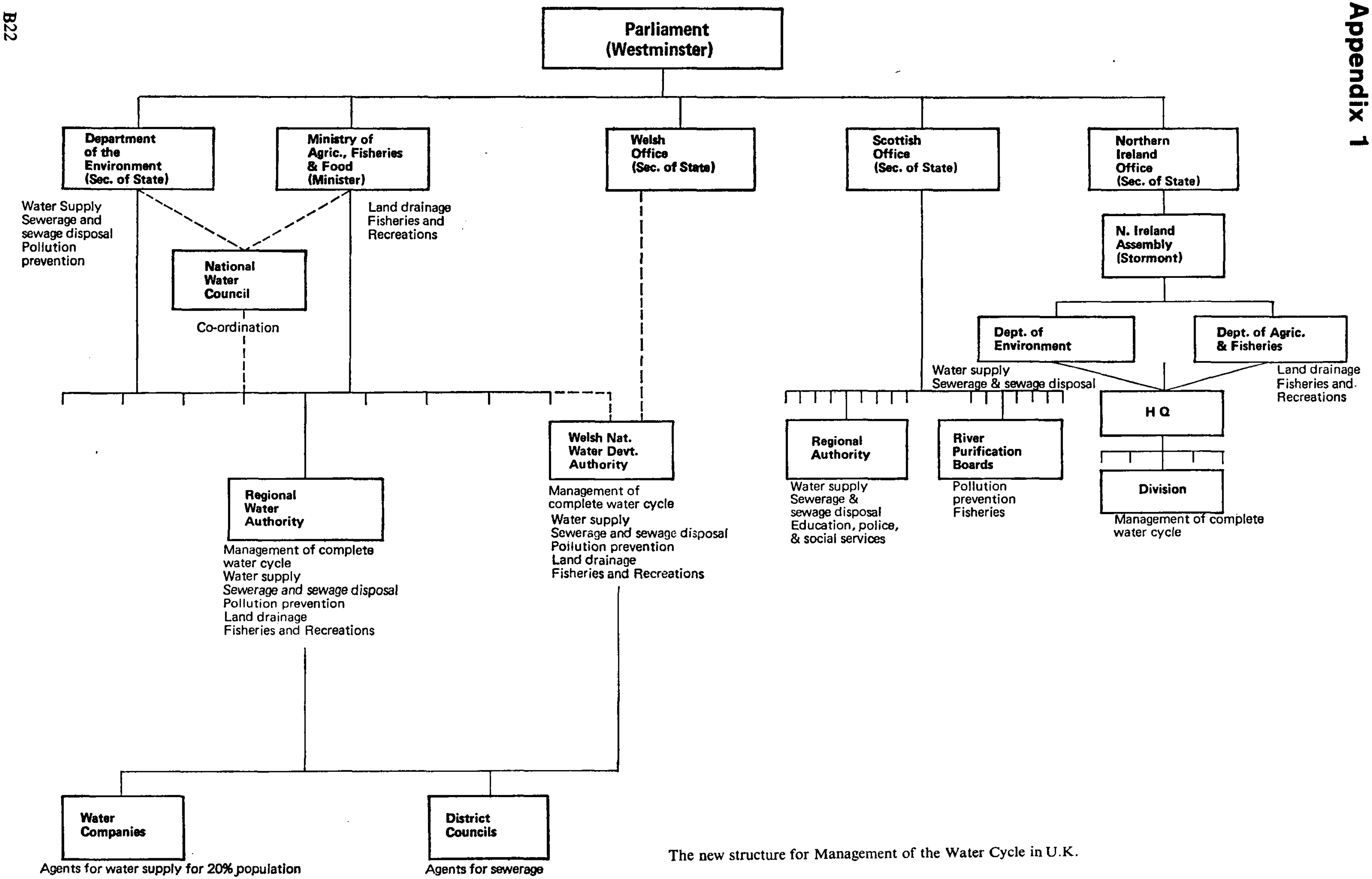
In the United Kingdom, hydro-electric power is of limited potential except in the Severn Estuary where the second highest tidal range in the world has a potential to produce 10% of the United Kingdom's electricity demand, if the usual tidal problems of intermittent generation can be overcome. With the increase in energy costs, the Central Electricity Generating Board is reassessing its economic and technical feasibility (Reference 9). Increasingly the ingenuity of United Kingdom engineers will be orientated to low cost, low energy solutions of water supply problems to meet future needs.

In Italy, works are not readily being modified to reduce energy costs as capital costs are increasing faster than energy costs. In the last two years capital costs have increased 3,5 times compared with 2,2 for electricity.

The Netherlands also report the surprisingly low figure of 1–2% of total water cost being represented by the cost of power so that increases in power costs still have only a small effect. In spite of this, one water company is known to be reducing night pressures to save energy and the sizing of trunk mains is also likely to be increased to decrease power consumption.

One of the waterworks in the Moscow area of U.S.S.R., having a capacity of 50 000 m³/d, has managed to decrease undue water use by 10% and lower its electrical power consumption by 15%. This has been achieved by carrying out a careful programme of economy measures.

Appendix 1



The new structure for Management of the Water Cycle in U.K.

Appendix 2

LIST OF NATIONAL RAPPORTEURS WHO CONTRIBUTED TO THIS REPORT

1. This General Report was drawn up following a review and analysis of the information supplied in the National Report submitted by the following countries:

Bulgaria	Professor Hristo Hadjev
Czechoslovakia	Mr. M. Chalupa
Denmark	Mr. P. Friis
Finland	Mr. R. Piippo
France	Mr. P. Schulhof
Ghana	Mr. K. M. Addison
Ireland	Mr. K. O'Donnell
Italy	Dr. F. Meucci
Japan	Mr. Kiyoharu Taniguchi
Netherlands	Mr. W. C. Wijntjes
Sweden	Mr. R. M. Stenberg
U.S.S.R.	Professor F. Chevelev

2. In addition, the General Rapporteur, Mr. J. A. Young, who also acted as National Rapporteur for Great Britain, acknowledges the valuable help and support received from the following Water Authorities, Water Companies, Research Organisations and individuals in preparing the British National Report:

Bristol Waterworks Company, J. R. Browning.

Chester Waterworks Company, S. C. Watler.

East Worcestershire Waterworks Company, R. H. Burch and D. E. Burgess.

Essex Water Company, P. G. Spencer and D. F. Moye.

Newcastle and Gateshead Water Company, H. D. M. Speed.

Northumbrian Water Authority, D. F. H. Pain.

Rickmansworth and Uxbridge Valley Water Company, N. J. F. Mackett.

Severn-Trent Water Authority, D. A. D. Reeve and B. Rydz.

Tendring Hundred Waterworks Company, J. A. W. Rayner.

Thames Water Authority, E. C. Reed.

Water Research Centre, D. Gill.

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NOTE: These are other than the reports submitted by National Rapporteurs

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Contrôle De La Demande Pour L'eau Potable

Résumé

par J. Anthony Young, C.Eng., F.I.C.E., F.I.W.E., M.I.Struct.E., M.B.I.M.

2.2 Il y a trois méthodes principales pour régulariser la demande en eau potable:

(a) méthodes techniques

(b) méthodes financières et légales

(c) méthodes d'éducation.

2.3 Méthodes techniques

Il faut s'attaquer aux différents éléments en combinaison:—le remplacement des conduites maîtresses vieilles et défectueuses; la réduction des pressions excessives; la réduction des surpressions et des coups de bélier; l'emploi des matériaux qui résistent à la corrosion et à la vibration, etc. . . .

Le remplacement des conduites maîtresses et des branchements entraîne des dépenses énormes. En période d'inflation, il n'est pas possible de remplacer assez rapidement les conduites maîtresses (ni les égouts). Le débit des fuites s'accroît donc, spécialement quand les réseaux sont centenaires.

La consommation, spécialement dans les jardins, est souvent excessive si la pression est trop haute; les fuites s'accroissent aussi. Mais la pression doit être suffisante et, en général, il faut maintenir une charge de 30 m d'eau.

Des expériences en Afrique du Sud ont montré un rapport intéressant entre la pression et la consommation; quand la pression monte de 60%, la consommation s'accroît de 30%. On cite aussi les résultats d'autres études aux Pays-Bas, en Bulgarie, en Grande-Bretagne, en Finlande, etc. On finit par recommander la réduction de la pression en quelque lieu que ce soit. Quand on remplace les conduites maîtresses, il faut examiner la pression d'alimentation et la réduire si c'est possible.

La découverte et la réparation des fuites est une tâche permanente, si bien conçu et entretenu que soit le réseau des conduites maîtresses. On décrit les deux types de compteurs de fuites qui sont employés en Grande-Bretagne et un système alternatif utilisé en Suède dans les réservoirs de service. Un nouveau détecteur à corrélation acoustique de fuites développé par le Water Research Centre, (G.B.), donne de grandes espérances pour l'avenir.

Les pertes dans les locaux de consommation domestique seront analysées en détail lors d'une étude proposée par le Severn Trent Water Authority, G.B. On a choisi 1178 maisons pour cette étude, et la consommation et l'utilisation des appareils domestiques qui consomment l'eau seront enregistrées chaque jour.

Il faut réglementer avec soin la fabrication et l'installation des robinets et accessoires pour minimiser les pertes. On pourrait économiser jusqu'à 10% par l'adoption de chasses d'eau à volume variable: 4,5 l (usage secondaire), 9 l (usage majeur). On discute aussi les avantages des citernes domestiques, compte-tenu du risque de contamination.

2.4.2 Méthodes financières et légales

On peut régler la demande en eau, spécialement si l'on emploie les compteurs, par le mécanisme des prix. Il faut fournir l'eau pour les usages principaux (par exemple: hygiéniques ou culinaires) à prix raisonnable mais, pour les usages de luxe (par exemple: pour le jardin, le lavage des autos, etc. . . .) on peut facturer un prix réaliste reflétant la valeur réelle économique de l'eau, spécialement quand les ressources sont rares.

En général, on facture aux consommateurs industriels le volume d'eau consommée, ce qui implique l'emploi de compteurs. On facture de plus en plus d'une manière semblable les consommations domestiques dans la plupart des pays qui ont soumis un rapport national, à l'exception du Royaume-Uni et de l'Irlande. Mais il faut trouver un tarif qui soit réaliste et qui limite effectivement la demande. On discute les tarifs utilisés en Italie, aux Pays-Bas, en France, aux Etats-Unis, en Russie, etc. . . ., et on compare ensuite les avantages et les désavantages des compteurs pour les consommateurs domestiques.

2.4.3 On peut diviser en deux catégories les méthodes légales:

(i) règlementation des conduites et des installations

(ii) restrictions pendant les périodes sèches ou après un accident (par exemple: une grande pollution).

Il semble que les consommateurs se ressentent plus des restrictions, spécialement quand le prix de l'eau est élevé. Mais une période de restriction une fois en dix ans par exemple semble être raisonnable. Il faut éviter des investissements excessifs dans les ressources qui sont rarement utilisées.

2.5.2 Méthodes d'éducation

Le public a besoin d'éducation et de publicité en ce qui concerne le caractère essentiel du service et l'importance d'économiser l'eau spécialement dans certaines régions où l'eau est devenue une ressource rare. Il faut bien comprendre que le public regarde le développement des ressources en eau comme une atteinte à l'environnement; on ne peut donc pas négliger l'information du public dans l'espérance qu'il nous considérera toujours comme des bienfaiteurs de l'humanité.

2.5.3 Il faut commencer l'éducation dans les écoles et la continuer par les visites, l'information (par exemple: les annonces, la télévision, etc. . . .) et la participation du public à la réalisation et à l'exploitation des nouveaux travaux.

On cite trois exemples de campagnes de publicité au Japon, en Irlande et en Angleterre pendant des périodes sèches.

2.6.4 L'utilisation de l'eau en pays développés comportant des zones arides par exemple les Etats-Unis, l'Australie, l'Afrique du Sud, est très grande.

La grande consommation d'eau à Windhoek en Afrique du Sud Ouest a obligé à construire une usine pour la purification d'eaux résiduaires aux normes de l'eau potable. La méthode était compliquée et coûteuse et il y avait aussi le problème d'éliminer les virus, mais elle a été très utile pendant que l'on faisait construire un nouveau réservoir.

Par contraste, dans les pays en développement, la consommation de l'eau est inconnue et supprimée par indigence ou manque de ressources.

La réalisation d'un réseau d'adduction d'eau, sain et suffisant est essentielle pour lancer le développement d'une communauté.

En Arabie où les zones arides ont de grandes ressources en pétrole et gaz, on doit compter sur l'extension des usines de dessalement.

3.1 Pour récupérer des eaux résiduaires industrielles, il est possible de recirculer l'eau potable, après utilisation industrielle, pour une autre utilisation de qualité plus basse.

3.3.1 On peut construire un double réseau de canalisations pour fournir de l'eau secondaire moins chère.

3.4.1 3.4.2 Il est possible d'utiliser l'eau d'égout épurée avec ou sans traitement additionnel, pour utilisation industrielle par réinjection dans une formation aquifère.

On discute les méthodes employées par la France, le Danemark, le Royaume-Uni, le Japon (70% réutilisés en 1975!), la Bulgarie et les Pays-Bas.

En Afrique du Nord (Libye) par exemple, l'eau d'égout épurée d'une qualité 10.10 après traitement au chlore est distribuée à cent fermes pour l'irrigation des légumes.

3.5.1 L'utilisation des eaux résiduaires pour réalimenter les formations aquifères présente quelques problèmes. Par exemple, il est possible que des produits chimiques dangereux passent dans l'aquifère et il y a aussi la possibilité que les fissures soient bloquées par le dépôt des solides pendant leur passage.

Cependant leur utilisation continue de s'accroître.

3.5.2 Le Rhin est si pollué qu'on doit considérer son utilisation par les Hollandais pour l'eau potable comme utilisation d'eaux résiduaires et depuis longtemps son traitement avant infiltration dans les dunes pour La Haye et Amsterdam est bien connu.

Il y a aussi des exemples de réalimentation d'une formation aquifère à Long Island aux Etats-Unis et pour le refroidissement de câbles électriques souterrains dans le Royaume-Uni. Après utilisation on renvoie l'eau dans la craie par des puits.

4.1 Les distributions d'eau commencent à se ressentir des effets de l'augmentation du coût de l'énergie.

Elles constatent aussi un ralentissement du taux d'accroissement traditionnel de la demande en eau. Il en résulte une augmentation des prix de revient en raison d'une consommation d'eau moindre.

4.2 Par exemple, au Danemark la demande est plus basse aujourd'hui qu'avant la crise du pétrole.

Dans beaucoup de pays les avantages de pomper la nuit pour remplir les réservoirs et réduire le coût de l'électricité sont bien connus.

Le pompage et l'utilisation des produits chimiques sont le sujet, en France d'une étude spéciale, et aussi le retour aux méthodes de traitement biologiques et traditionnelles.

Pour l'étude des conduites maîtresses, un nouvel élément entre dans les formules classiques. En France, un programme d'ordinateur VERSEAU* a été mis au point pour ces calculs.

Il est possible, aussi, d'utiliser l'énergie mise en charge dans un réservoir d'eau potable pour la production d'électricité spécialement dans les débuts d'existence du réservoir quand la demande en eau n'est pas très grande. Par exemple, le réservoir Kielder dans le Royaume-Uni (180×10^6 m^3 de capacité) où on propose d'installer 3 turbines capables de produire 1,6 MW chacune et aussi un turbo alternateur de 200 kW pour desservir le réseau électrique national. On espère économiser £100 000 en année pluvieuse.

Au Japon, on estime que les coûts ont augmenté de plus de 200%.

On doit reconsidérer les sites des barrages hydroélectriques comme l'estuaire de Severn dans le Royaume Uni où il y a une forte marée, la deuxième du monde en hauteur.

En U.R.S.S. à Moscou, l'usine de distribution d'eau dont la capacité est de 50 000 m^3/jour a réduit la consommation en eau de 10% et la consommation en électricité de 15%.

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* Vérification Economique des Renforcements des Services d'Eau.

Automatic control of large water supply systems

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1 Introduction

The purpose of this Special Report is to summarize the general situation in automatic control or instrumentation of large water supply systems at the present time.

Automatic control, however, remains a secondary or selective component of overall waterworks facilities, and its development and installation are greatly affected by the social circumstances of the countries concerned. Therefore, this paper is confined to the current situation in Japan with certain comments for future development.

"Automatic control" is not always clearly defined. Sometimes it means automation or automatic operation, whilst at other times it is limited to the narrow sense of feedback or feed-forward control. The author interprets it in a broad fashion to include the techniques and devices applied to supervising, controlling and data-processing in the operation of water supply systems. As to the utilization of automation in the waterworks field, several reports to the Congress have treated the subject (1, 2, 3). Here its composition and the terminology used are so arranged that they may suit the subject of this paper.

With regard to "large water supply systems", the author has in mind both the twenty-seven existing water supply systems in Japan, having nominal capacities of 200 000 m^3/d and over, up to 6 232 000 m^3/d, and the fifty-nine purification plants within a range of rated production of 100 000 to 1 820 000 m^3/d.

2 General

2.1 History of instrumentation in water supply systems

Water supply systems in Japan have developed rapidly within the last thirty years, e.g. annual treated water supply and population coverage ratio in 1950 was approximately 2 billion cubic metres and 25,0 per cent respectively, which had grown to 12 billion cubic metres and 86,7 per cent in 1974.

In accordance with such a rapid expansion, up-to-date instrumentation has been introduced extensively especially into purification plants, adopting factory type controls such as used in the chemical industry, with the aim of better and more economical operation.

Modern control techniques were initiated in pumping stations as one-man control in 1950, and have progressed to speed regulation with power-loss recovery, which is widely employed at present.

Instrumentation with analogue instruments and sometimes with sequential controllers, having appeared in 1952 has almost established a standard pattern within ten years in new purification plants. Digital control techniques were first introduced into plants as data-loggers in 1963, and at present, the majority of large purification plants are equipped with computer control systems (CCS), which function mainly as data-loggers, whilst developing the proper operation of CCS step-by-step.

Several water supply systems are arranging computer-aided centralized supervisory control systems in a hierarchical composition, for the optimization of water supply systems control (WSSC). This appears to be the likely mainstream of computer utilization in the field of waterworks control.

2.2 Instrumentation yardstick

A yardstick for automation has been defined (4) in ten classes as: A0-none, A1-energy, A2-dexterity, A3-diligence, A4-judgement, A5-evaluation, A6-learning, A7-reasoning, A8-creativeness, and A9-dominance.

Assuming that a minor loop of feedback control (FBC) corresponds to A4, then sequential control (SQC) is equivalent to A3, 5; combined FBC to A4, 5; feed-forward control (FFC) to A5; optimal control to A6; and WSSC to A7, respectively. Generally speaking, therefore, automation (or instrumentation) up to the level of A5 has now been put to wide practical use, and further, A6 and A7 are at the phase of positive research and development in the field of waterworks. Current concern is focussed on the alternatives of operational and informational control, or instinctive and intellectual levels of control.

2.3 Evaluation of instrumentation

It is hard to estimate the merit of instrumentation quantitatively, yet the following are generally recognized factors:

(1) Concentrated structures on a larger scale, responding to needs as circumstances require, have become feasible, which has enabled both economical construction and operation.
(2) Quicker response to a change in operational conditions has become possible.
(3) Quantities of manual work and labour involved have been considerably reduced.
(4) Although to a limited extent, consumption of chemicals and electrical power have been appreciably reduced.
(5) An active attitude has been introduced into the waterworks field to improve facilities, making use of up-to-date techniques.

On the other hand, several of the problems connected with instrumentation are as follows:

(1) Correct waterworks techniques which have been developed to warrant the utmost safety and reliability in supplying water with the least "control", seem not to be sufficiently adaptable to the current stage of automatic control; however, as waterworks processes have advanced appreciably, so the controllability should be improved further.
(2) Present quality of instruments, especially detecting elements, is not entirely adequate for waterworks requirements.
(3) The reaction of waterworks personnel to instrumentation is not always the same. Generally it is more positive at the construction phase than at the operational where the latter seems to be too anxious for rapid technological revolution.

(4) The more precise the instrument technique becomes, the more it reveals weakness against an unforeseen accident, which seems to require support by certain backup means and manpower. Troubles tend to shift worryingly from the hardware part to software.

(5) The qualification and training system for the operators, in what is a new function, are very inadequate.

It should be noted that there has appeared recently amongst Japanese waterworks personnel a tendency towards a belief that there is over-emphasis on instrumentation and undue reliance upon it, and that further consideration should be given to the most adequate technology to be used for water supply systems. Also, more serious attention should be paid to human nature.

Expenditure on instrumentation for newly built purification plants is generally considered to be three to five per cent of the total construction cost.

3 Elements and techniques of instrumentation

3.1 Elements of instrumentation

Instrumentation comprises (a) detecting, (b) display, (c) control, (d) operating, and (e) transmission elements. Amongst these, (b) and (c) in a new instrumentation system with CCS would be best suited to be arranged as (b) man-machine interface and (c) data-processing elements, software inclusive.

The fundamental components of instrumentation techniques are classified as follows:

(1) Centralized supervisory and control functions.

(2) Process control, which refers to feedback control (FBC) with minor or combined loop on closed-loop basis; including constant-value, ratio, programme and cascade control mode; and proportional (P-), integral (I-), and differential control action (D-action).

(3) Sequential control (SQC), on open-loop basis; many controlled processes in purification plant and pumping station are organic aggregations of FBC and SQC components.

(4) Computer control system (CCS), including on-line control, operation guide control, set-point control (SPC), and direct digital control (DDC).

(5) Adaptive and optimal control (incl. optimizing control), based on feed-forward control (FFC).

(6) Telemetering and telecontrol.

(7) Water supply systems control (WSSC), by which the author means optimal control applied to a whole waterworks complex: further, a concept of total water supply systems control would be defined as a combination of WSSC with a management information system (MIS).

3.2 Signal and operating element

For detecting elements in waterworks, electric and electronic signals have been used widely, the latter to be standardized to 4 to 20 mA DC range following the Recommendations of the International Electro-technical Committee (IEC). For operating lines, electrical, electronic, pneumatic, and hydraulic signals are adopted, of which some examples are mentioned below:

(1) For a small sized control valve, as in chemical feeders, the pneumatic system is still prevalent.

(2) For medium size with slow action, e.g. effluent control of filters, an electrically driven valve has been adapted increasingly in a velocity type action which is convenient for CCS.

(3) For a large size, in need of rapid or precise action as in the delivery of a pump, hydraulic drive is often applied.

In case of a pump capacity control system, the speed controlling gears, e.g. are regarded in aggregate as the operating element.

3.3 Detecting element (5)

3.3.1 *Functions Monitored.* The following functions are commonly open to monitoring whether as measured quantities or as situation signals.

(1) Hydraulic; flow quantity, total quantity or velocity in both pipe and open channel, level and pressure, density of liquid, etc.

(2) Water quality and chemical; temperature, turbidity, colour, alkalinity, electric conductivity, pH value, residual chlorine, chlorine demand (on trial), DO, concentration of liquids, etc.

(3) Meteorological; temperature, humidity, velocity and direction of wind, rainfall, etc.

(4) Mechanical; openings of gates and valves, rotating speeds of pumps and mixers, temperature of bearings, operating conditions of machines, flow, level, weight of powdery and granular materials, etc.

(5) Electrical; voltage, current, power, power factor, etc.

The detecting element is often called a sensor, and also called a transmitter on the market, inclusive of built-in transducer.

The detail on detecting elements in the following paragraphs is limited to certain special features of them, as their principles and structure are common the world over.

3.3.2 *Large Flow Quantity.* Measurement of large flow quantities is naturally important in waterworks, yet there are still difficulties with accuracy.

As to the implements, in addition to Venturi tubes, including the bi-directional one, electro-magnetic systems (used up to 2 400 mm) and super-sonic systems (up to 2 700 mm) are coming into wide application. The former is more accurate and not sensitive to the shape of conduit, whilst the latter can be installed in existing pipes regardless of diameter, and discriminates flow direction. The response characteristics of the latter are being improved.

The flow in an open channel is measured by electro-magnetic, ultra-sonic, and electro-static detectors, directly or in combination with a weir.

It is a fundamental rule to calibrate the measuring device with actual quantity, and especially in the case of large flow detectors this often requires to be done in the field.

3.3.3 *Water Quality* (6, 7, 8). Although the monitoring of water quality is undoubtedly the most important and essential thing in waterworks instrumentation, it remains still the most difficult in the present situation, as outlined below:

(1) Not all the detectors needed have been developed yet for industrial use, although they are expanding the application range step by step, e.g. the utilization of gas-chromatography in process measurement.

(2) Some of the detection principles differ from those of water quality standards.
(3) Maintenance usually requires considerable labour: the majority of detecting instruments are delicate and complicated in structure and consume an amount of reagent.
(4) In the case of a concentrated installation system, the water sampling pipes are apt to cause trouble in the form of altering water quality.

3.3.4

Particular Substances. Detecting devices, e.g. for the following particular substances have been devised and put to practical use.

(1) Rupture detection of the transmission main; also a unique example of a seismometer-actuated effluent valve for gravity distribution reservoirs.
(2) Chlorine gas leakage detector.
(3) Detectors of detrimental substances; phenols and floating oil detectors have become almost practicable.
(4) Monitoring of river water quality, carried on by the authorities in charge of river administration and pollution control, includes detection of COD, TOD, TOC, cyanide, ammonia, cadmium, copper, and mercury, in addition to the foregoing items in (3).
(5) Level and concentration of sedimented sludge.

3.4 Display device and process controller (man-machine interface)

The nucleus of the control room is a chain of panels installed with indicating instruments, various display devices and process controllers on them, accompanied by a set of operating desks equipped with switches to operate machines including sub-stations, pumps and valves.

In the case of CCS, the central processing unit (CPU) and the peripheral equipment are established in a data-processing room, ordinarily adjacent to the control room, and only the logging typewriters and a part of the in- and out-put devices (I/O) are placed in the latter.

Recently, especially since the appearance of CCS, I/O for operational use has become called the "man-machine interface". It is perhaps the part that has been transformed most remarkably as instrumentation techniques have advanced. The following deals in more detail with the above:

(1) The panel has progressed through the sequence of plain, graphic, and semi-graphic mode, and now is changing its leading role with the operating desk. The semi-graphic panel, which is the most prevalent in combination with analogue instrumentation at present, is also convenient for the remodelling often required in waterworks facilities; the mosaic-form panel is applied too. Since CCS was introduced, the number and significance of analogue instruments have decreased. Some new purification plants having CCS have eliminated all the instruments from the control room. The graphic panel is likely to be revived again as an auxiliary display device with CCS.
(2) Operating desks have grown up markedly, importing one after another, selective control equipment, logic controllers for SQC, industrial television monitors (ITV), display devices (especially cathode ray tubes), information input devices, and process controllers, to be the operator's console, which is now the focal point of the so-called man-machine interface. The engineer's console might also be established separately.
(3) With regard to process controllers, the high density type deviation-indicating controllers are to be preferred, together with recording or point-indicating controllers, some of which are modified to CSS-oriented design.
(4) The local operating panel (or desk) is placed according to each group of controlled units, e.g. chemical feeders, filters, and pumps, on which the components are operated individually. The local operating panel is also increasing in its function to become more independent as a terminal station in the hierarchical computer order, being equipped with a mini-computer or micro-processor.

3.5 Sequential control (SQC)

The sequential controller or sequencer has been developed by adapting the principal elements in a progression of drum timer, relay circuit, rotary switch, semi-conductor circuit, and integrated circuits, to become finally a general purpose logic controller or DDC. The sequential programme is given by wiring, pin-board, or memory circuits roughly in that order.

Electrical and pumping installations have been controlled by an originally developed SQC system, including special protection relays for safety, reliability and rapid response, but there is a tendency to introduce electronic techniques to make them simple, standardized and "intelligent".

3.6 Computer control system (CCS)

The greater part of large scale purification plants are equipped with CCS at present, whereas some newly built plants have not introduced it because it was judged that the time would be too early.

The digital control technique was first introduced into purification plants as a data-logger to save instruments, and produce better-arranged information. Thereafter its vast possibility for data-processing has raised expectations of improving the operational situation, initially of the purification plant, then of the whole waterworks complex.

As to the present condition of CCS in waterworks, the following points are mentioned:

(1) The majority of CCS are working as data-loggers, while collecting operational data, analysing them, and studying application techniques for more advanced control.
(2) After considerable efforts in developing the algorithm, SPC and DDC are just getting under way for some processes in the purification plant, e.g. chemical application, filtration and water apportionment.
(3) CCS has increasingly become expected to develop to WSSC as its most promising utilization.

The initial CCS to date have been based on the "centralism", i.e. multiplex use of a large scale CPU, where any back-up system, e.g. duplex or dual, has become very important, and the wiring between CPU and terminal implements is also very complicated.

The appearance of the mini-computer has opened up the possibility of distributed and hierarchical CCS, and this new trend will be decisively advanced by the practical application of the micro-processor and the data-highway or computer linkage techniques.

CCS can be adapted also as an in-line or operational guide control system, which might be applicable to old-fashioned facilities.

The computer has undoubtedly huge capabilities, but perhaps too much has been expected of it. It is still progressing, and possibly too rapidly, since auxiliary devices, e.g. sensing elements and the necessary software seem to have been left far behind in development.

Up to date, the analogue- or hybrid-computer has not been applied as a component of CCS.

3.7 Self-actuated automatic control

Automatic control technique originated as the self-actuated device. In addition to popular applications of the float-valve, reducing valve, etc., many ingenious devices have been adapted also to processes, e.g. those of chemical application, in purification plants.

Self-actuated control devices are not always easy to develop but need, as an essential step, consideration of the process itself. It is not very accurate in operation but requires little maintenance, and there would be many situations where it could be adequately adapted. There is also a recent tendency to re-evaluate the self-actuated control system, as in the case of siphonic automatic filters.

3.8 Telemetering and telecontrol

The techniques of signal transmission are classified as (a) wired direct selective, (b) wired carrier, and (c) radio carrier system.

(a) is applied for short distance with a lot of signals, e.g. control of sub-stations or pumping stations from a control room within a plant site. As to the transmission line, for (b) inclusive, both the private line and that supplied by the telephone corporation are available.

Of the wireless system dealing with a large number of signals, the multi-directional multiplex method on 400 MHz band employing a pair of waves is becoming prevalent, especially for the star form system.

The signals on the carrier system are mainly based on the cyclic digital type with error-detecting code.

3.9 Personnel and maintenance

In the present situation, no waterworks undertaking has any specialists in instrumentation, and appropriate engineers within the undertaking are charged purely by chance with planning and system development. Consulting firms or manufacturers most often play the leading role.

Nonetheless, where instrumentation is installed, the personnel of the waterworks undertaking are, as a rule, deeply engaged in its operation, maintenance, and development in co-operation with these firms.

Operation of instrumentation, i.e. that of the process itself, and the daily maintenance are performed by operating attendants on a three shift basis, and there has appeared a tendency to assign superior personnel in small numbers to the operation.

Systematic inspection and repairs are carried out both by an undertaking's own maintenance corps with repair shops, and by contractors once to several times a year.

3.10 Criteria and standards of instrumentation

Both of "Design Criteria for Waterworks Facilities" and "Guide-lines for Waterworks Technical Management" compiled by the Japan Water Works Association devote one of their nine chapters to instrumentation.

Technical terms and symbols for instrumentation and data-processing are established by Japanese Industrial Standards (JIS).

The shapes, sizes, performances and testing methods of instruments and other devices are determined by JIS and other various standards of the Associations concerned.

4 Instrumentation for purification plants and pumping stations

4.1 Centralized supervisory control

The supervisory and control functions of a large scale purification plant are usually concentrated into a control room. In the case where there are several control rooms, one of them is charged with overseeing the others.

The number of controlled variables collected are given roughly in the examples below:

A purification plant having a rated capacity of 1 700 000 m^3/d receives by CCS, 400 analogue, 84 coded, 16 pulse, and 104 sampled signals, and there are used 322 analogue instruments and 295 electrical meters besides.

Another plant of 140 000 m^3/d, which is designed with a view to simplifying the installation, handles 102 hydraulic and chemical variables and 48 electrical ones, connected with 98 indicating instruments including trend recorders.

As to alarm signals, in addition to the conventional heavy alarm (bell, followed by emergency stop to operations), light alarm (buzzer), and appeal (chime), each accompanied by lamp-flickering, verbal announcement has been found to be practical.

4.2 Chemical application

4.2.1 *Handling and Preparation of Chemicals.*

The majority of chemicals used in large scale purification plants, such as alum, poly-aluminium chloride, caustic soda, sulphuric acid, sodium silicate, etc., are of liquid form, and are transported by tanker lorries and stored in tanks, being controlled by levels.

Some plants apply the bulk system to chlorine, where stationary tanks receive liquefied chlorine from tanker lorries to store it, and are supervised by pressure and level.

Lime, activated carbon for dosing use, potassium dichromate, and some coagulant aids, e.g. sodium alginate, are usually powdery or granular in form, and are transported in bulk, in containers or in bags, and sometimes stored in silos, controlled by weight or level.

Chemical handling processes, where manual labour is inevitable, will be the most difficult field in the planning of instrumentation.

Batch processes that prepare chemicals, e.g. dissolving or neutralizing procedures, are often operated automatically by SQC.

4.2.2 *Chemical Pacing.*

The ratio control mode is the most popular and successful method applied to chemical feeders, including in the case of CCS, set-point control (SPC) or direct digital control (DDC) mode.

Automatic determination of coagulant dosage has become gradually practicable by means of CCS, whose methods are considered in the following:

(1) Experimental formulae or table reference system, with parameters including turbidity and alkalinity, and sometimes temperature and pH value.

(2) Retrieval system of historical data on similar conditions.

(3) Mathematical model system based on theoretical analysis of sedimentation.

(4) Pilot settling tank system.

(5) Pilot filter system, especially for filtering agent dosage.

Among them, (1) to (3) provide the basis of FFC, while with (4) and (5) FBC can be related.

Other than the coagulant, the following controls have been utilized up to date.

(1) Straight FBC for pre- and post-chlorination, both by analogue instrumentation and CCS.
(2) Pre-chlorination dosage control with regard to the residual chlorine in settled water, by means of FFC, estimating the decrement within sedimentation basins, combined with FBC.
(3) Post-chlorination dosage control, similarly based upon the decrement in the clear water tank on the basis of predicted delivery.
(4) Dosage of caustic soda solution as pH regulator is controlled by means of straight FBC alone or in combination with FFC according to the pH value in the treated water.

It is generally considered that the chemical feeding process has a longer dead time, which must be compensated by the sampled-data control mode, and is basically suitable for CCS.

4.3 Rapid sand filters

4.3.1 *Rate of Flow Control.*

The rate of flow of a rapid sand filter, except the automatic siphonic type, is detected mostly by short Venturi tube, or rarely by ultra-sonic flow meters. In some cases using CCS, it is derived from the opening of the effluent valve without any flow detecting device.

As regards a flow controlling device, usually set in effluent lines, each with a special butterfly control valve, an adaptation of the shut-off valve of water-tight butterfly or cone type, or a kind of siphonic controller is utilized.

The rate of flow control is carried out in the following ways:

(1) Constant rate basis is the most prevalent; the desired rate is set for a group of filters simultaneously, and this is called "master control". DDC has also been adopted.
(2) Declining rate filtration is applied to some plants by means of CCS. The rate is controlled on a group basis either by positioning all the effluent valves simultaneously or changing the number of working filters.

The automatic siphonic filter, with a separated water level and an effluent weir has self-regulation characteristics, which makes the effluent rate equal to influent after the single capacity lag; and the influent flow is controlled and shared equally by each working filter. Here there has to be considered some restriction for supply washing requirements.

4.3.2 *Filter Washing Operations.*

Filter washing operations are mostly controlled by SQC on a semi- or full-automatic basis: the former needs only a manual starting signal to the filter. With regard to the conditions for automatic selection of a filter to be washed, the limits for such as duration of filter-run, loss of head, effluent turbidity, and rate of flow (lower limit) are provided, but the first-mentioned is the most practical and currently used.

SQC is carried out by means of a sequencer, a logic controller in principle, or of DDC. The slow start at the re-opening procedure is usually included, but the drainage of the first water through the filter is rarely combined in the sequence.

The difficulty here would be the reliability of the great number of limit- and torque-switches attached to valves around filters, each of which has to take a part within the logic circuits of SQC.

4.4 Pumping installation (10, 11, 12)

The pump control technique consists of the operation, namely starting and stopping procedure and capacity control of the operating pump; the latter also includes two phases of its purpose and the technique to realize it.

4.4.1 *Operation of Pumps.*

The pumping installation, the largest machinery in a waterworks system, must be started, run and shut off according to a predetermined sequence in order to ensure hydraulic, mechanical, and electrical safety. It is a typical object of SQC.

Semi-automatic, or one-man control, is ordinarily applied to the main pumps dealing with raw and treated water which have to be operated selectively and less frequently than medium or small pumps, e.g. for filter washing or chemical feeding, which are often controlled fully automatically.

A pump operation system must consider possible emergency stoppages and any special precautions for preventing water-hammer, such as an interconnection with levels of one-way surge tanks, or a programmed closure of the discharge valve, etc. The system, especially on a booster pump, has to consider also the pressure or water level on the suction side, to avoid cavitation; in this connection, some have function generators to give the cavitation characteristics of the pump within the control loops.

4.4.2 *Purpose of Capacity Control.*

The purpose of pump capacity control is the regulation of discharge quantity, delivery or suction pressure (or level), as follows:

In a case where an open conduit or long pipeline comprises the discharge line, the sampled-data control mode becomes necessary to compensate for the dead-time.

(1) A raw water intake pump is most commonly controlled so that the discharge level is maintained constant. When the capacity of the receiving well is voluminous, the effluent flow to it is often incorporated into the control loop (as FFC). If the discharge well is situated at a distance, the water level can be calculated through the pipe characteristics, instead of direct telemetering, by monitoring the fluctuation of the suction level.
(2) Distribution and booster pumps should be controlled as part of WSSC, ideally speaking, yet in the present situation, constant-value or programme control mode on delivery pressure or discharge quantity is most currently applied. In the case where a distribution area supplied by a pumping station is limited for practical reasons, the constant end-point-pressure control (as FFC) is often adopted, based upon the lowest residual pressure in the distribution network with a hydraulic characteristics curve, which can be simulated, in analogue instrumentation, by a function generator.
(3) The wash-water pump supplying an elevated tank, and the wash-waste drainage pump are controlled in order to keep the level higher or lower than the set point, operating several units in a pre-set priority order. The direct back-washing and surface washing pumps are operated in a link of SQC for the filter washing procedure, and sometimes need monitoring to prevent too frequent starting.

4.4.3 *Techniques for Capacity Control.*

The means of capacity control of pumps come down finally to the regulation of the characteristics of the load (i.e.

discharge valve positioning) or of the pump (i.e. working number, rotating speed, and impeller vane pitch). They are mentioned as follows:

(1) Impeller vane pitch adjustment, though ideal in principle, has not been adopted so far for waterworks pumps with smaller specific speeds, owing to the structure.

(2) Discharge valve positioning, as an auxiliary or emergency means for large pumps, is the simplest device, but results in serious energy loss and sometimes brings problems of noise and vibration.

(3) The adjustment of the working number of pump units is adopted systematically for the case where standard sized pumps on the market can be accepted on a "many-small" basis, because residual pressure at the end point of a pipe may fluctuate within a wide range, and a relatively large energy loss has to be anticipated. Actually, because any pumping installation consists of a number of pump units, on account of diversification of risk, providing standby units, and manufacturing limits, the adjustment of number is applied naturally in most pumping installations.

(4) The regulation of speed of revolution systems, to be enumerated in the following paragraphs, has been more prevalent in capacity control methods the larger the pump unit has become, being established on the "few-large" rule.

4.4.4 *Rotational Speed Control.*

Many pump rotating speed control systems are based upon the slip regulation method, stated as follows, where an occurrence of energy loss is inevitable.

To recover the significant slip power emitted in the secondary circuit of an induction motor, secondary excitation techniques have been developed in several ways, keeping in step with the progress of semi-conductors.

(1) A variable-slip coupling, operating at a slip of (1–n), whatever type it may be, will emit an energy loss proportional to (n^2-n^3) as heat, and the means of cooling it become important in selecting it. For that reason, a hydraulic coupling, to which water cooling is easily adapted, is utilized more for medium sized pumps up to 400 kW, while the magnetic coupling, which is simpler to handle, is applicable for small pumps, and is standardized on the market.

(2) Regulation of the secondary rheostat coupled with a wound-rotor induction motor, which was originally used as a starter, is widely used as a slip adjusting device on pumps of up to 4 200 kW. Having a large power loss similar to that of the variable-slip coupling, it has become adopted for pumps with a small speed variation range, or as an auxiliary means. As to the rheostat, the liquid type is prevalent, with a view to step-less speed regulation and water cooling.

(3) The motor-generator inverter cascade (MG-Scherbius system) was applied first among secondary excitation systems, with the largest to date being 2 300 kW. The thyristor-invertor cascade (static Scherbius system) appeared in 1968 and has become the main means of capacity control for large pumps, with applications up to 6 500 kW, which is the largest so far for waterworks pumps. Also, small power sets of this kind have been standardized and are available on the market. The weakness against power cuts, the only and grave defect of this system, is now under active improvement.

(4) The induction motor with DC motor cascade (Kraemer system), one of the secondary excitation methods, was introduced to pump driving in 1964, the largest of its kind being 6 200 kW, with the highest efficiency of these kinds and the merit of requiring no additional space when coupled with a vertical-spindle pump. On the other hand, it has some of the restrictions of DC motors in design.

(5) The induction motor with commutatorless motor cascade, replacing the DC motor with a thyristor motor, was developed by a municipal waterworks utility as a countermeasure to power cuts, thus avoiding troublesome maintenance. It has been applied up to 1 900 kW since 1975.

(6) The AC commutator motor is also one variety of the secondary excitation method which needs only a voltage regulator as the auxiliary machine. It has been adapted to medium and small pumps up to 650 kW.

(7) The brushless motor has come to general notice as a maintenance-free control system with a high efficiency and a wide speed range, in spite of being expensive. For this purpose, synchronous- and asynchronous-thyrister motors have been developed and utilized for medium and small pumps up to 1 000 kW.

(8) The thyristor-Leonard system is also adapted to pumps, though it has not been very common so far.

4.5 Flow quantity apportionment

The series of flow quantities through a purification plant is controlled, based upon the clear water demand, so that the level in the clear water tanks may conform to a prescribed variation curve, keeping each flow quantity as constant as possible. Usually, manual operation is sufficient for this procedure because of the buffer action of detained water, although some automatic practices have appeared as follows:

(1) If a service reservoir also receives water from another line, transmission control would be difficult on the sole principle of single capacity lag. A very practical method in SPC has been adapted successfully, whereby water transmission is controlled from 1 to 5 a.m. so that the reservoir may reach high water level at 6 a.m., and for the rest of the time the quantity may be equal to the average transmission of the previous day.

(2) For the three-branch problem, each of the branches having a reservoir, there is an example of control on an analogue instrumentation basis, through a control programme of the transmission quantity in combination with that of levels.

(3) Filtered water quantity is ordinarily controlled with regard to the level of the clear water tank, to which raw water procurement is made to conform. In some cases, the filtered flow is regulated through a cascade control mode on the level of the clear water tank.

4.6 Miscellaneous installations

The remainder of the installations within the purification plant are supervised and controlled in the following ways:

(1) Electrical power installations: sub-stations and power plants, which used to be operated separately, have become considered to be a link in the chain of total instrumentation.

(2) Mixers and scrapers etc.: information on operating conditions of mechanical flocculators, mixers, drainage valves, and other machines of this kind are often centralized, but rarely controlled from the centre. Travelling scrapers of rectangular sedimentation basins are commonly of the eight-wheel type and operated fully automatically including transferring over basins.

(3) Sludge disposal facilities: any drainage from a purification plant has recently been subject to the Water Pollution Prevention Law, as a kind of industrial waste. Sludge disposal facilities, being still at a phase of development and not yet technically consolidated, consist at present of thickening and dewatering processes of various kinds, and are in many cases installed and controlled separately from the main installations of the plant. The instrumentation on them is mostly confined to interlocking to avoid danger, though partially general process control and SQC are also included. When alum is recovered, special modifications become necessary to control the coagulant feeding system.

5 Water supply systems control

5.1 Purpose of water supply systems control

By "water supply systems control (WSSC)" the author understands the idea of optimal control applied to the operation of the waterworks process (incl. impounding, purification, detention, and transportation) to secure the quality of service (incl. quality, quantity, and pressure of distributed water) according to the given load (incl. outflow of river, raw water quality, and water demand). Practically, it would consist of (a) reservoir operation, (b) raw water apportionment, (c) transmission control, and (d) distribution control.

The concept of WSSC is based on the concept of making more efficient use of limited water sources for an equalized service with economical operation.

The more facilities the water supply system has, the more they become closely-related and mutually complementary, and from this point of view, WSSC corresponds to the recent tendency toward the wide area water supply system in Japan.

An attempt is being made currently to apply WSSC techniques to cases of large scale water supply systems. They seem to be, however, showing slower progress than anticipated, though proceeding steadily.

5.2 Prerequisites for water supply systems control

WSSC will be realized as follows:

(1) The process is controlled on the basis of an optimum solution derived from its characteristics expressed in a mathematical model, with the input data of the predicted load.

(2) Actual service situation is monitored and the deviation from the desired condition is fed-back for corrective action.

The exploratory method of treating the process as a black-box without knowing the characteristics could not be put into practical use in this field. Consequently, the following would be the prerequisites for realizing WSSC:

(1) mathematical model of waterworks process, from reservoir to distribution networks is built up,

(2) outflow of river, raw water quality, and water demand are predictable,

(3) suitable means to control the process are prepared,

(4) service situation in distribution networks can be measured or estimated,

(5) the objective function is defined, and

(6) centralized supervisory control systems covering the whole facilities are established.

5.3 Centralized supervisory control system

Several water supply systems are progressing toward centralization of supervisory and control functions, as a preparation for WSSC, of which some examples are given below.

Graphical expression of functioning situations within the distribution network will be possible as a part of the transmission and distribution control. As to its display equipment, the combination of a graphic display panel and cathode ray tubes (CRT) would be the most feasible. In particular, the latter or similar to it, which are more flexible in application, might become the main types used in this area.

(1) A municipal waterworks, under extension to supply an area of 540 km² at an elevation ranging from 0 to 320 m, divided into 130 supply districts with a capacity of 771 000 m³/d, through 4 intakes including a reservoir, 9 purification plants, 76 pumping stations, 125 service reservoirs, 2 tunnels of 28 km with 20 junction wells, and a clear water reception tank from the service utility, has been preparing a centralized supervisory control system as follows: field installation measurements will be relayed to 49 local stations through private lines, which will then be combined with the control centre, through 3 transmitting stations, on 400 MHz band multiplex multi-directional wireless telemetering and telecontrol gears. The information collected will contain about 1 400 data, such as levels of the reservoir, junction wells, service reservoirs, and pump suction wells, flow rates in raw and clear water conduits, positions of valves, operating situations of pumps and power sub-stations, etc. Approximately 300 units including valves, pumps, and some items of purification plants will be controlled directly from the centre. The data-processing system there, having operated partially, will eventually consist of 4 central processors (CPU) with internal memories of 32 to 64 kilo-words.

(2) A prefectural water provision service utility, supplying four member cities with clear water at 1 460 000 m³/d to 19 delivery points, is advancing WSSC for facilities including a reservoir, 3 purification plants, 3 pumping stations, and raw water tunnel of 30 km in length, making use of a full-scale hierarchical control system. Two large scale CPU with built-in memories of 327 and 128 kilo-words in semi-dual composition (ordinarily one for control and the other for a data bank), linked together by two units of mini-computers, have been introduced as the pivot of a central data-processing system. Installations in the field are connected to four local control stations through cleared telephone lines, in each of which are installed two mini-computers in duplex connection, as the digital process controller, working on DDC basis in the main.

The communications between the local and central stations are by a radio system on 400 MHz band similar to the foregoing, where the latter is being prepared to provide a series of information including desired values of raw water procurement, chemical dosages, and flow quantity apportionment in each plant at fixed times, and any others of the data bank by inquiry of the former.

5.4 Arrangement of distribution facilities

Well-arranged distribution facilities are essential for realizing WSSC; and the block-system seems to be considered as the most suitable form.

A municipal water supply system (area 405,6 km^2, topographic range from 0 to 100 m, domestic water supply of 1 900 000 m^3/d, industrial water supply of 360 000 m^3/d from four purification plants simultaneously, plus four points of incoming bulk supply) has been arranging the distribution network to establish a controllable system. The area is divided into 21 independent blocks, each of which has its own reservoir and a pumping station if needed, and the distribution trunk mains are installed, as appropriate, for either gravity or pumped supply to the block concerned.

5.5 Development of water supply systems control

The development of WSSC techniques are being carried out by academic experts and engineering staff of some of the more active waterworks utilities, sometimes in co-operation with consulting firms and related manufacturers.

The present state of their work is set out briefly below.

5.5.1 *Objectives.* There may be a number of different objectives of WSSC which may function independently or in combination, according to the circumstances of the waterworks utility concerned:

(1) Efficient use of water, e.g. minimum ineffective discharge from the reservoir, maximum water procurement from the river, or maximum levels retained in a series of distribution reservoirs.

(2) Economical operation, e.g. minimum operational cost of water purification or of distribution.

(3) Stable operation, e.g. minimum fluctuation of operating conditions with time, or minimum deviation of level in distribution reservoirs.

5.5.2 *Prediction of Outflow of River.* The outflow from a river is predicted on a hydrograph in terms of rainfall. A study has been carried out successfully applying a tank model to ascertain the proportions of virtual rainfall appearing as surface, intermediate, and subsoil outflows in order to obtain the impulse response by means of a correlation method with an adaptive control technique. Consideration is given to the characteristics of river basin and river channel with regard to floods, and melted snow must be taken into account to improve the accuracy of the forecast.

5.5.3 *Reservoir Operation and Raw Water Apportionment.* Dynamic programming (DP) has been tried in an attempt to optimize the operation of a reservoir based on the predicted inflow. A series of simulations showed that better water utilization would have been possible over the hydrologically planned limit of water procurement for the majority of the past years.

Another study is being carried out using linear programming (LP), on a waterworks complex having an intricate combination of reservoirs, hydro-electric power stations, direct intakes from river, canals, and purification plants as a transportation network problem to optimize the raw water apportionment. Although a dynamic model is needed to obtain a result on the basis of the short term prediction of water demand, such as daily or hourly, a succession of static models is applied here instead, to avoid an excessive calculating capacity.

5.5.4 *Prediction of Water Demand.* Water demand prediction is classified according to its period, such as long term (year-long) mainly for planning, medium term (monthly, weekly) for reservoir operation, and short term (daily, hourly) for transmission and distribution control.

(1) Annual demand prediction: annual demand prediction on a monthly basis is derived from past operational data (5 to 10 years), avoiding irregular variations, based on the tendency and seasonal movements by means of time series analysis. The cyclic variation is not noticeable here.

(2) Weekly demand prediction: the pattern of weekly variation of water demand on a daily basis is recognized to be fairly clear. Relating to irregular variations, influences of rainy days and holidays are noticeable, and a significant correlation is observed between the weekly demand and quantity supplied on the day of prediction. These factors will be used to forecast the weekly demand, and to assign it to weekdays.

(3) Daily demand prediction: an apparent correlation is distinguished between total daily consumption and partially delivered quantity of the day for the purpose of forecasting. Daily consumption is predicted accurately on the basis of water supplied in the previous week, by means of multivariate regression analysis, introducing such parameters as peculiar days (e.g. single holiday, consecutive holidays), climate (applying weather forecasting), special operation (e.g. restricted supply), temperature compensation, days of week, etc., given as constants, formulae, or statistics. Some problems are, however, still left to be solved before actual utilization.

(4) Hourly demand prediction: the prediction of daily demand on an hourly basis would be most essential for distribution control. It can be derived simply by adapting the above-mentioned methods. A more precise technique has been studied, by means of a grouping method, to obtain the hourly variation coefficient through classified distribution patterns with regard to the elements affecting them, e.g. the seasons including temperature, the climate according to the seasons, and the days of the week. Though a prediction of this kind must primarily reflect local features in the supply district, nevertheless it is very difficult to identify them by such a macroscopic analysis; a more precise investigation on the consumer's side would be required (13, 14). Besides the above, the Wiener filter and Carman filter methods are studied to predict irregular variations of water consumption.

5.5.5 *Prediction of Water Quality and Its Changes* (15, 16). At the present time, prediction of raw water quality, whether for long or short term, is at the stage of basic study and trial.

Treated water changes its quality in passing through distribution facilities, and also alters hydraulic properties of the pipes by making them tubercularized. The factors acting here, however, have not been as yet sufficiently identified. Distribution control to date has been chiefly hydraulic, and water quality changes, especially decline of residual chlorine and turbid water troubles, are handled on a case-by-case basis.

Study has shown that the water retention time within a pipe might have significance over a much wider range than anticipated, and might affect water quality more than normally understood. In this regard research is being advanced in directions such as obtaining the probability distribution of retention time in networks based on the pattern of service flow, or analysing the share of flow and contribution to quality change of the route taken in relation to any given point.

The modified network analysis technique would be used also to determine countermeasures against an accident within the distribution facilities.

5.5.6 *Transmission Control.* A study has been promoted to optimize a large scale transmission system which includes: 10 purification plants, 14 distribution pumping stations each with by-pass route and direct distribution outlet, 15 terminal delivery points, and 69 trunk mains, making use of a sequential unconstrained minimization technique by Fletcher-Powell's solution (FP-SUMT). All of the 184 constraints are of linear form, comprising 254 variables derived from 69 independent ones. Input data are the hourly demand prediction at every delivery point. Then, raw water quantities to be procured, routes at distribution stations, rates and directions of flows in mains, pressures at nodes and terminal points, and level variations in reservoirs are all solved.

Here also a serial combination of static models is adapted instead of a dynamic model.

5.5.7 *Distribution Control.* Some research on the distribution control system applied to a network without reservoir has indicated its possibilities.

With the given demand predicted at every outlet point, the manipulating variables for pressure controlling valves to realize optimal pressure distribution will be obtained first. Making the set of pressures the desired value, the manipulating variables are corrected by means of sensitivity analysis for the pressure deviation.

Distribution control of this kind would naturally require a vast calculating capacity.

As a technique resolving the difficulty on the one hand, and meeting with the block system on the other, a modification of a network analysis has been investigated, which, having divided the system into subnetworks, might realize the optimization of each block and then reconciliation through the whole, making use of the minimax method.

Some are of the opinion that "relevant control" or "satisfactory control" would be more suitable than "optimal control", which is a mathematical concept, for practical purpose, especially in such a problem as distribution control.

5.5.8 *Controlled Area of Networks and Fire Fighting Water.* For large scale networks, the attention of the distribution control would have to be focused on trunk mains (e.g. 1 000 mm and over) and mains (e.g. 400 mm and over), and the greater part of branch mains would be left to manual control as before, though appreciably facilitated by the improved information system.

Usually for the large water supply systems considered here, the hourly maximum distribution quantity is larger than the average hourly demand plus fire fighting requirements. In this regard, fire-fighting water in the branch mains would not always be of significance in distribution control in many cases, but might rather be a problem of planning and designing of networks.

6 Automatic remote meter-reading system (17)

Large scale water supply systems are more or less interested in automatic remote meter-reading systems, and there have been some applications, although not many, and these restricted mainly to housing complexes.

One municipal waterworks utility is now attempting to establish the system experimentally to a suburban new-town under development which when completed will have a population of 390 000 in an area of 30 km^2, supplied with water at 154 000 m^3/d through about 100 000 service meters.

The meter-reading system has been developed in co-operation with the telephone corporation and a city gas company. Each meter-reading signal, transmitted to the control centre by telephone line, is processed there as a monthly consumption, then sent to the computer centre of the utility as a file record in magnetic-tape form.

Automatic meter-reading has reached the stage where it is technically feasible but still has economic problems to be solved.

7 Summary

(1) Throughout the whole course of rapid development in the last thirty years, water supply systems in Japan have been introducing instrumentation positively in expectation of improved and more economical operation. In purification plants and pumping stations, design on an analogue basis has been almost perfected.

(2) The digital control technique, having been introduced as a data-logger, is expanding its application aiming at a higher level of control. CCS is rapidly advancing toward the hierarchy and distributed system.

(3) Though quantitative evaluation of instrumentation is not easy so far, yet it has brought not a few results; the possibility of a large scale structure, quick response, labour saving, economical operation, activeness for up-to-date technology, etc.

(4) Present instrumentation has revealed some problems; an incompatibility of correct waterworks technology with it, quality of instruments, danger of unforeseen accident, passive attitude of personnel concerned, and insufficient operator training on waterworks undertakings' side, etc.

(5) Being forced into extremely effective utilization of limited water sources, under conditions of increasing pollution, large water supply undertakings have become interested in WSSC. Some of them are positively developing the basic techniques to realize it, whilst arranging the centralized supervisory control system of the waterworks complex. This might be where the computer would show its real ability.

(6) It seems to be the time to reflect and assess the instrumentation hitherto, and consider the most adequate technology, paying attention to the human factors, for water supply systems.

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Contrôle automatique des grands réseaux de distribution d'eau

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1 Introduction

Le but du présent rapport est de résumer l'aspect général du contrôle automatique ou instrumentation des grands réseaux de distribution d'eau à l'heure actuelle.

Le contrôle automatique, cependent, reste un composant secondaire ou optionnel de l'ensemble des ouvrages de distribution d'eau. Son développement et sa mise en place dépendent largement des circonstances sociales du pays concerné. Ce rapport est donc limité à la situation actuelle au Japon, avec quelques commentaires au sujet des développements futurs.

Le "contrôle automatique" n'est pas toujours clairement défini. Quelquefois il recouvre presque la même idée que l'automation ou exploitation automatique, mais d'autres fois il est limité au sens étroit d'un contrôle par réaction (feedback) ou par anticipation (feedforward). L'auteur l'interprète dans son sens large en incluant les techniques et les appareillages utilisés pour surveiller, contrôler et intégrer les données (data processing) dans l'exploitation des réseaux de distribution d'eau. En un sens, il est synonyme d'*instrumentation* et il est proche du côté technique de l'*automation*. Plusieurs rapports des congrès de l'A.I.D.E. (1, 2, 3) ont déjà traité de l'utilisation de l'automation dans les distributions d'eau. Ici sa composition et sa terminologie sont arrangés pour convenir au sujet du rapport.

En ce qui concerne les *grands réseaux de distribution d'eau*, l'auteur pense à la fois aux 27 services d'eau existant au Japon qui ont une capacité nominale de 200 000 à 6 232 000 m³/j et aux 55 stations de traitement de l'eau dont le débit nominal va de 100 000 à 1 820 000 m³/j.

2 Généralités

2.1 Histoire de i'instrumentation dans les réseaux de distribution d'eau

Les réseaux de distribution d'eau se sont remarquablement développés au Japon depuis 30 ans, le volume d'eau potable passant de 2 à 12 milliards de m³ et la population desservie de 25,0 à 86,7% de 1950 à 1974.

Parallèlement à cette rapide expansion, l'instrumentation la plus actuelle à l'époque a été utilisée intensivement, spécialement dans les stations de traitement; c'était essentiellement celle des usines de l'industrie chimique qui pouvait s'y adapter, pour obtenir une exploitation meilleure et plus économique.

Les techniques modernes de contrôle ont été introduites dans les stations de pompage en 1950 pour permettre la surveillance par un seul homme; elles ont ensuite progressé vers la régulation des vitesses avec récupération des pertes de puissance, qui est largement utilisée à l'heure actuelle.

L'instrumentation à l'aide d'instruments analogiques et quelquefois avec des contrôleurs séquentiels, étant apparue en 1952, est devenue à peu près standard en dix ans dans les stations de traitement nouvelles. La technique de contrôle numérique a été d'abord introduite dans les stations en 1963 avec des enregistreurs de données. A présent, la majorité des grandes stations de traitement est équipée de systèmes de contrôle par ordinateur dont la fonction est principalement d'enregistrer les données, tout en développant étape par étape les opérations convenant à ce contrôle par ordinateur.

Plusieurs réseaux de distribution d'eau disposent d'un système de contrôle centralisé aidé par ordinateur en composition hiérarchique, en vue d'optimiser le contrôle des réseaux de distribution d'eau, ce qui, indubitablement, semble le courant principal de l'utilisation des ordinateurs dans le domaine du contrôle des distributions d'eau.

2.2 Les degrés d'instrumentation

On a défini (4) dix degrés d'instrumentation: A0 nul, A1 énergie, A2 dextérité, A3 diligence, A4 jugement, A5 évaluation, A6 faculté d'apprendre, A7 raisonnement, A8 créativité, A9 dominance.

Si l'on admet qu'une petite boucle de contrôle en retour correspond à A4, le contrôle séquentiel sera équivalent à A3, 5, le contrôle en retour combiné à A4, 5, le contrôle par anticipation à A5, le contrôle optimal à A6, le contrôle du réseau à A7 respectivement. Donc d'une façon générale, l'automation (ou instrumentation) jusqu'au niveau d'A5 a été maintenant largement mise en pratique, et en outre A6 et A7 sont en phase positive de recherche et de développement dans les distributions d'eau. Les techniques correspondantes semblent maintenant confrontées à une frontière de contrôle opérationnel et informatique, ou à un niveau instinctif et intellectuel de contrôle.

2.3 Evaluation de l'instrumentation

Il est difficile d'estimer quantitativement le mérite de l'instrumentation, mais on reconnaît généralement les facteurs suivants:

(1) On peut réaliser des structures concentrées à grande échelle correspondant aux besoins résultant des circonstances, ce qui autorise des économies de construction et d'exploitation.

(2) Une réponse plus rapide à un changement des conditions d'exploitation est devenue possible.

(3) Les quantités de travail manuel et de main-d'oeuvre nécessaires ont été considérablement réduites.

(4) Même si ce n'est que dans une mesure limitée, la consommation de réactifs et d'énergie a été appréciablement réduite.

(5) Une attitude active a été introduite dans le domaine des distributions d'eau pour améliorer les installations en utilisant les techniques les plus avancées.

D'un autre côté, l'instrumentation soulève divers problèmes:

(1) Les techniques de distribution d'eau qui ont été mises au point pour assurer la plus grande sécurité et régularité à la fourniture de l'eau avec le moins de contrôle ne semblent pas suffisamment adaptables aux stades courants de contrôle automatiques; cependant, les processus des distributions d'eau ont fait des progrès appréciables, de sorte que la contrôlabilité devrait être encore améliorée.

(2) La qualité actuelle des instruments, surtout les éléments détecteurs, n'est pas entièrement suffisante pour les besoins.

(3) La réaction du personnel des distributions d'eau à l'égard de l'instrumentation n'est pas toujours la même. Elle est généralement plus positive à la phase construction, qu'en exploitation, où une révolution technologique trop rapide provoque une certaine anxiété.

(4) Plus la technique instrumentale devient précise, plus elle se révèle sensible aux accidents imprévus, ce qui semble exiger certains moyens et personnels d'appui. Les ennuis semblent se déplacer d'une façon préoccupante de l'appareillage (hardware) aux procédures d'utilisation (software).

(5) Les modes de qualification et de formation du personnel, nouvelle entreprise, sont très insuffisants.

Il faut noter qu'il est récemment apparu dans le personnel des distributions d'eau japonaises une tendance à réfléchir sur l'accent excessif mis sur l'instrumentation et sur la confiance injustifiée qu'on lui accordait, et à reconsidérer la technologie la plus adéquate à adopter pour les réseaux de distribution.

Il faut également accorder une attention sérieuse à la nature humaine.

On estime que les dépenses pour l'instrumentation dans les stations de traitement nouvellement construites représentent 3 à 5 % des coûts de construction.

3 Eléments et techniques d'instrumentation

3.1 Eléments d'instrumentation

L'instrumentation comprend les éléments suivants: (a) détection; (b) affichage; (c) contrôle; (d) exploitation; (e) transmission. Parmi eux, dans la nouvelle organisation instrumentale avec contrôle par ordinateur, il serait préférable de disposer (b) comme interface homme-machine et (c) comme mise en oeuvre des données, software inclus.

Les composants fondamentaux des technique d'instrumentation sont classifiés ici de la façon suivante:

(1) Fonctions centralisées de supervision et de contrôle.

(2) Contrôle de processus, ce qui englobe le contrôle en retour avec des boucles mineures ou combinées en circuit fermé; comprenant le mode de contrôle à valeur constante, proportionnel, par programme et en cascade; et les actions de contrôle proportionnelle (P-), intégralc (I-), et différentielle (D-action).

(3) Contrôle séquentiel, basé sur une boucle ouverte; beaucoup de processus contrôlés à la station de traitement et à la station de pompage sont des assemblages organiques de composants en retour et séquentiels.

(4) Système de contrôle par ordinateur, comprenant le contrôle en ligne, le contrôle guide d'exploitation, le contrôle en point fixe et le contrôle numérique direct.

(5) Contrôle adaptif et optimal (y compris le contrôle de l'optimisation) basé sur le contrôle par anticipation.

(6) Télémétrie et télécontrôle.

(7) Contrôle de réseaux de distribution d'eau, par lequel l'auteur entend un contrôle d'optimisation appliqué à l'ensemble d'un complexe de distribution d'eau: en outre, un concept de systèmes de contrôle total d'une distribution d'eau serait défini comme une combinaison de contrôle du réseau avec un système de gestion de l'information, ce qui serait le point extrême en ce domaine.

3.2 Elément de signalisation et d'exploitation

Comme élément détecteur dans les services d'eau, on a beaucoup utilisé les signaux électriques et électroniques, ces derniers devant être normalisés dans la gamme des 4 à 20 mA en courant continu, d'après la recommandation du Comité Electrotechnique international (IEC). Sur la ligne d'exploitation, on emploie un mélange de signaux *électriques, électroniques, pneumatiques et* hydrauliques dont voici quelques exemples:

(1) Pour les petites vannes de commande, comme dans les distributeurs de réactifs, le système pneumatique prévaut toujours;

(2) Pour les tailles moyennes à action lente, comme le contrôle de l'effluent des filtres, on emploie de plus en plus des vannes à commande électrique, du type actionné par la vitesse, ce qui convient au contrôle par ordinateur.

(3) Pour les grandes tailles à action rapide ou précise comme le débit des pompes, on emploie souvent la commande hydraulique.

Dans un système de contrôle de la capacité d'une pompe, les engrenages qui commandent la vitesse, entre autres, sont souvent considérés dans l'agrégat comme l'élément actif.

3.3 Elément détecteur

3.3.1 *Objets détectés.* Les objets suivants sont habituellement détectables soit comme quantités mesurées, soit comme signaux de situation:

(1) Hydrauliques: débit, volume total ou vitesse dans les conduites ou les chenaux ouverts, niveau et pression, densité du liquide, etc.

(2) Qualité de l'eau et chimie: température, turbidité, couleur, alcalinité, conductivité électrique, pH, chlore résiduel, demande de chlore (à l'essai), oxygène dissous, concentration des liquides, etc.

(3) Météorologiques: température, humidité, vitesse et direction du vent, précipitations, etc.

(4) Mécaniques: ouverture de vannes et de robinets, vitesse de rotation des pompes et des agitateurs, température des paliers, situation des machines, débit, niveau et poids de matériaux pulvérulents et granuleux, etc.

(5) Electriques: tension, intensité, puissance, facteur de puissance, etc.

L'élément détecteur, souvent nommé senseur, est aussi appelé transmetteur sur le marché, en y comprenant le transducteur incorporé.

Les éléments détecteurs supplémentaires cités dans les paragraphes suivants sont limités à certaines applications spéciales, car leurs principes et structures sont communs au monde entier.

3.3.2 *Gros débits.*

La mesure de gros débits est naturellement importante dans les distributions d'eau, mais la précision de cette mesure offre toujours quelques difficultés.

Pour les appareils, outre les tubes Venturi, y compris les tubes bi-directionnels, on utilise de plus en plus le système électro-magnétique (jusqu'à 2 400 mm) et les ultra-sons (jusqu'à 2 700 mm). Le premier est plus précis et il est insensible à la forme de la conduite, tandis que le second peut être installé sur des conduites existantes sans égard au diamètre, et distingue le sens de l'écoulement. Ses courbes de réponse sont en voie d'amélioration.

Le débit en chenaux ouverts est mesuré par des détecteurs électro-magnétiques, ultrasoniques ou électrostatiques, directement ou avec utilisation d'un barrage.

C'est une règle fondamentale que de calibrer l'appareil de mesure en fonction des volumes réellement écoulés; les grandes débitmètres spécialement ont souvent besoin de ce calibrage sur place.

3.3.3 *Qualité de l'eau* (6, 7, 8).

Bien que la détection de la qualité de l'eau soit indubitablement la question la plus importante en matière d'instrumentation des distributions d'eau, elle reste toujours à l'heure actuelle la partie la plus difficile car

(1) Tous les détecteurs nécessaires n'ont pas été encore mis au point pour l'utilisation industrielle, bien que leur domaine d'application s'étende peu à peu, par ex. utilisation de la chromatographie gazeuse pour les mesures en exploitation.
(2) Certains des principes de détection diffèrent de ceux des normes de qualité.
(3) L'entretien exige généralement un travail considérable; la majorité de ces appareils sont délicats et compliqués et consomment une certaine quantité de réactifs.
(4) Lorsque les installations sont concentrées, les tubes de prélèvement d'échantillons peuvent altérer les qualités de l'eau.

3.3.4 *Quantités particulières.*

Des dispositifs de détection, par ex. pour suivre les quantités particulières, ont été réalisés et sont d'emploi pratique.

(1) Détecteur de rupture d'une conduite principale; également, un exemple singulier de vanne d'évacuation actionnée par un sismomètre pour un réservoir de distribution gravitaire.
(2) Détecteur de fuite de chlore gazeux.
(3) Détecteurs de substances nuisibles: les détecteurs de phénol et nappe de pétrole sont presque devenus pratiques. On place souvent, dans les conduites de contrôle, des réservoirs à élevage de poisson et des cloches pour détecter les odeurs.
(4) La surveillance de la qualité des rivières réalisée par les administrations chargées de leu gestion et de la lutte contre la pollution comprennent la détection de la DCO, de l'oxygène dissous et du carbone organique totaux, des cyanures, ammoniaque, cadmium, cuivre, mercure, en plus des détections ci-dessus.
(5) Niveau et concentration de la boue.

3.4 Dispositifs d'affichage et contrôle des opérations (Interface homme-machine)

Le coeur de la salle de contrôle est une série de tableaux comportant des indicateurs, divers dispositifs d'affichage et des contrôleurs d'opération, accompagnés d'une série de consoles de manoeuvre équipées d'interrupteurs commandant les machines, sous-stations, pompes et vannes.

Dans le cas de contrôle par ordinateur, l'unité centrale de traitement et les équipements périphériques sont établis dans la chambre de traitement des données, généralement voisine de la salle de contrôle; seules des machines à écrire d'enregistrement et une partie seulement des dispositifs d'entrée et de sortie sont placés dans cette salle.

Depuis peu, et surtout depuis l'apparition des ordinateurs, les entrées et sorties pour l'exploitation ont été appelés "interface homme/machine". C'est peut-être la partie qui a été le plus remarquablement transfigurée par les progrès de la technique de l'instrumentation. En outre:

(1) Les tableaux se sont transformés dans l'ordre en modèles simples, graphiques et semi-graphiques et cèdent maintenant leur rôle directeur aux consoles de commande. Le tableau semi-graphique est maintenant le plus employé en combinaison avec l'instrumentation analogique; il se prête aussi aux modifications souvent nécessaires dans les distributions d'eau. On emploie également le tableau en mosaïque. Depuis l'introduction des ordinateurs, le nombre et l'importance des instruments analogiques ont diminué. Certaines stations de traitement récentes à contrôle par ordinateur ont éliminé tous les instruments des salles de contrôle. Avec le contrôle par ordinateur, le tableau graphique est appelé à renaître comme dispositif auxiliaire d'affichage.
(2) En même temps, les consoles de commande se sont remarquablement développées, adoptant successivement les équipements de contrôle sélectif, les contrôleurs logiques pour le contrôle séquentiel, la surveillance par télévision industrielle, les dispositifs d'affichage, surtout les tubes à rayon cathodique, les dispositifs à entrée d'information et les contrôleurs d'opérations; elles sont devenues les consoles de l'exploitant, qui est maintenant le pivot de ce que l'on appelle l'interface homme-machine. La console de l'ingénieur peut aussi être placée à part.
(3) Pour ce qui est des contrôleurs d'opérations, le type indicateur de déviation à haute densité est appliqué de préférence avec les indicateurs enregistreurs ou indicateurs de point, dont certains sont modifiés pour le contrôle par ordinateur.
(4) Un tableau (ou console) de commande locale est placé près de chaque groupe d'unités commandées, par ex. distributeurs de réactifs, filtres et pompes; les composants en sont commandés individuellement avec un minimum de liaisons. Le tableau de commande local voit également ses fonctions se développer vers plus d'intelligence pour devenir un terminal dans la chaîne hiérarchique de l'ordinateur, étant relié à un microcalculateur ou microprocesseur.

3.5 Contrôle séquentiel

Le contrôleur séquentiel s'est développé en adoptant comme élément principal, dans l'ordre, le temporisateur tambour, le circuit à relais, le commutateur rotatif, le circuit semi-conducteur et les circuits intégrés, pour devenir enfin un contrôleur logique d'emploi général ou contrôleur numérique direct. Le programme séquentiel est donné par câblage, tableau à broches ou circuit à mémoire, à peu près dans cet ordre.

Les installations électriques et les pompes ont été contrôlées par un contrôle séquentiel spécialement mis au point qui comprend des relais spéciaux de protection intéressant la sécurité, la fiabilité et la rapidité de réponse, et qui ont tendance à adopter les techniques électroniques pour devenir simples, normalisés et intelligents.

3.6 Contrôle par ordinateur

La plus grande partie des stations de traitement à grande échelle sont équipées maintenant en ordinateurs, mais certaines stations récemments construites n'en comportent pas en vertu de l'opinion technique que ce serait prématuré.

La technique du contrôle numérique a d'abord été appliquée dans les stations de traitement pour économiser les instruments d'enregistrement des données et pour produire une meilleure information. Par la suite, ses vastes possibilités de traitement des données ont été utilisées en vue d'améliorer les situations d'exploitation, d'abord dans la station de traitement, puis dans tout le complexe du réseau de distribution.

Les applications des ordinateurs dans les services d'eau sont les suivantes:

(1) La majorité des ordinateurs servent à l'enregistrement des données; ils collectent les données de l'exploitation, les analysent et permettent d'étudier les techniques d'application à un contrôle plus avancé.

(2) Après un effort considérable pour mettre au point l'algorithme, le contrôle en point fixe et le contrôle numérique direct commencent tout juste à être employés pour certains processus de traitement, par ex. l'application des réactifs, la filtration et la répartition des débits.

(3) De plus en plus on demande aux ordinateurs de contrôler tout le réseau, emploi qui semble le plus prometteur.

A l'origine, le contrôle par ordinateur était basé sur le "centralisme", c'est-à-dire l'usage multiplex d'une unité centrale de traitement, où les liaisons dans les deux sens étaient essentielles; les câblages entre l'ordinateur et les appareils terminaux étaient donc très compliqués.

L'apparition de mini ordinateurs a ouvert la possibilité d'une commande répartie et hiérachisée, et cette nouvelle tendance sera rendue définitive par l'application pratique du micro traiteur de données et des techniques d'échange d'informations entre ordinateurs.

L'ordinateur peut donc être placé en ligne ou comme contrôleur d'exploitation, même sur des ouvrages anciens.

L'ordinateur a incontestablement de grandes capacités, mais on a un peu trop attendu de lui. Il progresse toujours, peut-être trop vite, ce qui fait que les auxiliaires, par ex. les senseurs et le software d'utilisation semblent être restés très en arrière.

Jusqu'à présent l'analogue, ou ordinateur hybride, n'a pas été appliqué comme composant d'un contrôle par ordinateur.

3.7 Contrôle automatique auto-commandé

La technique de contrôle automatique a son origine dans les dispositifs auto-commandés. En plus des applications populaires comme la vanne à flotteur, la vanne réductrice, etc. . . . beaucoup d'inventions ingénieuses ont été adaptées aux diverses opérations, comme l'emploi des réactifs dans les stations de traitement.

Les dispositifs de contrôle auto-commandés ne sont pas toujours faciles à mettre au point; ils doivent essentiellement prendre en considération l'opération elle-même. Ils ne sont pas très précis en exploitation, mais exigent peu d'entretien et il y a beaucoup d'endroits où ils peuvent être commodément adaptées. Il y a donc une tendance récente à remettre en valeur le système de contrôle auto-commandé, comme dans le cas des siphons automatiques pour les filtres.

3.8 Télémesure et télécontrôle

Les techniques de transmission de signaux sont (a) le sélecteur à liaison directe; (b) la transmission par câble; (c) la transmission par radio.

Le système (a) est utilisé pour les courtes distances et un grande nombre de signaux, comme le contrôle d'une sous-station ou d'une station de pompage depuis la salle de contrôle d'une usine. Pour les transmissions par câble (b), on utilise des lignes privées ou le réseau téléphonique public.

Pour les transmissions par radio d'un grand nombre de signaux, le système prévalent est le multiplex multidirectionnel dans la bande des 400 MHz employant une paire de longueurs d'onde, surtout pour le système en étoile.

Les signaux transmis sont généralement du type numérique cyclique avec un code pour la détection des erreurs.

3.9 Personnel et entretien

A l'heure actuelle, il n'y a pas de service d'eau qui ait engagé de spécialistes en instrumentation, et des ingénieurs du service sont un peu au hasard chargés d'étudier et de réaliser les réseaux. Les bureaux d'études et les fabricants participent souvent à certaines parties de ce travail.

Mais lorsque l'instrumentation est en place, le personnel du service d'eau est en règle générale profondément engagé dans son exploitation, son entretien, et son développement en coopération avec ces firmes.

L'exploitation des instruments, c'est-à-dire celle du procédé lui-même, et l'entretien journalier sont réalisés par des agents de service en trois équipes, et il y a tendance à assigner à certains agents supérieurs diverses opérations.

L'inspection systématique et l'entretien son réalisés à la fois par le corps d'entretien propre au service d'eau, avec ses ateliers, par des visites frequentes, et par les constructeurs une ou plusieurs fois par an.

3.10 Critères et normes d'instrumentation

Les deux manuels "Critères d'étude des ouvrages de distribution d'eau" et "Directives pour la gestion technique des distributions d'eau" de l'Association Japonaise des distributions d'eau consacrent chacun l'un de leurs neuf chapitres à l'instrumentation.

Les termes techniques et les symboles de l'instrumentation et du traitement des données sont définis par les normes industrielles japonaises (JIS).

Les formes, tailles, performances et les méthodes d'essai des instruments et autres dispositifs sont déterminés par les normes JIS ou par diverses normes des associations intéressées.

4 Instrumentation à la station de traitement et à l'usine de pompage

4.1 Serveillance et côntrole centralisés

Les fonctions de surveillance et du contrôle dans une grande usine de traitement sont généralement concentrées dans la salle de contrôle. Au cas où il y a plusieurs salles de contrôle, l'une d'entre elles est chargée du contrôle générale des autres.

Le nombre de variables collectées est grossièrement donné dans les exemples suivants:

Une station de traitement de capacité nominale 1 700 000 m^3/j reçoit sur son ordinateur 400 signaux analogiques, 84 codés, 16 pulsés et 104 échantillonnés; elle utilise 322 instruments analogiques et 295 compteurs électriques.

Une autre station de 140 000 m^3/j conçue en vue de simplifier les installations traite 102 données hydrauliques et chimiques et 48 électriques, reliées à 98 indicateurs comprenant des enregistreurs de tendance.

Pour les signaux d'alarme, en plus de l'alarme lourde classique (sonnerie suivie d'un arrêt d'urgence de l'opération), alarme légère (ronfleur), et appel (timbre), accompagnés chacun d'un clignotement de lampes, on utilise aussi l'annonce parlée.

4.2 Applications chimiques

4.2.1 *Manipulation et préparation des réactifs* La majorité des réactifs utilisés dans une grande station de traitement, tels que le sulfate d'alumine, le polychlorure d'alumine, la soude caustique, l'acide sulfurique, le silicate de soude, etc. ... sont sous forme liquide, transportés par camions citernes et stokés dans des réservoirs. On contrôle les niveaux.

Certaines stations reçoivent le chlore en vrac, des camions citernes livrant le chlore liquide à des réservoirs fixes. On surveille les pressions et les niveaux.

La chaux, le charbon actif en poudre, le bichromate de potassium et quelques adjuvants de coagulation comme l'alginate de soude sont généralement en poudre ou en grains et sont transportés en vrac, en containers ou en sacs et quelquefois stockés en silos contrôlés par le poids ou le niveau.

Les manipulations de réactifs, où un travail manuel est inévitable, sont le domaine le plus difficile de la planification de l'instrumentation.

La préparation de cuvées, par ex. la dissolution ou la neutralisation de réactifs, est souvent réalisée automatiquement par contrôle séquentiel.

4.2.2 *Dosage des réactifs.* Le contrôle proportionnel est le moyen le plus populaire et le plus efficace utilisé pour les doseurs de réactifs, que ce soit pour la commande par ordinateur, par point fixe ou par contrôle numérique direct.

La détermination automatique du dosage des réactifs devient peu à peu possible grâce à l'ordinateur, dont les méthodes sont utilisées des façons suivantes:

(1) Formule expérimentale ou table de référence, avec des paramètres tels que l'alcalinité et la turbidité, et quelquefois la température et le pH.

(2) Recherche de données historiques se rapportant à des situations semblables.

(3) Modèle mathématique basé sur une analyse théorique de la décantation.

(4) Décanteur pilote.

(5) Filtre pilote, spécialement pour le dosage des agents de filtration.

Parmi ces procédés, (1) à (3) se prêtent au contrôle par anticipation, alors que le contrôle en retour peut être lié à (4) et (5).

En-dehors des coagulants, on a utilisé jusqu'à présent les contrôles suivants:

(1) Contrôle en retour direct pour la chloration et la post-chloration, à la fois avec des dispositifs analogiques et avec un ordinateur.

(2) Contrôle de la préchloration en fonction du chlore résiduel dans l'eau décantée, en contrôle par anticipation, par estimation du décrément dans le décanteur, combiné avec un contrôle en retour.

(3) Contrôle de la post-chloration, basé de la même façon sur le décrément dans le réservoir d'eau filtrée basé sur les livraisons prévues.

(4) Dosage de la soude caustique pour régulariser le pH, contrôle en retour directement ou en combinaison avec un contrôle par anticipation selon le pH de l'eau filtrée.

On estime généralement que les processus d'alimentation en réactifs ont un temps de réponse assez long, qui doit être compensé par le mode de contrôle des données d'échantillonage, et qui se prête basiquement à l'emploi d'ordinateurs.

4.3 Filtration Rapide

4.3.1 *Contrôle du débit.* Le débit d'un filtre à sable, s'il n'est pas du type à siphon automatique, est généralement mesuré par un tube Venturi court, ou rarement par un débitmètre à ultra-sons. En certains cas où l'ordinateur intervient, il est dérivé de l'ouverture de la vanne de sortie sans aucun dispositif de détection du débit.

Comme appareil de contrôle du débit habituellement placé sur les conduites de sortie, on utilise des vannes papillons de commandes spéciales, une adaptation de la vanne de fermeture du type papillon étanche ou cône, ou une sorte de commande par siphon.

Le contrôle du débit se fait de la façon suivante:

(1) Le marche à débit constant est la plus utilisée; le débit choisi est imposé à un groupe de filtres simultanément, par ce qu'on appelle le "contrôleur maître". On emploi également le contrôle numérique direct.

(2) La filtration à vitesse décroissante est employée dans certaines stations pourvues d'un ordinateur. Le débit est contrôlé sur un groupe de bassins soit en positionnant simultanément toutes les vannes de sortie, soit en modifiant le nombre de filtres en service.

Le filtre automatique siphonné, à niveau d'eau séparé et à barrage de sortie a des caractères d'auto régulation qui font que le débit de sortie est égal au débit d'entrée aux changements de capacité près, et le débit d'entrée est contrôlé et réparti également sur chaque filtre en service. Il faut alors tenir compte de quelques restrictions pour les besoins en eau de lavage.

4.3.2 *Lavage des filtres.* Le lavage des filtres est généralement réalisé par contrôle séquentiel semi ou complètement automatique. Le contrôle semi automatique exige simplement un signal manuel de démarrage. En ce qui concerne la sélection automatique du filtre à laver, on utilise des limites telles que la durée de fonctionnement du filtre, la perte de charge, la turbidité de l'effluent et le débit (limite inférieure), mais la première mentionnée est la plus pratique et la plus couramment adoptée.

Le contrôle séquentiel est réalisé par un séquenceur qui est en principe un contrôleur logique, ou par contrôle numérique direct. Le démarrage lent à la remise en service est généralement inclus, mais le drainage de l'eau filtrée au début est rarement combiné dans la séquence.

La difficulté ici serait la fiabilité d'un grand nombre de commutateurs à limite ou à torque attachés aux vannes qui entourent le filtre, et dont chacun doit intervenir dans les circuits logiques du contrôle séquentiel.

4.4 Installations de pompage (10, 11, 12)

Le contrôle des pompes porte sur leur marche: démarrage et arrêt, et sur leur capacité en fonctionnement; ce dernier contrôle comprend deux phases dans son but et dans la technique pour y atteindre.

4.4.1 *Démarrage et arrêt des pompes.* Les installations de pompage, qui sont les machines les plus grosses du réseau de distribution, doivent être mises en route et arrêtées suivant une séquence prédéterminée pour des raisons de sécurité hydraulique, mécanique et électrique. C'est l'objet typique du contrôle séquentiel.

On emploie généralement le contrôle semi automatique par un seul homme pour les pompes principales d'eau brute et d'eau filtrée qui doivent être démarrées prudemment et rarement, tandis que les pompes moyennes ou petites, par ex. pour le lavage des filtres ou de dosage des réactifs, sont souvent à marche entièrement automatique.

Le dispositifs de commande des pompes doivent tenir compte des possibilités d'arrêt brusque et prévoir les actions nécessaires pour éviter les coups de bélier, telles que liaison avec des réservoirs antibélier à sens unique, ou fermeture programmée de la vanne de sortie, etc. . . . Le dispositif, surtout pour les pompes de surpression, doit également tenir compte de la pression ou du niveau côté de l'aspiration, pour éviter la cavitation. Sous ce rapport, il existe parfois des générateurs de fonction qui donnent les caractéristiques de cavitation de la pompe dans les boucles de contrôle.

4.4.2 *Contrôle de la capacité.* Le but du contrôle de la capacité de la pompe est de régler le débit fourni, la pression de sortie ou la pression (ou niveau) d'aspiration des façons suivantes:

Au cas où la sortie comporte un chenal ouvert ou une longue conduite, le contrôle par données échantillonnées devient nécessaire pour compenser le temps mort.

(1) La pompe de puisage d'eau brute est généralement contrôlée de façon que le niveau de sortie soit maintenu constant. Quand la capacité du bassin récepteur est grande, le débit qui en sort est souvent incorporé dans la boucle de contrôle (contrôle par anticipation). Si le bassin récepteur est situé à une certaine distance, le niveau de l'eau peut être calculé en tenant compte des caractéristiques de la conduite, au lieu d'une télémesure directe, en considération des fluctuations du niveau d'aspiration.

(2) Les pompes de distribution et de surpression devraient être, idéalement, contrôlées dans le cadre de la commande du réseau de distribution. Mais à l'heure actuelle on adopte le plus couramment pour la pression de refoulement ou le débit refoulé une valeur constante ou programmée. Au cas où la zone de distribution alimentée par une station de pompage est limitée pour des raisons pratiques, on adopte souvent un contrôle par pression constante au point le plus éloigné (contrôle par anticipation), basé sur la plus basse pression résiduelle dans un réseau de distribution ayant certaines courbes de caractéristiques hydrauliques qui peuvent être données, dans l'instrumentation analogique également, par un générateur de fonction.

(3) La pompe d'eau de lavage alimentant un réservoir surélevé et la pompe de drainage des eaux de lavage sales sont contrôlées de manière à maintenir le niveau plus haut ou plus bas que le point fixé en mettant en route plusieurs unités suivant un ordre de priorité. Les pompes à lavage en retour direct et à balayage de surface sont mises en action dans le cadre du contrôle séquentiel de la procédure de lavage des filtres; il faut quelquefois prendre des précautions pour éviter des démarrages trop fréquents.

4.4.3 *Techniques de contrôle de la capacité.* Les moyens de contrôle de la capacité de la pompe sont finalement de régler les caractéristiques de la charge (étranglement de la vanne de sortie) ou de la pompe (c'est-à-dire nombre en service, vitesse de rotation et angle des ailettes du rotor).

(1) Le réglage de l'angle des ailettes du rotor, bien qu'idéal en principe, n'est pas adapté jusqu'à présent aux pompes de distribution d'eau à vitesse spécifique lente, en raison de leur structure.

(2) L'étranglement de la vanne de sortie, moyen auxiliaire ou de secours pour les grosses pompes, est le dispositif le plus simple, mais il provoque une sensible perte d'énergie et amène parfois des bruits et vibrations gênants.

(3) La variation du nombre de pompes en marche est systématiquement adoptée au cas où on peut accepter d'avoir un certain nombre de pompes normalisées, tandis que la pression résiduelle à l'extrémité de la conduite peut varier largement, et que l'on peut prévoir une perte d'énergie relativement importante. En fait, étant donné que pour diversifier le risque, pour avoir des unités en réserve et à cause des limites des possibilités de fabrication, toute installation de pompage comprend plusieurs unités, la variation du nombre de pompes en marche est utilisée dans la plupart des stations de pompage.

(4) Le réglage de la vitesse de rotation, qui fait l'objet des paragraphes suivants, est le plus courant pour commander la capacité des grosses pompes des stations où elles sont peu nombreuses.

4.4.4 *Contrôle de la vitesse de rotation.* La plupart des procédés de contrôle de la vitesse de rotation sont basés sur la méthode de régulation par glissement, où une certaine perte d'énergie est inévitable.

Pour récupérer l'énergie de glissement importante émise dans le secondaire d'un moteur à induction, on a mis au point diverses méthodes de techniques d'exitation qui ont suivi les progrès des semi-conducteurs.

(1) Un accouplement à glissement variable, opérant à un glissement de (1–n), de quelque type qu'il soit, émettra sous forme de chaleur une perte d'énergie proportionnelle à (n^2–n^3), et pour le choix du type, les moyens de refroidissement

jouent un rôle important. Pour cette raison, l'accouplement hydraulique, auquel le refroidissement par eau s'adapte facilement, a un peu plus d'utilisations pour les pompes moyennes jusqu'à 400 kW, alors que le couplage magnétique, plus simple à manier, s'applique aux petites pompes et se trouve normalisé sur le marché.

(2) La régulation du rhéostat secondaire couplé avec un moteur à induction à rotor bobiné, qui a été à l'origine utilisé pour le démarrage, est largement utilisé pour ajuster le glissement pour des pompes jusqu'à 4 200 kW. Ayant une perte de puissance importante semblable à celle de l'accouplement à glissement, il est adopté pour les pompes à faible gamme de variation de vitesse, ou comme procédé auxiliaire. Comme rhéostat, le type liquide prévaut, en raison de la variation continue qu'il permet et du refroidissement à l'eau.

(3) L'inverseur moteur-générateur en cascade (système MA-Scherbius) a été appliqué à l'origine à l'excitation secondaire, jusqu'à 2 300 kW. La cascade thyristor-inverseur (système Scherbius statique) est ensuite apparue en 1968 et est devenue la moyen le plus courant de contrôle de la capacité des grosses pompes, jusqu'à 6 500 kW, ce qui est le record pour les pompes de distribution d'eau. Il existe aussi sur le marché des petites unités de ce genre, normalisées. L'inconvénient provenant d'une coupure instantanée d'électricité, seul défaut grave de ce système, est en cours d'amélioration.

(4) Le moteur à induction à cascade avec moteur à courant continu (système Kraemer), qui est l'une des méthodes agissant sur l'excitation secondaire, a été adopté pour la commande des pompes en 1964. Son record est de 6 200 kW, ce qui est la 2ème record, et son rendement est le meilleur de l'espèce. Il a en outre le mérite de ne pas exiger d'espace supplémentaire quand il est couplé avec une pompe à axe vertical. D'un autre côté, il y a certaines restrictions dues à la conception du moteur à courant continu.

(5) Le moteur à induction avec une cascade de moteurs sans commutateur où le moteur à courant continu est remplacé par un thyristor, a été mis au point par un service d'eau pour éviter l'inconvénient d'une coupure instantanée d'électricité, évitant un entretien coûteux. Il est employé depuis 1975 jusqu'à 1 900 kW.

(6) Le moteur alternatif à commutateur est aussi une espèce de méthode à excitation secondaire, qui n'exige comme auxiliaire qu'un régulateur de tension. On l'emploie pour des pompes moyennes et petites jusqu'à 650 kW.

(7) Le moteur sans collecteur a été généralement reconnu comme système de contrôle ne demandant pas d'entretien, d'un haut rendement et d'une grande gamme de vitesse, malgré son coût. On a mis au point dans ce but des moteurs synchrones et asynchrones avec thyristors qui sont utilisés pour les pompes moyennes et petites jusqu'à 1 000 kW.

(8) Le système Leonard à thyristor s'adapte également aux pompes, mais n'est pas encore très commun.

4.5 Répartition des débits

Les séries de débits à travers une station de traitement sont contrôlés, d'après la demande en eau filtrée, de façon que le niveau dans le réservoir à eau filtrée suive une courbe de variation préétablie, en maintenant chaque débit aussi constant que possible. Ordinairement, cette procédure peut être suivie à la main en raison de l'effet tampon de l'eau dans le réservoir, bien que l'on ait vu apparaître aussi les commandes automatiques suivantes:

(1) Si un réservoir de distribution reçoit également de l'eau d'une autre origine, la transmission du contrôle serait difficile si elle n'est basée que sur les variations de volume. Une méthode très pratique de contrôle en point fixe est utilisée avec succès: les mouvements de l'eau sont contrôlés de 1 à 5 h du matin de façon que le réservoir atteigne son niveau maximal à 6 h, et le reste du temps les volumes produits peuvent être conformes à la moyenne du jour précédent.

(2) Le problème des trois branches, chaque branche disposant d'un réservoir, a été dans un exemple résolu par une instrumentation analogique, par un contrôle programmé des volumes produits en combinaison avec les niveaux.

(3) Le débit d'eau filtrée est généralement contrôlé en fonction du niveau programmé du réservoir d'eau filtrée, auquel on adapte le puisage d'eau brute. En certains cas, le débit d'eau filtrée est réglé par un contrôle en cascade d'après le niveau du réservoir d'eau filtrée.

4.6 Installations diverses

Le reste des installations dans la station de traitement est surveillé et commandé de la façon suivante:

(1) Livraison de l'électricité: les sous-stations et usines électriques, que l'on avait l'habitude d'exploiter séparément, sont maintenant considérées comme un maillon dans la chaîne des ouvrages.

(2) Mélangeurs, racleurs, etc. . . .: les informations sur l'état de marche des floculateurs mécaniques, agitateurs, vannes de drainage et autres appareils de ce genre sont souvent centralisées, mais rarement commandées depuis le centre. Les racleurs mobiles des décanteurs rectangulaires sont généralement du type à huit roues et commandés complètement automatiquement, y compris le passage d'un bassin à l'autre.

(3) Traitement des boues. Toutes les boues des stations de traitement tombent maintenant sous le coup de la loi relative à la pollution de l'eau comme une eau usée industrielle. Les installations de traitement des boues, qui sont encore en phase de mise au point techniquement non encore parfaites, comprennent divers dispositifs d'épaississement et de déshydratation qui sont généralement indépendants de la station de traitement et contrôlés séparément. L'instrumentation s'y réduit généralement à des interverrouillages pour éviter les accidents, bien qu'on y trouve également un contrôle général partiel et séquentiel. Lorsqu'on récupère le sulfate d'alumine, des modifications sont nécessaires pour contrôler l'approvisionnement en coagulant.

5 Contrôle des réseaux de distribution d'eau

5.1 Buts du contrôle des réseaux de distribution

Par "Contrôle des réseaux de distribution d'eau" nous entendons le contrôle optimal appliqué à l'exploitation du réseau (ressources, traitement, réservoirs et transport) en vue d'assurer la qualité du service (qualité, quantité et pression de l'eau distribuée) et en tenant compte des bontraintes (débit de la rivière, qualité de l'eau brute, demande). Il comprend pratiquement (a) l'exploitation des réservoirs, (b) l'allocation d'eau brute, (c) le contrôle de l'adduction et (d) le contrôle de la distribution.

Le concept de contrôle des réseaux est basé sur un effort visant à rendre plus efficient l'emploi de ressources limitées pour un service équilibré avec une gestion économique.

Plus le réseau comporte d'ouvrages, pous ces ouvrages deviennent étroitement liés et mutuellement complémentaires et à ce point de vue, le contrôle des réseaux correspond à la tendance récente, au Japon, à créer des réseaux très étendus.

Une tentative a été récemment faite pour appliquer les techniques de contrôle à ces grands réseaux de distribution d'eau. Elles semblent cependant progresser plus lentement qu'il n'était prévu, bien que ces progrès soient continus.

5.2 Conditions pour le contrôle des réseaux

Ce contrôle sera réalisé de la façon suivante:

(1) L'exploitation est contrôlée sur la base d'une solution optimale dérivée des caractéristiques exprimées dans un modèle mathématique avec comme donnée d'entrée la charge prévue.

(2) La situation réelle du service est détectée et la déviation par rapport à la condition désirée est introduite en retour en vue de l'action corrective.

La méthode exploratoire traitant l'exploitation comme une boîte noire en ignorant ses caractéristiques n'a pas pu être mise en pratique en ce domaine. En conséquence, les conditions préalables pour mettre en oeuvre le contrôle des réseaux sont:

(1) on a construit un modèle mathématique des opérations sur le réseau, y compris l'alimentation du réseau par les réservoirs,
(2) le débit de la rivière, la qualité de l'eau brute et la demande en eau sont prévisibles,
(3) on a préparé les moyens convenables pour contrôler les opérations,
(4) on peut détecter ou estimer la situation en service dans le réseau,
(5) la fonction objectif est définie, et
(6) on a établi un système de surveillance centralisé couvrant l'ensemble des installations.

5.3 Réseau de surveillance centralisé

Plusieurs services de distribution progressent vers la centralisation des fonctions de surveillance et de contrôle, comme préparation du contrôle total du réseau. Nous en donnons ci-dessous des exemples.

L'expression graphique des situations en cours à l'intérieur du réseau de distribution pourra faire partie du contrôle de l'adduction et de la distribution. En ce qui concerne l'équipement indicateur, une combinaison de tableaux graphiques et de tubes cathodiques serait la plus réalisable. Les derniers surtout, ou tout dispositif similaire, étant plus souples d'emploi, pourraient devenir les plus utilisés en ce domaine.

(1) Un service d'eau municipal, en cours d'extension pour alimenter une zone de 540 km² dont l'altitude va de zéro à 320 m, divisée en 130 secteurs d'alimentation, capacité 771 000 m³/j, possédant quatre prises dont un barrage réservoir, 9 stations de traitement, 76 stations de pompage, 125 réservoirs de distribution, 2 tunnels de 28 km avec 20 puits de jonction, et recevant de l'eau potable d'un autre service, a préparé un réseau de contrôle centralisé dans lequel les installations seront regroupées par des lignes privées sur 49 stations locales, qui se combineront avec le centre de contrôle, par l'intermédiaire de 3 stations de transmission grâce à des émetteurs radio multiplex multidirectionnels de télémesure et télécommande. Les informations collectées porteront sur 1 400 données, telles que niveau du barrage, des puits de jonction, des réservoirs de distribution et des bâches d'aspiration des pompes, débit dans les conduites d'eau brute et d'eau filtrée, position des vannes, état de marche des pompes et dessous-stations électriques, etc. ... Le centre contrôlera directement 300 objets, vannes, pompes et certains appareils des stations de traitement. Le dispositif de traitement des données qui a partiellement fonctionné, comportera finalement 4 unités de traitement centrales avec des mémoires internes de 32 à 64 000 mots.

(2) Un service départemental de fourniture d'eau en gros, livrant 1 460 000 m³/j à quatre villes par 19 points de livraison met au point un réseau de contrôle hiérarchique à grande échelle de ses ouvrages qui comprennent un barrage réservoir, 3 stations de traitement, 3 stations de pompage et un tunnel d'adduction de 30 km de long. Deux grandes unités de traitement des données avec des mémoires intégrées de 327 000 et 128 000 mots en agencement semi-binaire (normalement un pour le contrôle et l'autre comme banque de données), reliées par deux mini-ordinateurs, seront le pivot de ce réseau centralisé de traitement des données. Les installations locales comprennent quatre stations de contrôle reliées par des lignes téléphoniques louées, pourvues chacune de deux mini-ordinateurs en liaison duplex, comme le contrôleur de traitement numérique qui travaille en contrôle numérique direct à la station principale. Les communications entre les stations locales et centrale sont assurées par un réseau radio dans la bande des 400 MHz comme le précédent. La station centrale est étudiée pour fournir une série d'informations qui comprennent les valeurs désirées de l'eau brute puisée, les doses de réactifs et la répartition du débit entre les usines à intervalles fixes, et toute donnée enregistrée dans la banque sur demande des stations locales.

5.4 Aménagement du réseau de distribution

Un bon aménagement du réseau de distribution est indispensable pour la réalisation du contrôle du réseau, et le bloc-système semble l'arrangement le plus favorable.

Un service de distribution d'eau couvrant une surface de 405,6 km², variant de 0 à 100 m d'altitude, qui distribue 1 900 000 m³/j d'eau potable et 360 000 m³/j d'eau industrielle à partir de 4 stations de traitement simultanément et de 4 points de réception d'eau fournie en gros, a aménagé son réseau de façon à établir un système contrôlable. La surface est divisée en 21 blocs indépendants, dont chacun a son propre réservoir et une station de pompage si nécessaire, les conduites principales étant établies de façon à alimenter soit par gravité, soit par pompage le bloc concerné.

5.5 Mise au point du contrôle des réseaux de distribution d'eau

La mise au point des techniques de contrôle des réseaux est réalisée par des experts académiques et par les ingénieurs de certains services d'eau actifs, quelquefois en coopération avec les bureaux d'étude et les industriels concernés.

Le stade actuel peut être à peu près décrit par les exemples ci-dessous:

5.5.1 *Fonction d'objectif.* Il peut y avoir différents points de vue pour fixer la fonction d'objectif, indépendamment ou en combinaison, suivant les circonstances:

(1) Utilisation efficace de l'eau, par ex. minimum de pertes par trop plein d'un réservoir, puisage en rivière maximal, ou maintien de niveaux maximaux dans une série de réservoirs de distribution.

(2) Exploitation économique, par ex. coût d'exploitation minimal du traitement ou de la distribution de l'eau.

(3) Exploitation stable, par ex. minimum de fluctuations des conditions d'exploitation avec le temps, ou déviation minimale du niveau dans les réservoirs de distribution fixée par le programme.

5.5.2 *Prédiction du débit d'une rivière.* Le débit d'une rivière est prédit sur un hydrogramme en terme de pluie. Une étude a été réalisée avec succès en utilisant un modèle en cuve de chacune des pluies virtuelles allouées aux débits de surface, intermédiaire et souterrain pour obtenir une réponse par impulsion au moyen de méthodes des corrélation avec une technique de contrôle adaptative. On tient compte des caractéristiques du bassin versant et du lit de la rivière en ce qui concerne les crues, et il faut prendre en compte la neige fondue pour améliorer l'exactitude des prévisions.

5.5.3 *Exploitation des barrages réservoirs et répartition des débits puisés.* On a essayé la programmation dynamique pour optimiser l'exploitation d'un barrage réservoir basé sur les apports prévus. Une série de simulations ont montré que l'on aurait pu, pour la majorité des années passées, mieux utiliser l'eau dans des limites de captage hydrologiquement planifiées.

Une autre étude est en cours sur la programmation linéaire en ce qui concerne un complexe d'ouvrages de distribution d'eau ayant une combinaison imbriquée de réservoirs, d'usines hydro-électriques, de prises directes en rivière, de canaux et de stations de traitement en vue d'optimiser la répartition de l'eau brute dans le réseau de transport.

Bien qu'il faille naturellement un modèle dynamique pour obtenir un résultat pour la prédiction de la demande à court terme, journellement ou horairement, on emploie ici à la place une succession de modèles statiques pour éviter de recourir à une capacité de calcul excessive.

5.5.4 *Prédiction de la demande en eau.* La demande en eau est distinguée suivant sa périodicité à long terme (année) pour la planification, moyen terme (mois, semaine) pour l'exploitation des réserves et court terme (jour, heure) pour le contrôle des apports et de la distribution.

(1) Prédiction de la demande annuelle. Cette demande, établie par mois, est basée sur les données d'exploitations passées (5 à 10 ans) en évitant les variations irrégulières et en tenant compte des tendances et des mouvements saisonniers par analyse de séries d'intervalles de temps. On ne note pas dans ce cadre de variations cycliques.

(2) Prédiction de la demande hebdomadaire: le cycle de variations journalières de la demande au cours de la semaine est assez clair. Les jours de pluie et les vacances introduisent des irrégularités remarquables, et l'on observe une corrélation significative entre la demande hebdomadaire et la quantité livrée au jour prédit. Ces facteurs sont applicables pour prévoir la demande hebdomadaire et pour l'assigner aux jours de la semaine.

(3) Prédiction de la demande journalière: pour cette prévision, on distingue une corrélation apparente entre la consommation journalière totale et les quantités partielles livrées dans la journée. On peut prévoir avec exactitude la consommation journalière en tenant compte des fournitures faites la semaine précédente, grâce à une analyse de régression à variables multiples, en introduisant des paramètres tels que le jour particulier (par ex. jour férié simple, succession de jours fériés), le climat (grâce aux prévisions du temps), les opérations spéciales (par ex. fournitures réduites), la compensation de température, le jour de la semaine donnés comme constante, formule ou statistique. Il reste encore cependant certains problèmes à résoudre avant l'utilisation effective.

(4) Prédiction de la demande horaire: la prédiction de la demande horaire dans la journée est la plus essentielle pour le contrôle de la distribution. Elle peut être simplement dérivée d'après les méthodes ci-dessus décrites. Une technique plus précise a été étudiée, par la méthode des faisceaux, en vue d'obtenir le coefficient de variations horaires par des schémas de distribution classifiés en tenant compte des éléments qui les affectent, par ex. les saisons et les températures qu'elles entraînent, le climat suivant les saisons, et les jours de la semaine. Les prédictions de ce genre doivent d'abord refléter les caractéristiques locales de la région desservie, mais il est très difficile de les identifier par une telle analyse macroscopique, car une étude plus précise du côté des consommateurs est nécessaire (13, 14). En outre, on étudie également la méthode du filtre de Wiener et du filtre de Carman pour prédire les irrégularités de la consommation en eau.

5.5.5 *Prédiction de la qualité de l'eau et de ses modifications* (15, 16). A l'heure actuelle, la prédiction de la qualité de l'eau brute, à long comme à court terme, est tout juste en phase d'étude préliminaire et d'essais.

L'eau potable change de qualité en circulant dans le réseau de distribution et modifie aussi les propriétés hydrauliques des conduites en les tuberculisant. Mais les facteurs qui interviennent en ces domaines ne sont pas encore suffisamment connus. Le contrôle de la distribution, jusqu'à présent, a été principalement hydraulique et les changements de qualité de l'eau, surtout la diminution du chlore résiduel et les inconvénients dus à la turbidité de l'eau sont réglés cas par cas.

Certaines études montrent que le temps de séjour de l'eau dans les conduites peut avoir une importance dans beaucoup plus de domaines qu'on ne l'imaginait, et pourrait affecter la qualité de l'eau plus qu'on ne le croyait jusqu'ici. A cet égard, des recherches sont en cours en diverses directions, par ex. pour obtenir la distribution probable du temps de séjour dans les réseaux compte-tenu des débits en service, ou pour analyser la part dans le débit et la contribution aux modifications de qualité du trajet relatif à tout point donné.

La technique modifiée d'analyse du réseau permettra également de contrebattre un accident dans le réseau de distribution.

5.5.6 *Contrôle de l'adduction.* D'une façon générale, l'analyse de réseau à utiliser pour le contrôle de l'adduction et de la distribution n'est possible qu'en respectant les conditions de continuité et de fermeture des pertes de charge, spécialement s'il y a des réservoirs; elle doit être résolue comme un problème extrême sous une fonction objective.

Une étude est en cours pour l'optimisation d'un grand réseau d'adduction comprenant 10 stations de traitement, 14 stations de pompage de distribution possédant chacune un by-pass et une sortie de distribution directe, 15 terminaux de livraison et 69 conduites principales. Elle s'appuie sur la technique de minimisation non contrainte par la solution de Fletcher-Powell. Les 184 contraintes sont toutes linéaires et comprennent 254 variables dérivées de 69 variables indépendantes. Les données d'entrée sont la prédiction de la demande journalière à chacun des points de livraison; les quantités d'eau brute à puiser, les cheminements vers les stations de distribution, les débits et les sens d'écoulement dans les conduites, les pressions aux noeuds et aux terminaux et les variations de niveau dans les réservoirs sont tous résolus.

Là aussi on a adopté une combinaison sérielle de modèles statiques au lieu d'un modèle dynamique.

5.5.7 *Contrôle de la distribution.* Certaines recherches sur le contrôle de la distribution dans un réseau sans réservoir ont montré ses possibilités.

Connaissant d'avance la demande à chaque point de prélèvement, on obtiendra d'abord les variables de manipulation pour les dispositifs de contrôle de la pression en vue de réaliser une distribution optimale des pressions. En faisant de la série de pressions la valeur désirée, on corrige les variables de manipulation, sous une fonction d'objectif propre, à l'aide d'une analyse de sensibilité pour la déviation de la pression.

Un contrôle de distribution de ce genre exige naturellement une grande capacité de calcul.

Comme technique de résolution de cette difficulté d'une part, et pour l'adapter au bloc-système d'autre part, on a étudié une modification de l'analyse du réseau qui, ayant divisé le réseau en sous-réseaux, permet l'optimisation de chaque bloc et la conciliation de l'ensemble, grâce à la méthode mini-max.

Certains pensent que l'idée de "contrôle convenable" ou "contrôle satisfaisant" conviendrait mieux que celle de "contrôle optimal", concept mathématique, pour des raisons pratiques, surtout dans un problème tel que le contrôle de la distribution.

5.5.8 *Aire contrôlée des réseaux et besoins d'incendie.* Pour les réseaux très étendus, l'attention du contrôle de la distribution sera focalisée sur les grosses conduites (par ex. 1 000 mm et plus) et les conduites principales (par ex. 400 mm et plus), la plus grande partie des conduites secondaires étant contrôlée manuellement comme auparavant, travail appréciablement facilité par l'amélioration du réseau d'information.

Généralement dans les grands réseaux considérés ici, les volumes maximaux disponibles pour la distribution sont supérieurs à la demande moyenne horaire plus les besoins d'incendie. A cet égard, l'eau pour les besoins d'incendie qui se trouve dans les conduites secondaires n'aura souvent pas de signification pour le contrôle de la distribution; ce sera plutôt un problème de planification et de conception du réseau.

6 Réseau automatique de lecture des compteurs (17)

Les grands réseaux de distribution d'eau sont plus ou moins intéressés par la lecture automatique à distance des compteurs, et il y en a eu quelques exemples d'application, assez rares, surtout dans des complexes de logements.

Un service d'eau municipal essaie actuellement d'établir un tel réseau de lecture dans une ville nouvelle suburbaine en cours de réalisation qui comptera 390 000 hab. sur 30 km^2, consommant 154 000 m^3/j et comptant 100 000 compteurs quand elle sera achevée.

Le réseau de lecture correspond à celui que le service a mis au point en coopération avec le service du téléphone et une société de distribution de gaz.

Chaque signal de lecture de compteur, transmis par téléphone au centre de contrôle, est traité pour déduire la consommation du mois, puis envoyé à l'ordinateur sous forme de bande magnétique archivable.

La lecture automatique des compteurs en est au point où elle est possible techniquement, mais elle pose encore des problèmes économiques.

7 Conclusion

(1) Dans le cours d'un remarquable développement en trente ans, les réseaux de distribution d'eau Japonais ont mis en place une instrumentation dans l'espoir d'améliorer et de rendre plus économique l'exploitation. Dans les stations de traitement et de pompage, la conception sur une base analogique est devenue presque standard.

(2) Le contrôle numérique, introduit pour la collecte des données, étend ses applications et vise à un plus haut degré de contrôle. Le contrôle par ordinateur progresse à travers la hiérarchie et dans le réseau de distribution.

(3) Bien qu'il soit encore difficile d'apprécier quantitativement l'instrumentation, celle-ci a cependant apporté un certain nombre de résultats: possibilité de structures à grande échelle, réponse rapide, économie de main-d'oeuvre, économie d'exploitation, appel à une technique avancée, etc. . . .

(4) L'instrumentation actuelle a révélé certains problèmes: non conformité avec la technologie propre aux distributions d'eau, qualité des instruments, dangers d'accidents imprévus, attitude passive du personnel concerné, formation insuffisante des exploitants du côté du service d'eau, etc. . . .

(5) Etant contraints à une utilisation extrêmement efficace de ressources en eau limitées, dont la pollution va croissant, les grands services d'eau s'intéressent au contrôle des réseaux. Certains d'entre eux développent effectivement les techniques pour y parvenir, tout en aménageant un système de surveillance centralisé du complexe des ouvrages de leur réseau. Il est possible que ce soit là que l'ordinateur montrera ses capacités réelles.

(6) Il semble qu'il soit temps de la réflexion: vérifier le résultats déjà donnés par l'instrumentation, considérer la technologie la plus adéquate, faire attention au facteur humain, pour les réseaux de distribution d'eau.

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Traitment des boues

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Le problème des boues

Les besoins sans cesse croissants en eau potable et industrielle ont obligé les sociétés de distribution de chercher les sources nécessaires: en général l'on a exploité les réserves d'eau souterraines au maximum; quand ses réserves étaient épuisées l'on a utilisé les eaux de surface.

Normalement ces eaux de surface contiennent une certaine quantité de matières en suspension.

Dans tous les procédés de traitement ces matières en suspension sont enlevées et concentrées dans les eaux résiduaires de la station.

particulièrement pour les problèmes de la qualité des eaux de surface. Le but à atteindre serait la production d'eau potable sans ajout de produits chimiques. La réalité est toute autre: Sans cesse les distributeurs d'eau deviennent de jour en jour de meilleurs clients de l'industrie chimique (ref. 1). (fig. 1, fig. 2).

Entre 1970 et 1980 l'on attend aux Etats-Unis un accroissement annuel de la production de coagulents de 4,6%, ce qui correspond à une augmentation du prix de vente de 7,6%. Cet accroissement est dû, entre autres, à la nécessité de garantir la production d'une eau potable de qualité.

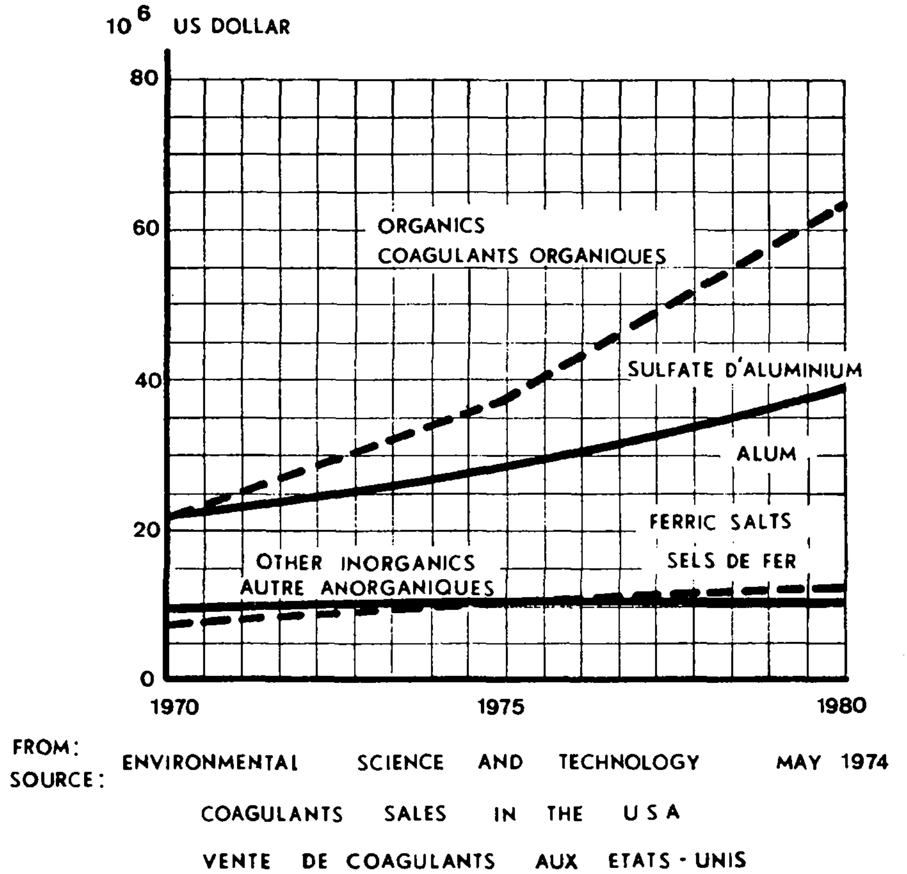


Figure 1

Selon le degré de difficulté du traitement nécessaire l'on doit faire appel à toutes sortes de systêmes pour obtenir la qualité requise: les traitements chimiques appliqués ont comme conséquence que, outre les matières en suspension précitées, l'on retrouve dans les eaux résiduaires de grandes quantités de sels d'aluminium ou de fer.

Actuellement l'opinion publique est fortement sensibilisée pour les questions d'environnement et plus

Le résultat de cette évolution est évidemment une production accrue d'eaux résiduaires.

Lors de l'élaboration d'un projet de station d'épuration le traitement de ces eaux résiduaires forme un problème supplémentaire presqu'inconnu dans le passé. Très souvent aussi ce problème est négligé, une solution judicieuse n'étant pas trouvée.

Pourtant deux principes de base généralement acceptés pour l'activité humaine nous invitent expressé-

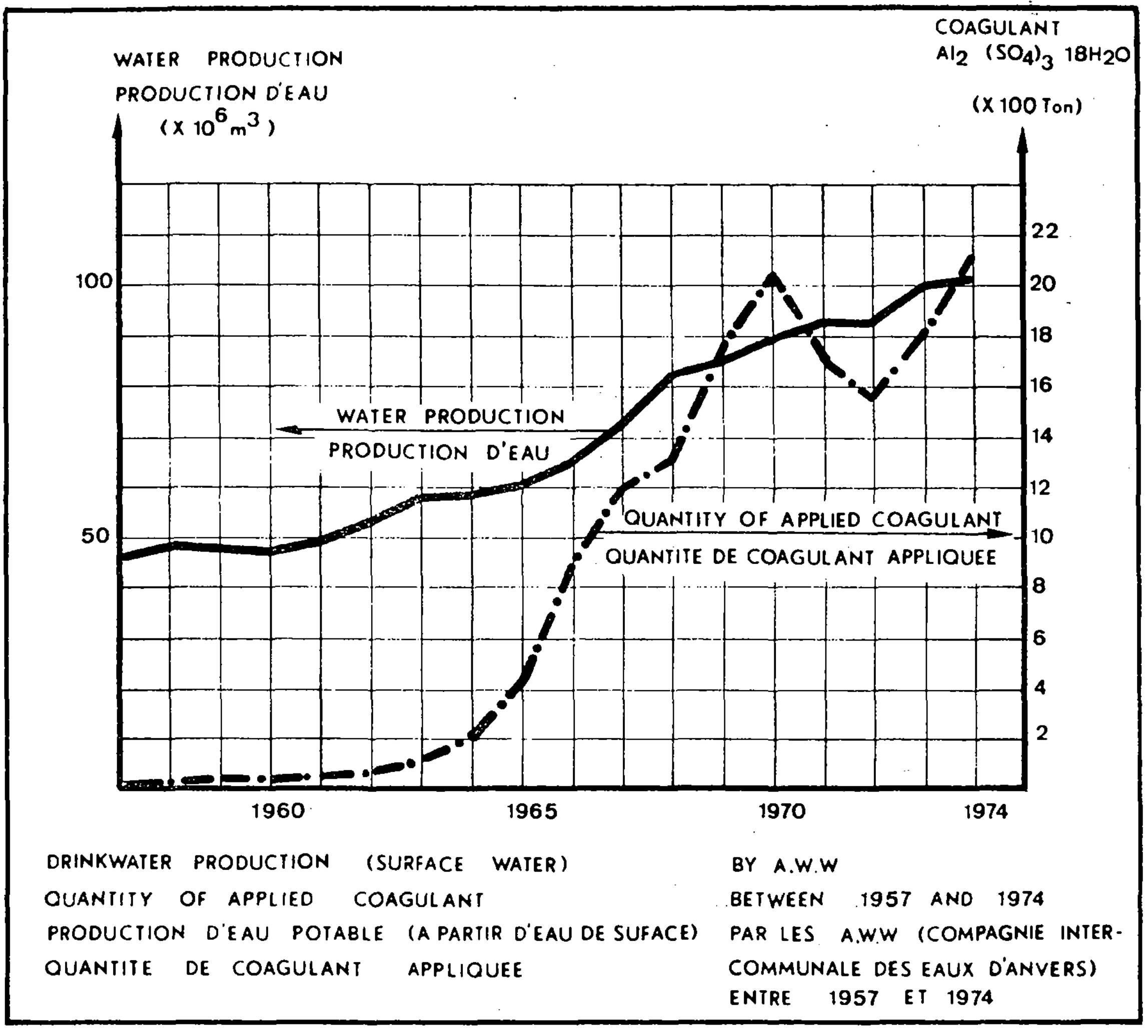


Figure 2

ment à trouver une solution au problème des eaux résiduaires.

1. Nos activités ne peuvent pas perturber notre environnement. Il n'est pourtant pas aisé de déterminer si des modifications dans l'environnement doivent être attribuées à l'intervention humaine ou à l'évolution de la nature même.

   Nous sommes généralement enclins à accepter que le monde forme un ensemble statique, parce que cette idée simplifie notre façon de voir: nous devons être conscients que la nature est en équilibre dynamique, des phénomènes de toutes sortes, indépendant de l'intervention humaine, réalisant l'évolution de cet équilibre.

   Ainsi par exemple en 1950 la disparition des sardines le long de la côte méridionale californienne a été attribuée aux activités humaines.

   Les eaux anaérobies au fond du bassin de Santa Barbara contenant peu de vie biologique, les couches de sediments se sont superposés, qui permettent de se former une idée de la vie aquatique telle qu'elle s'est présentée au cours de siècles. L'on a pu démontrer que les sardines ont disparu et sont revenues plusieurs fois avant la présence de l'homme dans ces parages (ref. 2).

   Il s'agit donc, de se former une opinion bien nette des évolutions naturelles et d'y superposer l'influence des activités humaines.

2. Les déchets resultant inévitablement de nos activités doivent être réutilisés sous d'autres formes.

   Ce second principe découle essentiellement du premier.

Nature de nos eaux résiduaires

Les caractéristiques de ces eaux dépendent de ses composants.

(a) Les matières présentes dans l'eau brute et éliminées par le traitement: principalement nous y retrouvons des matériaux inertes (sable, argile) et des matières organiques dissoutes ou en suspension (e.a. algues, bactéries et autres microorganismes).
La composition de ces matières est fortement tributaire des saisons. Les poussées d'algues, les précipitations, les phénomènes de stratification dûs aux variations de température provoquent une évolution continue de la composition.

(b) Les floculants et cofloculants (sulfates d'alumine ou de fer, etc.) continuellement adaptés à la composition de l'eau.

(c) De grandes quantités d'eau (plus de 99%).

Le rejet de ces eaux résiduaires à la rivière sans traitement préalable est encore appliqué dans la plupart des cas. Cette façon de faire peut avoir des conséquences néfastes pour la vie biologique et perturber gravement la bonne marche des stations d'épuration en aval.

Une attention particulière doit être donnée aux matières en suspension, qui peuvent former des dépôts dans le lit de la rivière.

D'autres éléments importants sont: l'acidité, les matières en solution, les coliformes, les chlorures, les métaux lourds, le COD et BOD (ref. 24).

Ces éléments influencent la rivière aussi bien au point de vue esthétique que biologique.

Réagissant contre les abus illimités des rejets d'eaux résiduaires dans les cours d'eau, l'on est enclin de supprimer toutes les autorisations de décharges.

C'est pourquoi l'on a cherché durant la dernière décennie des solutions alternatives pour évacuer les eaux résiduaires.

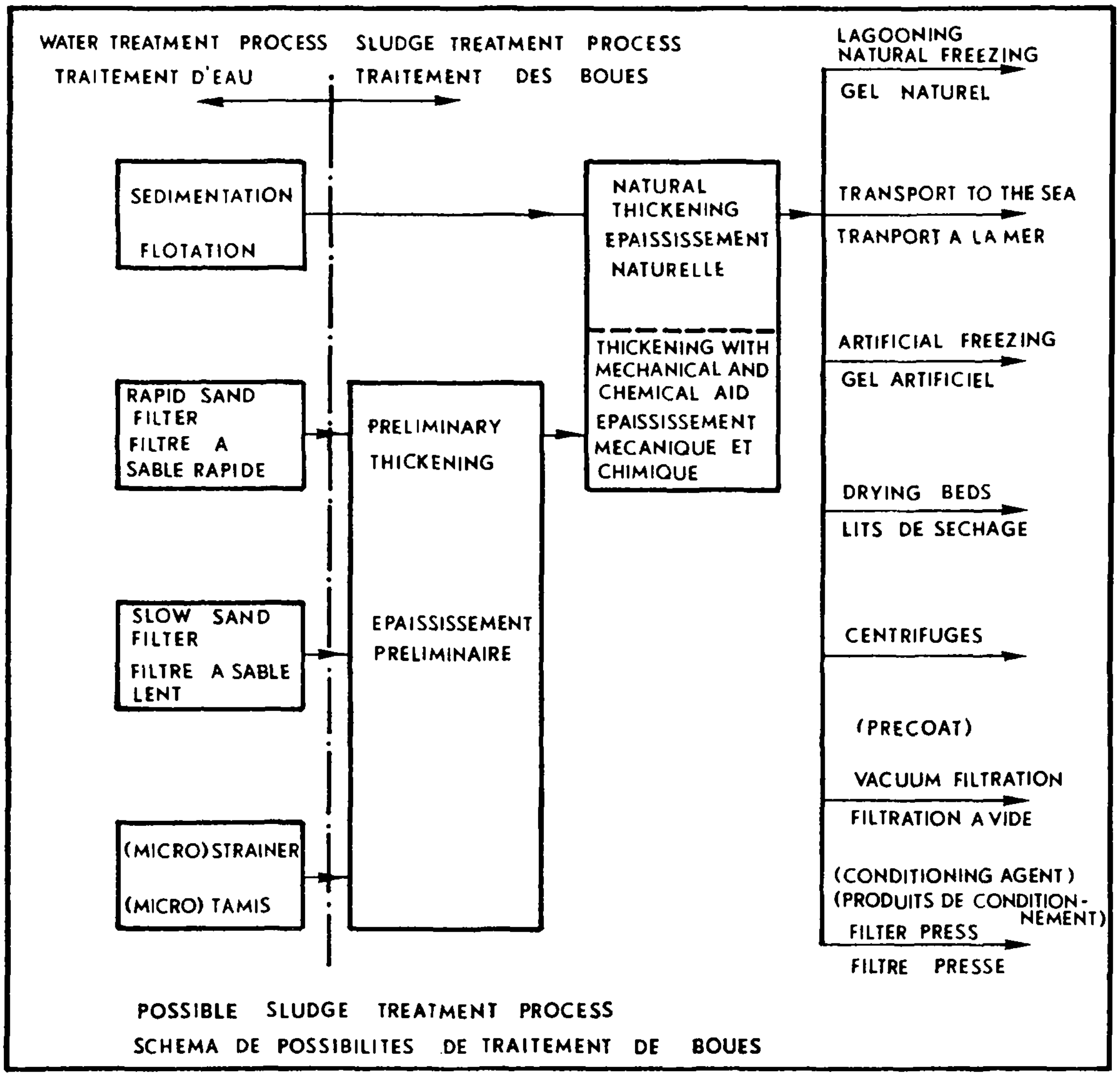
Méthodes d'évacuation des eaux résiduaires

Le choix judicieux de l'endroit de rejet dépend de la faune maritime, de l'orientation des vents dominants et des courants dus aux marées (ref. 2, ref. 9).

Les mêmes considération sont de mise lorsque l'installation se trouve le long d'une grande rivière.

Dans les deux cas l'influence des sédiments véhiculés naturellement par le fleuve prédomine largement celle due à n'importe quelle station de traitement.

Si dans les environs se trouve une station de traitement d'eaux résiduaires urbaine, l'on peut éventuellement envisager de commun accord l'évacuation des boues vers cette station (ref. 21).



Fignre 3

A Méthodes simples

Lors de la recherche d'une solution adéquate le climat et le site, dans lequel l'installation est implantée, doivent être considérés comme paramètres importants.

Si l'on dispose de grand domaines de moindre valeur dans une région peu peuplée, l'on peut prendre en considération la construction de grands bassins de décantation. Les matières en suspension décantent et l'eau claire à la surface peut être évacuée à l'égout ou rejetée dans la rivière.

Si le climat est rigoureux en hiver l'on peut laisser geler les eaux résiduaires en couches minces: lors du dégel le volume des sédiments est tellement faible que de tels réservoirs peuvent être utilisés pendant plusieurs décades sans obligation d'évacuation des boues (ref. 16).

Si la distance jusqu'à la mer n'est pas trop importante l'on peut y décharger les boues.

B Méthodes mécaniques

Les méthodes précitées, conditionnées par le site et le climat, ne sont en général appliquables que pour très peu des stations d'épuration.

Généralement l'on est confronté avec le problème de trouver rapidement une solution, dans un volume réduit, à faible prix de revient.

Dans une station de traitement d'eau de surface l'on retrouve habituellement trois centres importants de production de boues:

1. *Les décanteurs.*
   Ici la plus grande partie des matières en suspension, des fractions des matières en solution et les floculants se retrouvent dans les boues, qui contiennent de 5 à 10 kg de matières solides par m^3.

2. *Les filtres rapides.*
Les eaux de lavage contiennent environ 0,2 kg de matières en suspension par m³.

3. *Les tamis et micro-tamis au droit des prises d'eau.*
Leur apport dépend essentiellement des dimensions des matières en suspension et des mailles des tamis.

Plus de 90% de matières solides se trouvant dans les boues proviennent des décanteurs. Cet apport est continu. Par contre les filtres ne sont lavés que par intermittence avec des debits de l'ordre de 50 m³/h par m² de surface filtrante.

En égard à la grande différence de la teneur en matières en suspension, et puisqu'il faut tendre à diminuer les volumes de boues, il est indiqué d'épaissir séparément les eaux de lavage. L'on peut utiliser des bassins de décantation munis d'un trop plein et de fosses à boues; ces boues étant évacuées au moyen de pompes à membranes.

La durée de fonctionnement de ces pompes est déterminée par l'apport moyen de boues, les dimensions des fosses, les caractéristiques des pompes et le degré d'épaississement obtenable. L'on peut également amener les eaux de rinçage des tamis des prises d'eau dans ces bassins de décantation.

Après décantation des eaux de lavage, les boues épaissies ainsi que les purges des bassins de décantation sont mélangées.

A cet endroit il est opportun d'installer des déversoirs triangulaires pour contrôler séparément les deux débits.

Quelque soit le système ultérieur de traitement l'on a toujours intérêt à diminuer au préalable autant que possible le volume des boues. Cette diminution a toujours une influence favorable sur les frais d'investissement de l'appareillage de déshydratation et, quoique dans une moindre mesure, sur les frais d'exploitation.

Un appareil qui convient particulièrement pour l'épaississement des boues est l'épaississeur cylindrique muni d'une grille à axe vertical et à rotation très lente. La grille est composée de barres rondes verticales. Des systèmes racleurs concentrent les boues épaissies dans le fond tronconique de l'épaississeur (ref. 8).

Avec ce système il est possible d'obtenir des concentration de boues de l'ordre de 4%, c. à d. 40 kg de matières solides par m³ à un débit d'eau boueuse de 0,1 m³ par m³ d'épaisseur. Eventuellement l'on peut ajouter un adjuvant d'épaississement. Un épaississeur bien conçu ne demande que peu d'entretien, l'usure étant réduit au minimum.

Il est utile de signaler que l'utilisation de la flottation comme système d'élimination des matières en suspension donne lieu à la formation d'écumes dont la teneur en matières solides peut atteindre 8%.

De cette façon l'on réalise un important épaississement des boues durant le procès même de traitement d'eau.

La technique de flottation peut s'imposer quand l'eau brute donne lieu à la formation de petits flocs légers, qui remontent à la surface par adsorption de très petites bulles d'air.

Ceci est le cas quand l'eau brute est captée dans de grands réservoirs et en présence d'algues.

Après ce stade le traitement des boues peut se faire de différentes façons.

1. L'épaississement déjà cité dans de grands bassins ouverts. Le procès ne se réalise que très lentement, parce que les boues sont en général très hygrophile: même après plusieurs années les boues concentrées ne sont pas transportables, la concentration restant inférieure à 10% alors qu'il faut au moins 20% pour que les boues soient pelletables.

2. Le gel artificiel. La température doit être inférieure à −15°C parce que le point de gel de la boue épaissie, riche en sels dissous, est inférieur à celui de l'eau. De plus le procès doit se réaliser assez lentement, le risque de formation de particules miniscules à décantation lente n'étant pas négligeable. Si un épaississement préalable est nécessaire pour diminuer le volume de boues à geler, l'utilisation de chaux comme adjuvant n'est pas indiquée. Ce produit à une influence néfaste sur le procès.
Avec ce procédé l'on peut obtenir de façon irréversible des concentrations de 30 à 45%.
Il est évident que si les conditions climatiques sont favorables le gel naturel dans les bassins ouverts précités permet d'obtenir les mêmes résultats.

3. Les lits de séchage et les centrifuges ont ceci de commun que la concentration obtenue ne dépasse par les 20%.
Les lits de séchage ont comme désavantages d'utiliser de grandes surfaces et de nécessiter beaucoup de main-d'oeuvre.
Les centrifuges n'ont pas ces défauts et peuvent éventuellement être combinées avec le gel artificiel.

4. De meilleurs résultats sont obtenus avec les filtres sous vide. L'utilisation d'un precoat ou le dosage de chaux permet d'obtenir des concentrations de 30%.
Pour tous les systèmes précités à l'exception du gel artificiel les boues sont remises en suspension par contact avec l'eau; la formation de dépôt par stockage s'avère difficile.

5. Quoique l'application de filtres-presses a comme désavantage spécifique la discontinuité du procès, et le manque de posibilité d'automatisation complète, les résultats obtenus avec ce système sont comparables a ceux atteints avec le procédé du gel artificiel.
Un conditionnement adéquat préalable des boues permet d'obtenir un filtrat clair et une concentration dc 45% dans lcs gâteaux.
Le dosage d'un cofloculent n'est pas toujours favorable (ref. 4).
Puisque la technologie des filtres presses semble plus simple que celle des procédé par gel artificiel, son application semble plus indiquée, le rendement et les frais étant comparables.
Une automatisation partielle est possible et les gâteaux sont deshydratée de façon irréversible. (fig. 4).

C Destination finale des déchets

Même pour les meilleurs systêmes tels que les filtres presses et le procédé du gel artificiel le problème de la destination finale des gâteaux ou des sédiments reste posé. Dans les meilleurs conditions, les gâteaux contiennent encore 50% d'eau.

L'utilisation comme remblai de terrains, de carrières etc. semble à l'heure actuelle la meilleure destination.

Cependant il faut avoir la certitude:

1. que l'eau incluse dans les pores des gâteaux puisse s'évaporer à l'atmosphère;
2. qu'au contact avec l'eau de pluie les boues ne se remettent pas en suspension;
3. que le site ne subit pas des effets nuisibles dus aux matières déposées.

On pourrait même envisager d'utiliser des gâteaux conditionnés à la chaux pour le traitement de sols acides.

Le problème économique se ramène dans ce cas à un problème de transport.

Le compostage n'est possible que si les gâteaux contiennent suffisamment de matières organiques, ce qui n'est généralement pas le cas.

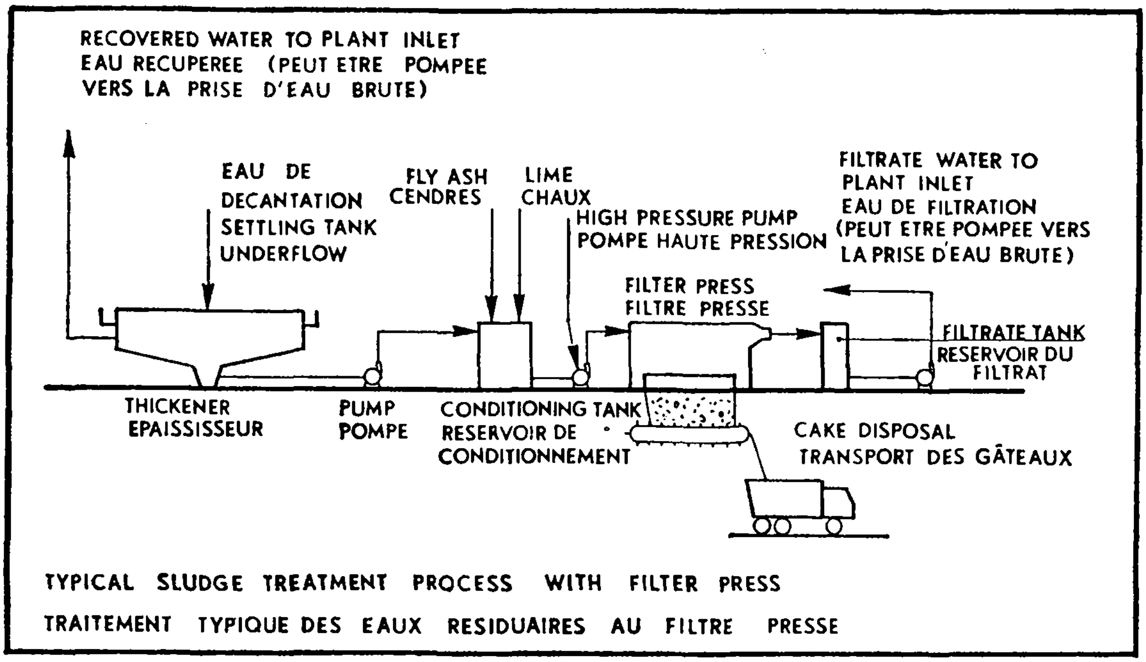


Figure 4

Méthodes alternatives—un problème d'actualité

Il est logique de rechercher une diminution globale pour le volume des boues. Ceci peut se réaliser de différentes façons:

—soit de façon directe par la réutilisation des eaux de lavage, et par la récupération de floculants;

—soit de façon indirecte en remplaçant les floculants classiques par d'autres adjuvants, plus facilement récupérables, ou pouvant être utilisées en quantité moindre.

Dans les stations de traitement on peut mélanger les eaux de lavage à l'eau brute en amont de l'endroit où sont dosés les coagulents.

Cette méthode permet de récupérer toutes les eaux de lavage; par ailleurs il n'est pas exclu que les matières en suspension dans les eaux favorisent la floculation avec comme conséquence possible une diminution de la consommation en coagulants.

Dans les stations, ou l'on n'utilise pas de floculants, et dans les stations à filtration directe cette méthode ne saurait être appliquée, l'accumulation progressive des matières solides provoquant le blocage complet de l'installation.

Dans ce cas on peut prévoir une unité pour le traitement des eaux de lavage c. à d. une station d'épuration séparée, conçue pour traiter environ 3% du débit de la station principale. Les eaux de lavage sont floculées.

Après décantation les eaux claires sont ramenées à la prise d'eau ou dans les conduites d'alimentation des filtres. Ce procédé est applicable aussi bien pour les stations à filtration directe que pour les stations ordinaires (ref. 20).

La récupération du sulfate d'alumine est assez simple, offre l'avantage d'épaissir efficacement des boues et de diminuer fortement les frais d'exploitation (ref. 22).

Deux facteurs pourtant ont limité jusqu'à présent sérieusement l'application de cette technique.

Si la qualité de l'eau brute laisse à désirer, les systèmes de récupération du coagulant peuvent donner lieu à l'accumulation de matières organiques, de fer, de manganèse, etc.

L'application de la récupération exige un contrôle permanent du procès et des incidences sur le traitement de l'eau même.

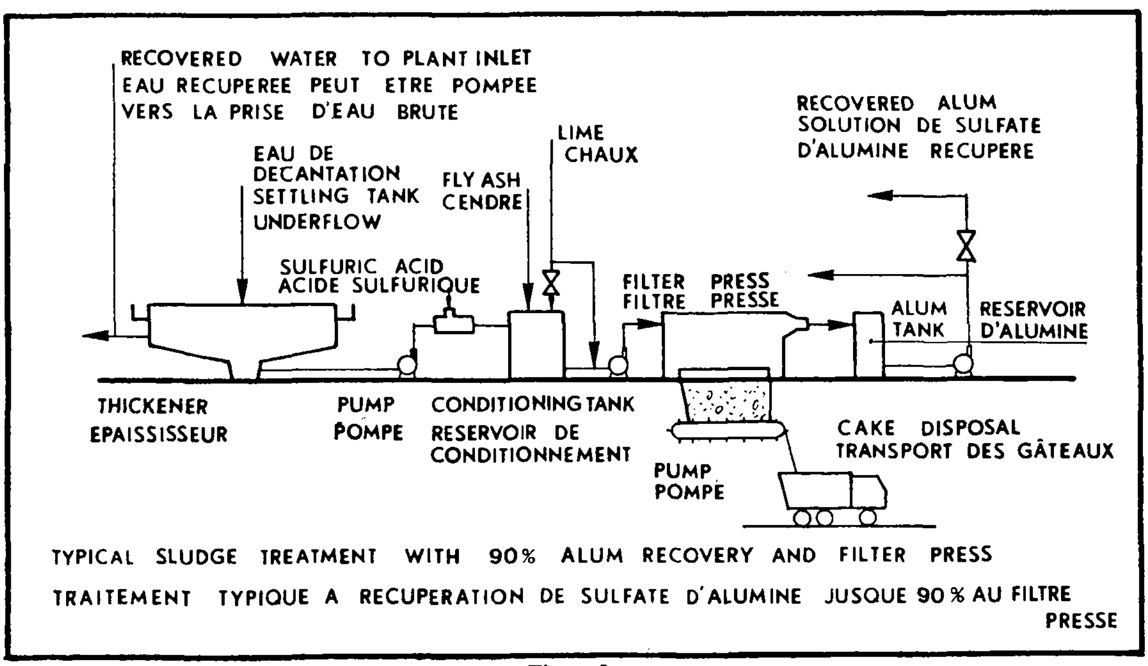


Figure 5

La récupération peut être réalisée en trois stades. (fig. 5).

1° L'épaississement pour réduire le volume des boues, mais également pour obtenir avec certitude une solution suffisamment concentrée de sulfate d'alumine: la solution doit avoir une concentration d'au moins 2% (20 000 mg/l), la coagulation à concentration inférieure étant incertaine. Une concentration des boues d'au moins 2% est absolument nécessaire.

2° Dosage d'acide sulphurique, réagissant avec l'hydroxyde d'aluminium, jusqu'à ce qu'un degré d'acidité de 2 à 3 soit atteint. De cette façon le sulfate d'alumine est remis en solution et l'épaississement ultérieur des boues est fortement activé.

3° La séparation de la boue et de la solution de sulfate d'alumine. Les boues contenant encore toujours une certaine quantité de sulfate la récupération n'est jamais totale.

Après neutralisation à la chaux, les boues peuvent être deshydratées dans un filtre-presse avec obtention d'une concentration en matières sèches de 40 à 45%.

Une récupération jusqu'à 90% de sulfate d'alumine est pourtant possible si le filtre-presse est à même de résister à l'agressivité de boues fortement acides.

Dans ce cas les gâteaux doivent être neutralisés pour faciliter le transport et le stockage. A cet effet on peut pomper en fin de cycle du lait de chaux à travers le filtre-presse.

Si des matières indésirables s'accumulent dans la solution récupérée il faut prévoir de temps en temps l'utilisation de sulfate d'alumine frais. (ref. 22).

Comme floculants alternatifs, qui réalisent une diminution des boues pour la simple raison que les quantités nécessaires sont faibles, on peut citer les polymères organiques. Comme "filter-aid" les quantités nécessaires ne dépassent pas quelques dizaines de µg/l. Comme floculant quelques centaines de µg/l. suffisent.

Les boues formées seraient de plus faible volume et leur déshydratation plus aisée; les floculants inorganiques étant remplacés par un produit organique l'on pourrait même prendre en considération la combustion des boues. Quoique les polymères sont plus onéreux, l'utilisation possible de très faibles quantités pourrait diminuer sensiblement les frais globaux d'exploitation (ref. 25).

Comme floculant alternatif facilement récupérable l'on cite le carbonate de manganèse. Dans ce procès l'on dose, outre le carbonate de manganèse, de la chaux, ce qui permet d'obtenir la formation des flocs et l'adoucissement de l'eau. (ref. 7).

Dans les boues ont peut récupérer le manganèse, la chaux et du bioxyde carbonique.

Si l'on procède à cette récupération les frais d'exploitation sont comparables à ceux des systèmes classiques.

Comme avantages spécifiques on cite:

—la réutilisation des floculants;

—la formation de flocs lourds;

—l'obtention d'un pH de l'ordre de 11,5, ce qui élimine les bactéries et les virus, la préchlorination n'étant de ce fait plus nécessaire;

—la simplicité du procès, appliquable, sans difficulté aucune, dans les installations existantes.

Nécessité d'une solution générale pour le problème des déchets

Quoique certaines sociétés de distribution ont sporadiquement fait des efforts non négligeables pendant la dernière décennie pour donner au problème de leurs eaux résiduaires une solution sérieuse la plupart des sociétés considèrent ce problème comme secondaire.

Uniquement aux Etats-Unis, où la décharge directe d'eau résiduaires dans les rivières ne sera plus tolérée à partir de 1985, l'on a fait un effort considérable, ce qui d'ailleurs apparait dans les revues techniques spécialisées.

Comme références nous pouvons indiquer surtout les rapports de 1963, 1971 et 1973 (ref. 15, 24, 25).

Citons que les efforts réalisés sont dus aux mesures légales prises dans ce pays, les boues des stations de traitement étant considérées comme eaux résiduaires industrielles.

En toute logique il devrait être parfaitement superflu, que les sociétés de distribution, qui exigent avec véhémence la protection de leurs sources aient besoin de policiers pour éviter qu'il ne deviennent eux-mêmes les pollueurs.

Les installations de traitement des boues devraient faire partie intégrale des installations classiques.

Cette solution, difficile, est pourtant la seule valable.

La difficulté majeure réside dans le fait qu'il faut distiller des possibilités multiples précitées, celle qui offre le plus de garanties dans la pratique à un prix raisonnable.

Le choix judiciaire du système dépend de plusieurs variables et ne peut être déterminé que par voie expérimentale en laboratoire et en station-pilote.

Les ingénieurs responsables ont donc tout intérêt à disposer d'une documentation aussi étendue que possible. De ce point de vue il serait utile:

1° de rassembler dans un centre d'information et de mettre à la disposition des intéressés toutes les données des expériences faites par les différentes sociétés de distribution;

2° d'uniformiser les prises d'échantillon et les analyses pour arriver à des résultats comparables et compréhensibles pour tout le monde.

Le point de départ pouvant être formé par les travaux déjà réalisés aux Etats-Unis.

Pour éviter le travail inutile et afin d'évaluer rapidement la qualité de nouvelles techniques, les tâches devraient être réparties.

—les problèmes non résolus, comme p.e. l'utilisation rationnelle de la boue déshydratée, devraient être confiés à quelques sociétés judicieusement choisies;

—les nouvelles techniques, comme l'utilisation de polymères organiques et du carbonate de manganèse, devraient également être étudiées par quelques sociétés. Par ailleurs la collaboration avec les fabricants semble absolument nécessaire.

Les tâches ainsi confiées devraient être supervisées, et les résultats mis à la disposition de toutes les sociétés.

Il est évident que les répercussions financières devraient être réparties suivant des critères objectifs.

Appendice 1

Six années d'expériences avec une installation de traitement des boues. (ref. 5). (fig. 6, 7, 8, 9).

Après des essais minutieux de laboratoire et en station-pilote une installation de traitement et de déshydratation des boues a été mise en service en 1969.

Cette installation a été incorporée dans la station de traitement des A.W.W. à Oelegem.

Dans cette station l'eau brute est captée dans le canal Albert, alimenté par la Meuse. La production journalière s'élève à 100 000 m^3.

Le traitement comporte une coagulation au moyen de sulfate d'alumine et de silice activée, une décantation, suivie de filtration rapide et lente.

Suite à la navigation intense la teneur en matières en suspension (argileuses) peut être très élevée. L'on enregistre des fluctuations très importantes de cette teneur en fonction de l'intensité de la navigation (différences sensibles entre les jours et la nuit, entre les jours de semaine et de week-end). Lors de travaux d'élargissement du canal plusieurs kilomètres en amont de la prise d'eau, la teneur en matières en suspension pouvait atteindre 500 mg/l, la teneur moyenne étant de l'ordre de 50 mg/l.

La production moyenne de boues sèches est donc de l'ordre de 5 700 kg par jour dont plus de 90% proviennent des décanteurs, le restant provenant des eaux de lavage des filtres et des eaux de rinçage des tambours rotatifs à la prise d'eau.

Dans un premier épaississeur muni d'un mécanisme de brassage lent, les eaux résiduaires épaissies ont une concentration d'environ 4%. Un trop-plein évacue environ 75% des eaux résiduaires sous forme d'eau claire. Les autres 25% soit environ 6 m³/h sont enlevés du fond de l'épaississeur au moyen de pompes à membrane. Les boues contiennent environ 40 kg de matières sèches par m³.

Les boues traversent alors deux mélangeurs en série (temps de passage 2 × 10 minutes) dans lesquels ont dosé de la chaux et sont amenées dans un second épaississeur.

Le dosage de chaux permet d'obtenir ici une concentration des boues de 8% et plus. Le second épaississeur joue également le rôle de réservoir-tampon avant le filtre-presse.

Des pompes à membrane refoulent alors sous haute pression (1000 k Pa) les boues dans le filtre-presse. La durée des cycles varie entre 30 et 90 minutes. L'on forme 60 gâteaux dont les dimensions sont 800 mm × 800 mm × 45 mm.

Chaque gâteau contient de 15 à 18 kg de matières sèches.

Un cycle est achevé lorsque le débit du filtrat atteint une valeur minimale indexée. La décharge des gâteaux se fait automatiquement mais sous contrôle visuel. Les gâteaux sont évacués par ruban transporteur vers un dépôt temporaire.

Régulièrement des camions amènent les gâteaux vers leur destination finale: un terrain à remblages se trouvant à environ un kilomètre de la station.

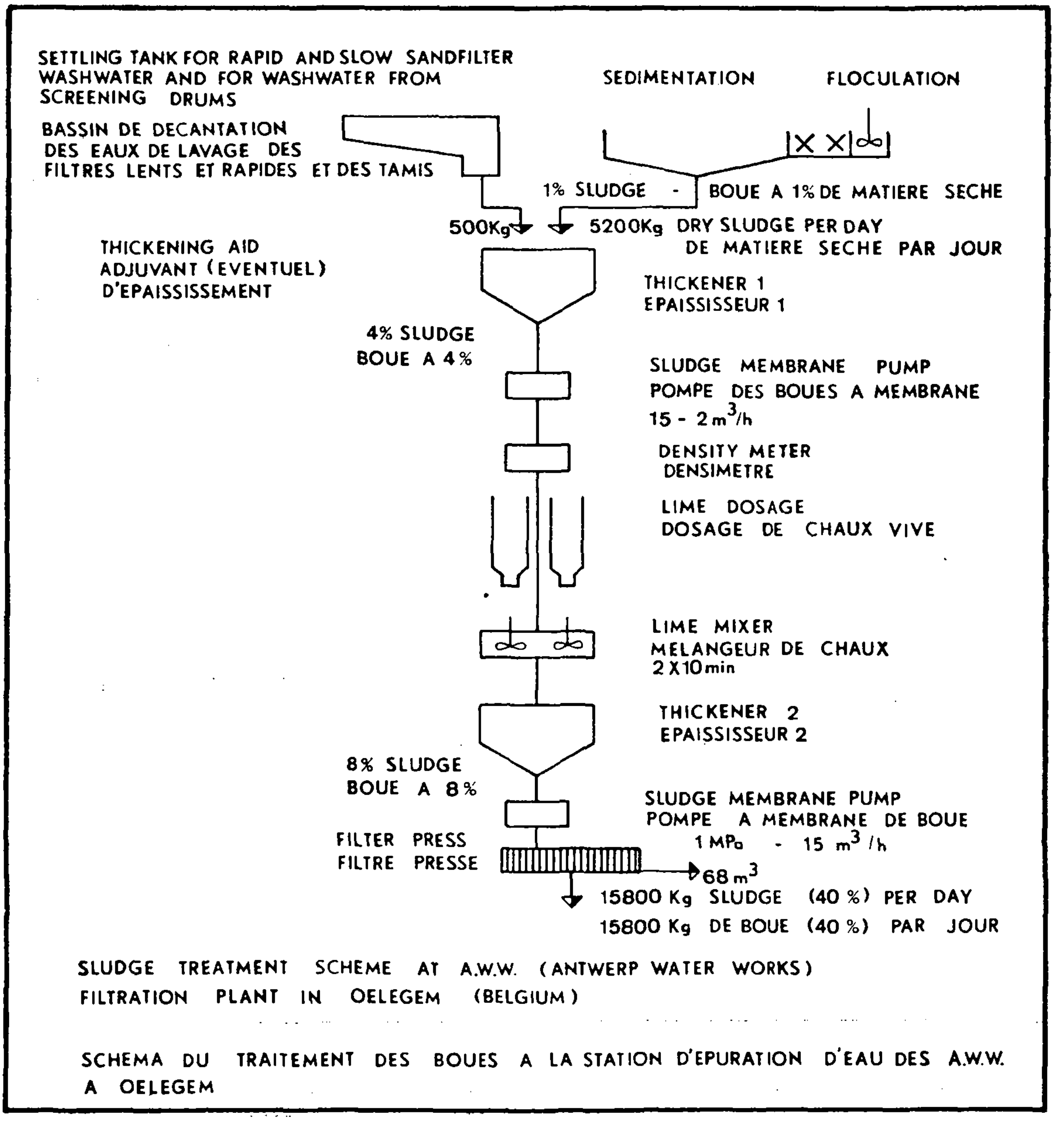
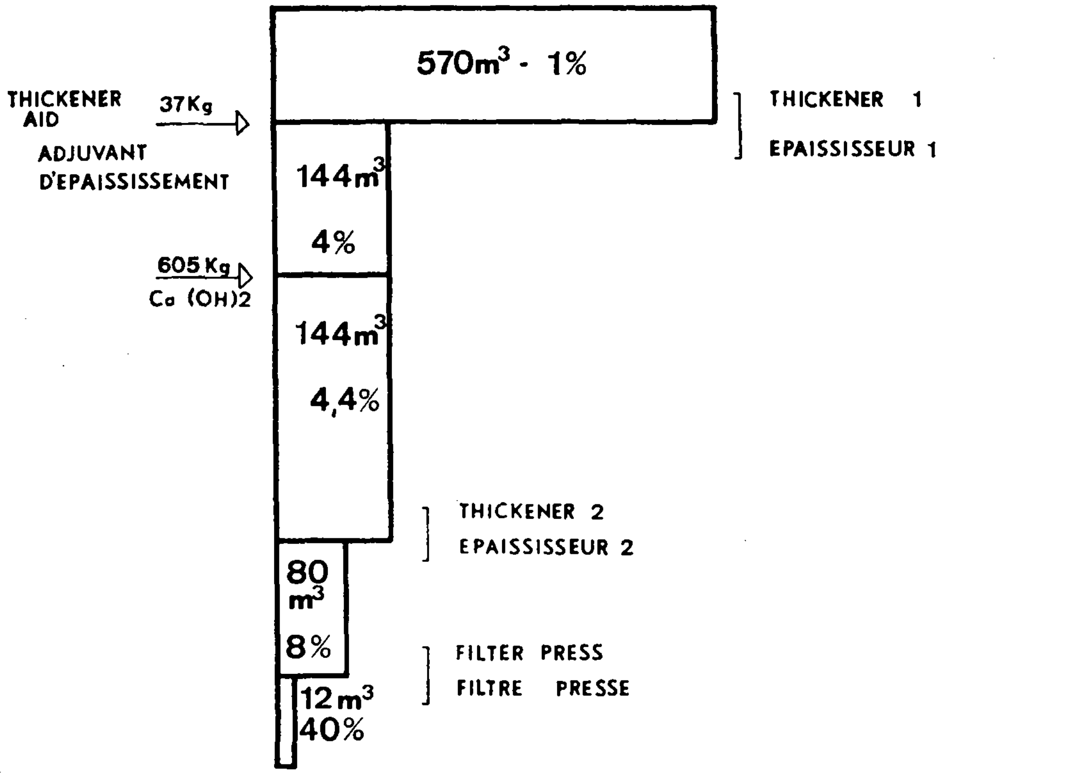


Figure 6

Si possible l'épaississement est réalisé uniquement par brassage lent. Si le niveau des boues épaissies s'élève anormalement (fonction de la quantité et de la qualité des boues) l'on dose un adjuvant d'épaississement.

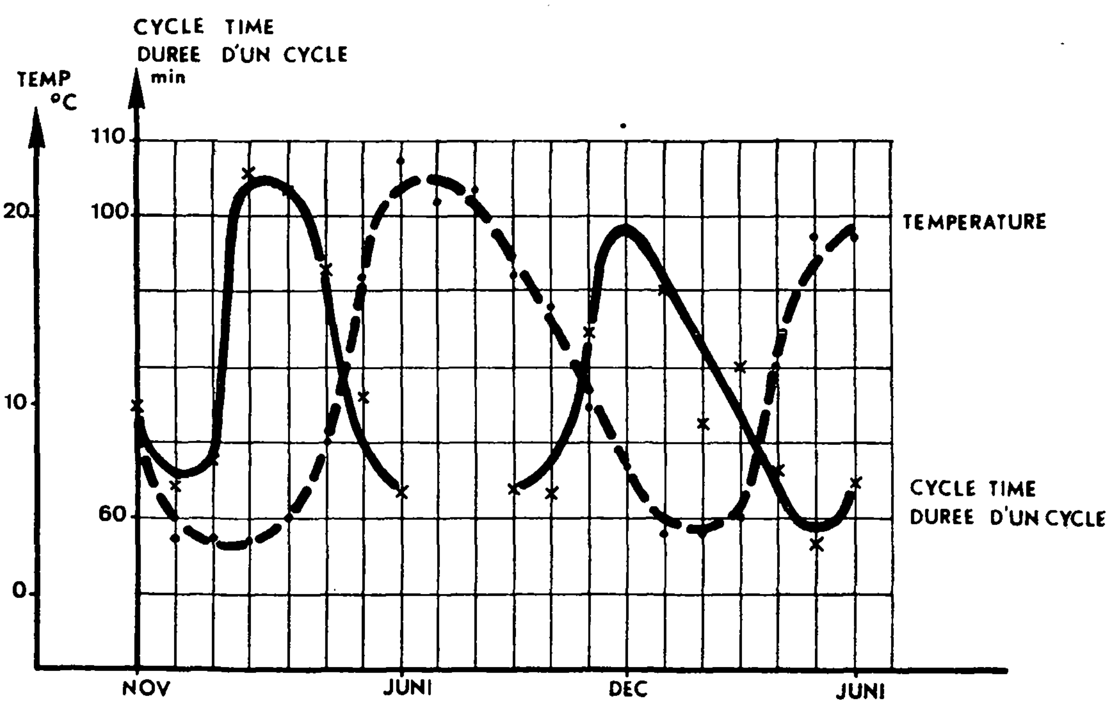
Le débit des pompes à membrane est réglé automatiquement au moyen d'un densimètre. De ce fait les



SCHEMATIC DIAGRAM OF THE DAILY VOLUME REDUCTION OF THE TREATED SLUDGE IN THE FILTRATION PLANT OF THE A.W.W. WATERWORKS IN OELEGEM AT SEVEN CYCLES A DAY PRODUCTION RATE

PRESENTATION SCHEMATIQUE DE LA DIMINUTION DU VOLUME DES EAUX BOUEUSES DANS LES DEUX EPAISSISSEURS ET DANS LE FILTRE PRESSE POUR UNE PRODUCTION DE 7 CYCLES DE FILTRE PRESSE PAR JOUR A LA STATION D'EPURATION D'EAU DES A.W.W. A OELEGEM

Figure 7



CORRELATION BETWEEN SLUDGE TEMPERATURE AND FILTER PRESS CYCLE TIME (MEAN VALUES PER MONTH)

CORRELATION ENTRE LES TEMPERATURES DES BOUES ET LES DUREES D'UN CYCLE (VALEURS MOYENNES PAR MOIS)

Figure 8

Tableau 1 donne un aperçu des resultats d'exploitation du filtrepresse.

YEAR ANNEE	ANNUAL NUMBER OF FILTER PRESS CYCLES NOMBRE DE PRESSAGES PAR AN	TOTAL NUMBER OF FILTER PRESS CYCLES NOMBRE TOTAL DE PRESSAGES	LIFE TIME OF FILTER CLOTH (CYCLES) DUREE DE VIE DES TOILES (CYCLES)	DATE OF REPLACEMENT OF CLOTHES DATE DE REMPLACEMENT DES TOILES	NUMBER OF WASHINGS WITH HCL NOMBRE DE RINCAGES AVEC HCL	SLUDGE PRODUCTION PER YEAR PRODUCTION DE BOUE PAR AN M^3
1970	1041	1041	906	1970-12-07	4	1720
1971	3117	4158	1216	1971-07-06	19	5149
1972	4465	8623	3223	1972-03-12	7	7376
1973	3973	12596	3295 3000	1973-01-05 1973-09-28	4	6563
1974	3463	16059	2534	1974-05-29	3	5721
1975 TILL JUSQU'AU 10-31	2331	18390	2648	1975-03-75	4	3851

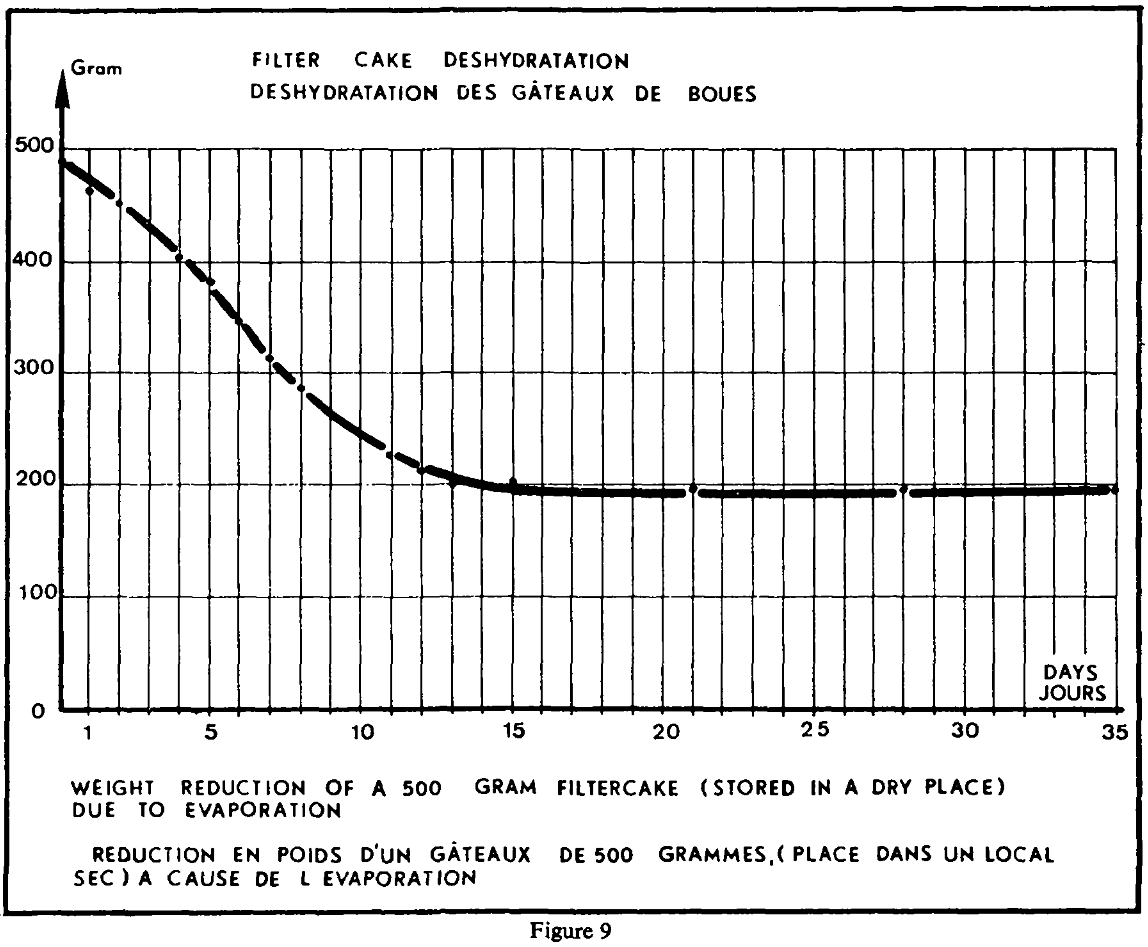


Figure 9

Frais d'exploitation d'une installation de traitement des boues, telle que décrite plus haut en prix de 1975 (100 F.B. = $2,5).

Investissement

(a) Génie civil comportant:

—deux décanteurs de 700 m^3, pour l'épaississement préalable des eaux de lavage des filtres et des eaux de rinçage des tambours rotatifs, et un réservoir d'attente souterrain (dimensions: longueur 40 m, largeur 17 m, profondeur de l'ensemble 6 m);

—deux épaississeurs de 8,5 m de diamètre et de 5 m de profondeur;

—un bâtiment comportant un étage et une cave, où l'équipement électro-mécanique est installé. (Dimensions 18 × 9 m, hauteur 10 m);

—aire pour le dépôt temporaire des gâteaux.

Prix global 22 millions F.B.

(b) Equipement électro-mécanique „ „ 10 „

On en déduit les frais annuels suivants:

Amortissement du génie civil 1,80 „
(taux d'intérêt 8% pendant 50 ans).

Amortissement de l'équipement électro-mécanique 1,02 „
(taux d'intérêt 8% pendant 20 ans).

Frais de personnel, d'entretien, de transport des gâteaux 1,80 „

Frais variables par pressage:

chaux (150 kg)	150 F.B.
Adjuvant d'épaississement (4 kg)	120 F.B.
Toiles filtrantes	40 F.B.
Energie (30 kWh)	40 F.B.
	350 F.B.

En moyenne il y a 7 pressages par jour. Ces frais s'élèvent donc à 350 × 7 × 360 0,9 million F.B.

L'on peut conclure que les frais annuels totaux s'élèvent à ± 5,5 „ „

Il est facile de déterminer l'incidence de ces frais sur le prix de vente de l'eau.

Comme la station d'Oelegem produit environ 36,5 million de m³ d'eau par an, le traitement des boues coûte $\frac{5{,}5 \times 10^3}{36{,}5} = 151$ F.B. par mille m³ d'eau potable.

Ceci représente environ 1,4% du prix de vente de l'eau.

Appendice 2

Essais de laboratoire pour la récupération des coagulants, Considérations économiques.

Des essais ont été faits sur les boues de la staion d'Oelegem pour déterminer la possibilité de récupération du sulfates d'alumine.

Dans un récipient rempli de boues l'on a mélangé de l'acide sulfurique.

Après 30 minutes de décantation l'on a déterminé le pourcentage du volume des boues.

Le liquide au-dessus de la boue concentrée a été enlevé. Après quoi l'on a déterminé la teneur en aluminium et en fer de ce liquide. La somme des quantités de $Al_2(SO_4)_3 18 H_2O$ et de $FeCl_3$ a été considérée comme coagulants récupérables. (tableau 2, a et b).

TABLEAU II a

ESSAIS DE LABORATOIRE
RECUPERATION DE COAGULANTS

LABORATORY TESTS
RECOVERY OF COAGULANTS

DATE	1974 07-18	1974 08-06	1974 08-06	1974 08-30	1974 10-21	1974 10-28	1974 10-30	1974 11-21	1974 11-25	1974 11-26	1974 11-28	DATE
BOUE DE LA STATION A OELEGEM												SLUDGE FROM OELEGEM PLANT
MATIERE SECHE g/l	10,5	12	24	10	6	10,3	7,1	10	10	10	10	g/l SOLIDS
pH	7,6	7,5	7,5	8,1	7,6	7,45	7,5	7,05	7,35	7,4	7,3	pH
TEMPERATURE °C	18	19,5	19,5	20	10,5	10	9	9	8,5	8	7,5	°C TEMPERATURE
DOSAGE D'ACIDE SULFURIQUE PAR LITRE meq/l	35,4	36	72	27	18	31	21,3	30	30	30	30	SULFURIC ACID DOSAGES meq/l PER LITER
PAR GRAMME DE MATIERE SECHE meq/g	3,4	3	3	2,7	3	3	3	3	3	3	3	meq/g PER GRAM SOLIDS
pH APRES 30 MIN	2,45	3,35	3	2,5	3,05	3,15	2,7	3,7	3,7	3,05	2,45	pH AFTER 30 MIN
EPAISSISSEMENT DE BOUE VOLUME DE BOUE APRES 10' %	28	45	81		18	45	20	49	88	38	48	SLUDGE THICKENING % SLUDGE VOLUME AFTER 10'
VOLUME DE BOUE APRES 30' %	17	29	53	19,5	13	27	12	28	60	22	26,5	% SLUDGE VOLUME AFTER 30'
COAGULANTS EN SOLUTION Fe mg/l	34	17	22	9,8	10,1	13,4	9,8	4,9	12	14	10	DISSOLVED COAGULANTS mg/l Fe
$FeCl_3$ (a) mg/l	99	49	64	28	30	39	27	14	35	41	29	mg/l (a) $FeCl_3$
Al mg/l	192	210	520	138	101	204	132	228	188	190	169	mg/l Al
$Al_2(SO_4)_3+18H_2O$ (b) mg/l	2370	2600	6400	1700	1250	2520	1620	2800	2320	2340	2090	mg/l (b) $Al_2(SO_4)_3+18H_2O$
COAG. TOTAL EN SOLUTION (a)+(b) mg/l	2470	2650	6460	1730	1280	2560	1650	2820	2350	2380	2120	TOTAL DISSOLVED COAG. mg/l (a)+(b)
mg/meq H_2SO_4	70	74	90	64	71	83	77	94	78	79	71	mg/meq H_2SO_4
RECUPERATION DU COAG. APRES EPAISSISSEMENT DE 30' mg/l	2050	1880	3040	1390	1110	1870	1450	2050	940	1790	1580	COAGULANTS RECOVERY mg/l
mg/meq H_2SO_4	58	52	42	52	66	60	69	68	31	59	53	mg/meq H_2SO_4

TABLEAU IIb

ESSAIS DE LABORATOIRE — LABORATORY TESTS

RECUPERATION DE COAGULANTS — RECOVERY OF COAGULANTS

DATE	1974 12-02	1974 12-18	1975 01-13	1975 01-13	1975 01-29	1975 02-27	1975 03-05	1975 03-20	1975 05-21	1975 07-18	DATE
BOUE DE LA STATION A OELEGEM											SLUDGE FROM OELEGEM PLANT
MATIERE SECHE g/l	10	10,9	21,6	21,6	12,6	10	6	10	10	10	g/l SOLIDS
pH	7,45	7,4	7,9	7,9	9,3	—	7,4	—	—	—	pH
TEMPERATURE °C	8	6	7,5	7,5	6	5	7	6	17	21	°C TEMPERATURE
DOSAGE D'ACIDE SULFURIQUE PAR LITRE meq/l	30	33	65	76	49	20	29	30	30	30	SULFURIC ACID DOSAGE meq/l PER LITER
PAR GRAMME DE MATIERE SECHE meq/g	3	3	3	3,5	3,9	2	4,8	3	3	3	meq/g PER GRAM SOLIDS
pH APRES 30 MIN	3,75	3	3,85	3,2	2,6	3,3	3,25	2,9	3,6	3,5	pH AFTER 30 MIN.
EPAISSISSEMENT DE BOUE VOLUME DE BOUE APRES 10' %	64	34	80	68	44,5	17	14	34	23,5	—	SLUDGE THICKENING % SLUDGE VOLUME AFTER 10'
VOLUME DE BOUE APRES 30' %	40,5	19,5	57	54	28	6	9	20	14	—	% SLUDGE VOLUME AFTER 30'
COAGULANTS EN SOLUTION Fe mg/l	7,8	9	23	32	42	12,3	6,7	6,7	3,2	—	DISSOLVED COAGULANTS mg/l Fe
$FeCl_3$ (a) mg/l	23	26	65	92	120	36	19,5	19,5	9,3	—	mg/l (a) $FeCl_3$
Al mg/l	205	228	526	563	270	93	197	147	111	99	mg/l Al
$Al_2(SO_4)_3 + 18H_2O$ (b) mg/l	2530	2810	5260	7060	3330	1150	2430	1815	1360	2430	mg/l (b) $Al_2(SO_4)_3 + 18H_2O$
COAG. TOTAL EN SOLUTION (a) + (b) mg/l	2550	2840	5350	7150	3450	1190	2450	1830	1370	2430	TOTAL DISSOLVED COAG. mg/l (a) + (b)
mg/meq H_2SO_4	80	81	83	94	70	63	84	61	59	81	mg/meq H_2SO_4
RECUPERATION DU COAGULANT APRES EPAISSISSEMENT DE 30' mg/l	1520	2260	2300	3270	2980	1120	2320	1460	1310	—	COAGULANTS RECOVERY mg/l
mg/meq H_2SO_4	51	68	35	43	50	59	77	49	56	—	mg/meq H_2SO_4

Pour les 21 essais indiqués, les pH ont varié entre 2,45 et 3,9.

Le rapport moyen était de 77,5 mg de coagulant en solution/meq/H_2SO_4 et la consommation moyenne d'acide sulfurique: 3,1 meq H_2SO_4/g de matière sèche.

Les prix actuels en Belgique des produits chimiques sont les suivants:

1 kg d'$Al_2(SO_4)_3 18H_2O$ 3,32 F.B.

1 eq. H_2SO_4 (66°Be, qualité technique) 0,128 F.B.

ce qui donne le rapport suivant:

$$\frac{\text{prix 1 meq } H_2SO_4}{\text{prix 1 mg } Al_2(SO_4)_3 18H_2O} = \frac{0{,}128 \times 10^{-3}}{0{,}00332 \times 10^{-3}} \doteqdot 39.$$

Si donc le dosage d'un meq de H_2SO_4 permet de récupérer plus de 39 mg de coagulants, la récupération est profitable.

En se basant sur ces données la récupération éventuelle des coagulants à Oelegem pourrait donner le bilan annuel suivant.

Quantité de matières produites: 7000 kg/j (matières en suspension 70 mg/l en moyenne) soit 2,55 million kg/a

Quantité nécessaires de H_2SO_4 (3,1 eq/Kg de matières sèches): 7,92 millions eq. au prix de 7,92 $\times$ 10^6 $\times$ 0,128 =1,01 millions F.B./a ... ... (a)

Quantité de coagulants récupérés: cette quantité dépend essentiellement du degré d'épaississement.

(a) Un épaississement des boues de 1% (10 g/l) à 4% (40 g/l) permet de récupérer

77,5 $\times$ 0,75 = 58 g de coagulant/eq H_2SO_4

soit au total

58 $\times$ 7,92 $\times$ 10^3 = 460 000 kg de coagulants/a

La consommation annuelle de coagulants étant de $\pm$ 730 000 kg., la récupération permet une économie de

3,32 $\times$ 460 000 = 1,53 million F.B. ... ... (b)

Le bénéfice net s'élève à [(b) – (a)]: 0,52 millions FB/a

La récupération s'élève à 460 000: 730 000 = 64%.

(b) Si l'épaississement est suivi d'une déshydration par filtre-presse, la récupération possible est de l'ordre de 77,5 g de coagulant/eq H_2SO_4 soit au total:

77 $\times$ 7,92 $\times$ 10^3 Kg = 614 000 kg/a

L'économie réalisée est de:

3,32 $\times$ 614 000 = 2,06 millions de F.B. ... ... (c).

Le bénéfice s'élève à [(c) – (a)]: 1,03 millions F.B./a, la récupération étant de 84%.

Ces conclusions ne sont valables que si la composition des boues reste constante et que si le pouvoir de coagulation de la récupération reste égal à celui du produit frais.

La qualité du produit récupéré doit être étudiée en station-pilote.

Les travaux nécessaires comportent:

1° Stockage et dosage de l'H_2SO_4.

2° La mesure du pH.

3° La protection contre l'aggressivité de l'épaississeur.

Soit un investissement de 3 millions de F.B. donnant lieu à des frais annuels de 300 000 F.B.

Pour la solution (a) il reste finalement un bénéfice net de 220 000 F.B./a. Par ailleurs l'utilisation d'un adjuvant d'épaississement n'est plus nécessaire, ce qui permet de réaliser une économie supplémentaire de 200 000 F.B./a.

Solution (b)

les économies réalisables sont les suivantes:

Récupération du coagulant		1,03 millions F.B.
Non utilisation d'un adjuvant d'épaississement		0,20 „
Diminution drastique de la consommation en chaux		0,25 „
Total:		1,48
Frais inhérents aux investissements	–	0,30 „
Bénéfice:		1,18 „

Tenant compte d'un taux d'intérêt de 8% en 20 ans les frais d'investissement pour l'appareillage de déshydratation pourraient s'élever à 11,6 millions F.B.

En ce qui concerne les essais déjà réalisés, on peut encore attirer l'attention sur les points suivants:

1° Outre l'aluminium et le fer, l'acide sulfurique met encore en solution des matières organiques de l'arsenic, de la manganèse et du cadmium. (tableau 3)

Les effets d'enrichissement en matières nocives restent à étudier en station-pilote.

TABLE III / TABLEAU III — TYPICAL ENRICHMENT EFFECTS / EFFETS D'ENRICHISSEMENT TYPIQUE

SLUDGE FROM / BOUE DE	1975-07-02	1975-07-18	PRECISION OF MEASUREMENT / PRECISION DE MESURE
pH		3,5	0,1
Al mg/l	97	197	10 %
Fe mg/l	58		10 %
As 10^{-3}mg/l	90		10 %
Cd 10^{-3}mg/l	42	21	10 %
Cr 10^{-3}mg/l	50		10 %
Cu 10^{-3}mg/l	80		10 %
Zn 10^{-3}mg/l	1250		10 %
Mn 10^{-3}mg/l	3400	3500	10 %
Pb 10^{-3}mg/l	20		10 %
Hg 10^{-3}mg/l	< 0,1		0,1 µg/l

2° Quoique l'épaississement des boues est fortement amélioré par le dosage d'acide sulfurique, la filtration sur membrane de 0,45 microns (diamètre nominal 40 mm) avec une sous-pression de 500 mm de mercure demande plus de temps que la filtration de boues traîtées à la chaux.

L'on peut supposer que la durée des cycles du filtre-presse sera plus longue pour des boues acides, ce qui nécessiterait des dimensions plus grandes pour ce filtre.

3° L'influence de l'utilisation d'acide sulfurique sur le volume résiduel des boues, le pH, l'aluminium et le fer en solution et sur la récupération de coagulants est indiquée sur les fig. 11, 12, 13, 14, 15.

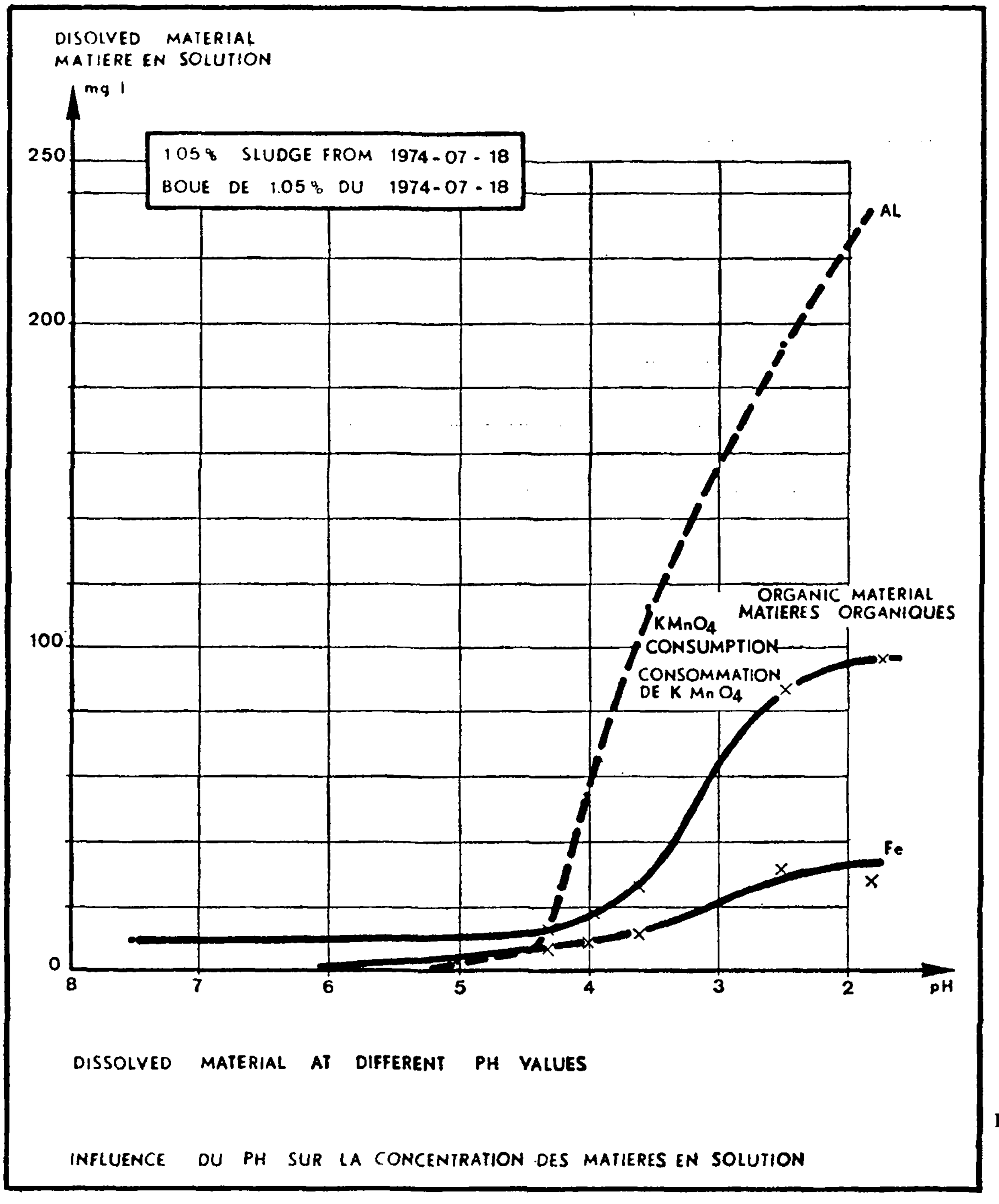


Figure 10

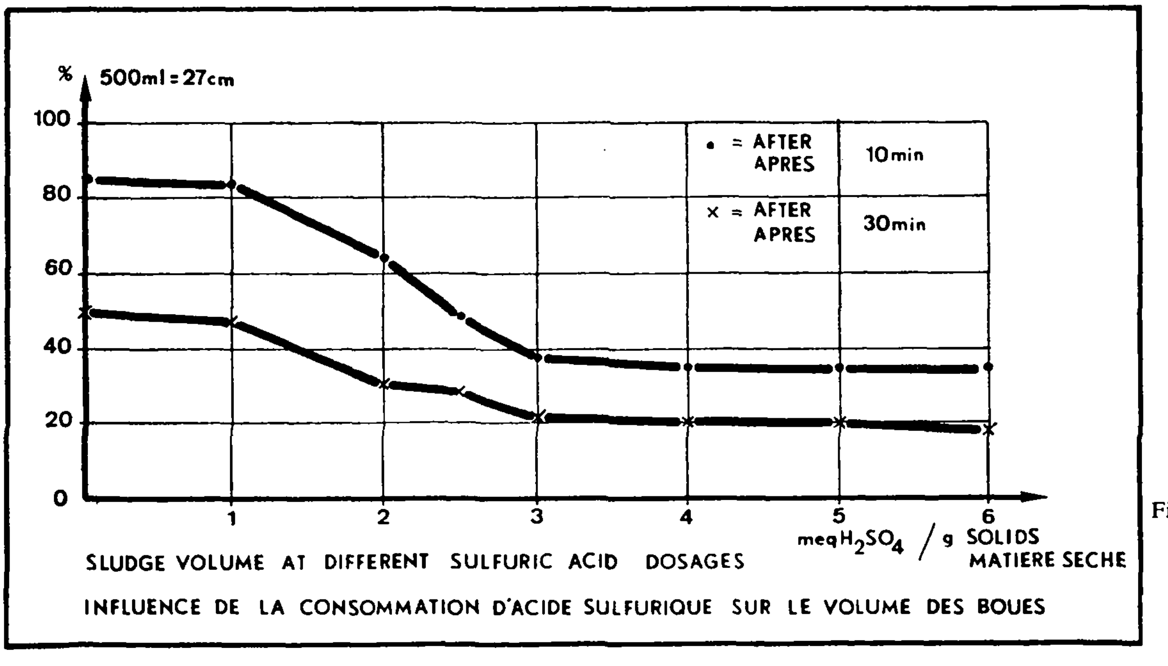


Figure 11

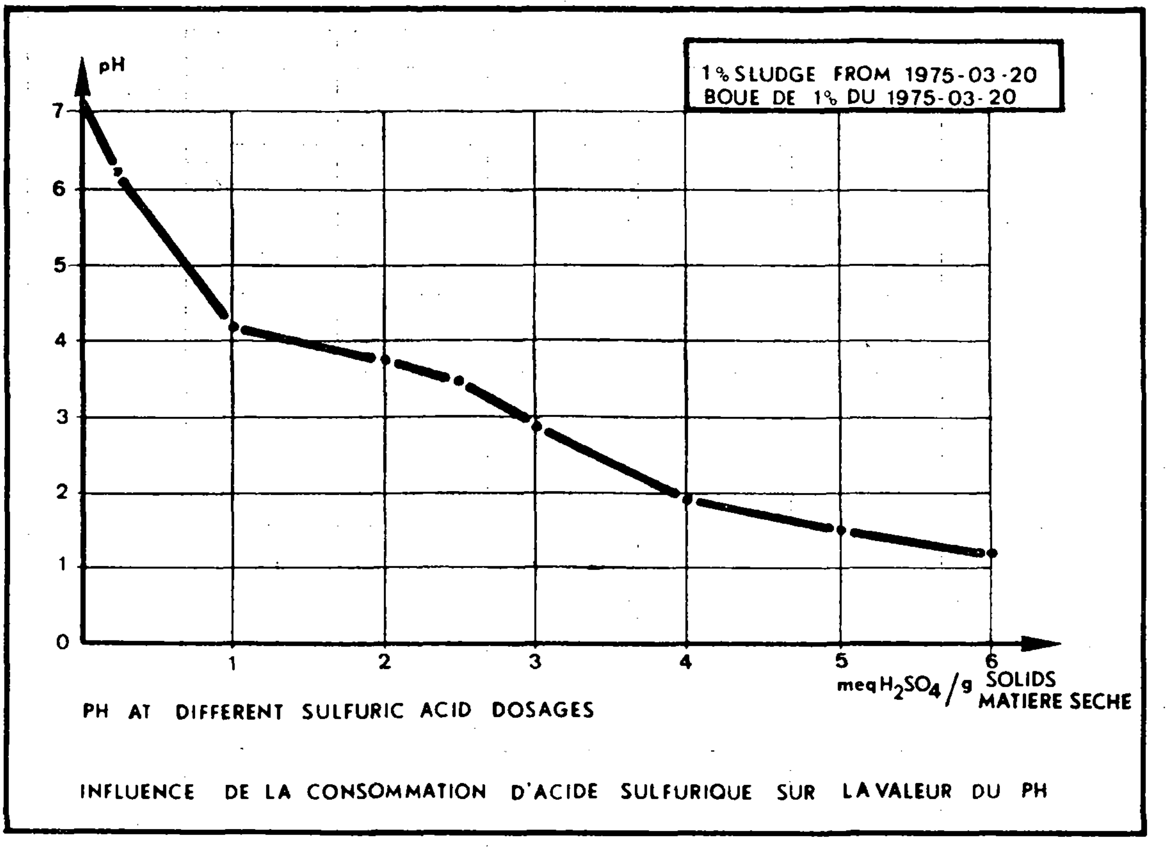


Figure 12

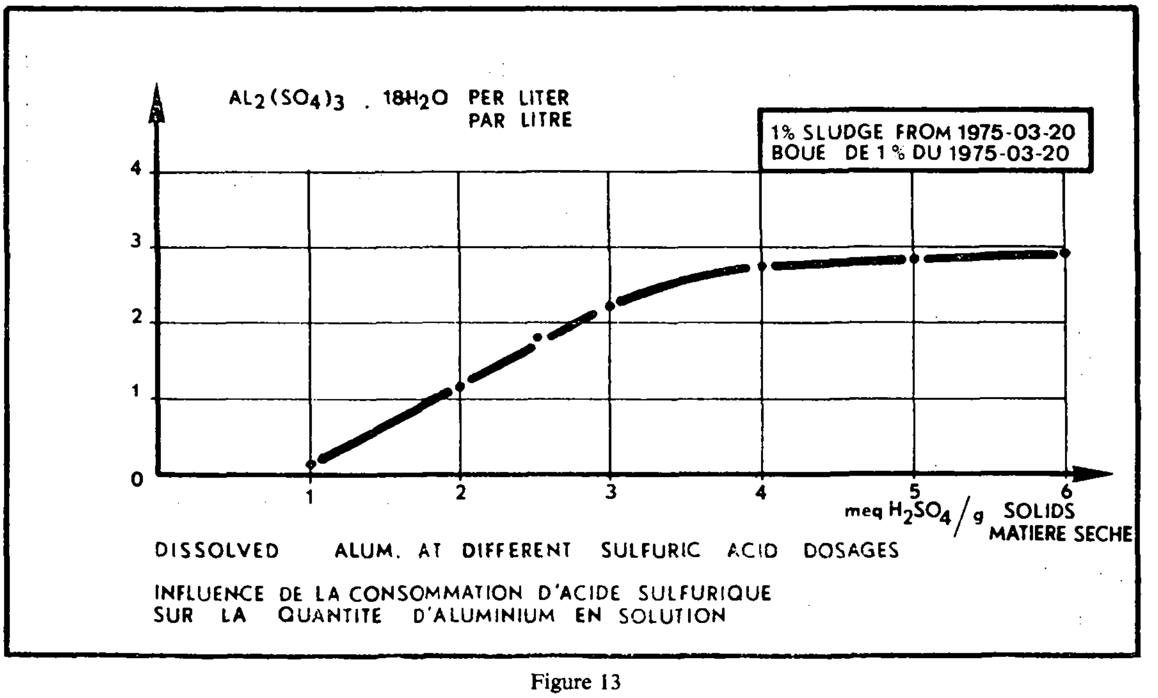


Figure 13

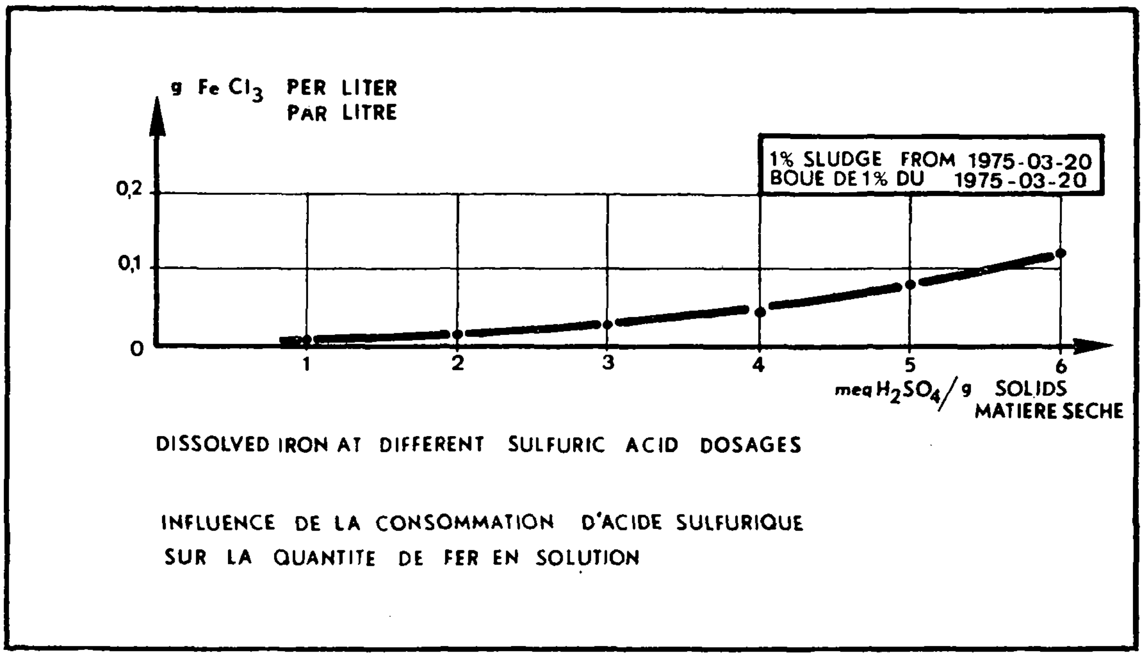


Figure 14

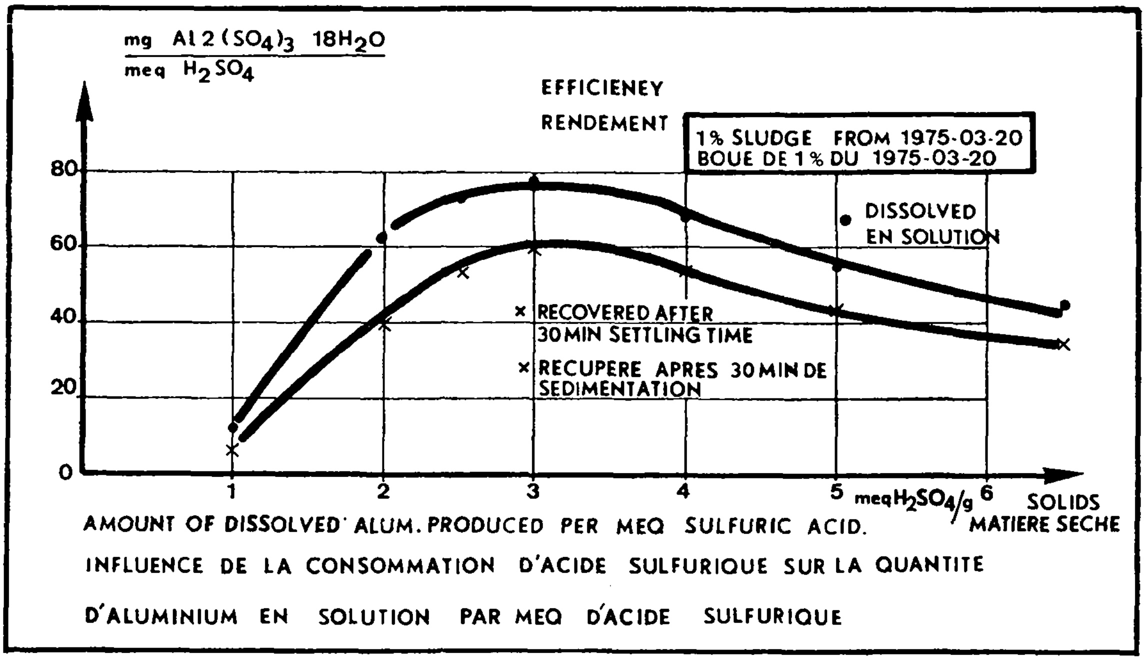


Figure 15

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Summary

The amount of sludge to be dealt with in potable water treatment plants has dramatically increased as surface waters have come to be used as raw water sources.

The solutions we have to give to the sludge problem must rely upon following rules:

—our environment should not be damaged by our activities;

—the deposits of human activities are to be turned to the use of mankind.

As sludge disposal in a potable water treatment plant is determined by the plant's surroundings, one has to make a choice between several possible ways:

(a) natural solutions:
 —dumping at sea or into a big river;
 —settling basins;

(b) mechanical methods:
 —high rate settling tanks;
 —centrifuges;
 —freezing;
 —filter-presses.

Final disposal remains a problem.

Sludge disposal problems can be tackled by reducing the amount of flocculants used, and by recovering flocculants.

Research is needed in the field of organic polymers, magnesium carbonates used as flocculants, and the recovery of inorganic flocculants.

In well chosen plants, research on those new sludge disposal solutions should be tried out on a pilot-plant scale and subsequently even on an industrial scale, and the results should be published to everyone's benefit.

In appendix 1, a survey is given of the 6 year activities of a filter-press sludge disposal system used at the Antwerp Water Works filter-plant in Oelegem, Belgium. The real cost of the system is calculated and found to be less than 0,004$ per m^3 of potable water produced, or about 1,5% of the cost of the potable water.

In appendix 2, some laboratory tests on alum recovery are discussed.

Pilot-plant studies are considered necessary before any conclusions can be drawn on enrichment effects.

Based on these tests, the break-even cost of a recovery installation is calculated, assuming the recovered iron and alum has the same flocculation capacity as the fresh salts.

Revue Générale des Maladies amenées par l'Eau

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Introduction

Jusqu'à une époque relativement récente, une revue générale des maladies amenées par l'eau n'aurait guère débordé la bactériologie. Aujourd'hui, nos moyens s'étant considérablement renforcés, les résultats de nos investigations montrent que les implications de la qualité de l'eau dans la maladie sont plus importantes qu'on le supposait autrefois.

Il semble que, pour le responsable de la distribution de l'eau, le sujet puisse être envisagé selon trois parties :

- Maladies en correspondance avec la structure des eaux, risques à long terme, par carence ou par surcharge ;
- Maladies correspondant à la présence dans l'eau d'éléments toxiques, risques à moyen terme par accumulation ;
- Maladies correspondant à la présence dans l'eau de bactéries, de parasites ou de virus ; risques à court terme.

Les éléments constitutifs de cette revue sont connus, aussi l'orienterais-je vers des réflexions sur chacune de ses composantes afin qu'une large discussion puisse s'instaurer. Je remercie tous ceux dont les travaux ont été utilisés ; les références en sont données pour la plupart en bibliographie.

1. Maladies en correspondance avec la structure des eaux, risques à long terme par carence ou par surcharge

1.1. Présentation

Bien que le sujet soit considérable, nous ne citerons que les goîtres, les caries dentaires, les troubles cardio-vasculaires, les modifications de l'hémoglobine.

1.11. Le goître endémique

Son diagnostic se fait par la fixation d'iode radioactif, I_{131}, et la scintigraphie permettant de distinguer les goîtres avides d'iode, sans augmentation du métabolisme basal et traitables, de ceux qui fixent mal I_{131} et échappent au simple traitement médical. Les goîtres, en tant qu'hypertrophie de la thyroïde, peuvent être bénins et n'être gênants que par les troubles de compression de voisinage qu'ils déterminent ; ils peuvent devenir toxiques lorsqu'ils évoluent vers le goître basedowifié ; ils peuvent enfin subir une dégénérescence, parfois maligne. Le goître et sa thérapeutique sont connus depuis longtemps ; cependant, en 1960, on estimait qu'il existait encore deux cents millions de goîtreux de par le monde, les femmes et les enfants étant plus sensibles que les adultes hommes. Ce qu'il faut en retenir pour notre propos, c'est que la quantité d'iode contenue dans l'eau de boisson est généralement le reflet de la richesse en iode du sol. C'est ainsi que dans les régions goîtrigènes on n'en trouve que de 0,1 à 2 µg/litre d'eau contre 2 à 15 µg/l dans les contrées non goîtrigènes. On rapporte également que, dans l'eau de boisson de certains villages de Grande-Bretagne, l'incidence goîtrigène atteint 56 % pour 2,9 µg/l, alors qu'elle n'est que de 3 % dans d'autres villages où la teneur en iode des eaux est de 8,2 µg/l. Mais la carence relative en iode des eaux n'est pas le seul facteur goîtrigène et il convient de rappeler que les « Brassica », largement consommés dans les pays d'altitude, contiennent des substances inhibitrices de la fixation d'iode.

Géographiquement, on distingue l'influence régionale, l'effet de la plus récente glaciation, la distance à la mer, la faiblesse des précipitations annuelles. Sur la base de ces deux derniers facteurs, on a calculé que 20 à 50 mg d'iode par acre (soit 32 à 80 env. par ha) sont apportés par les pluies de l'Atlantique, près des côtes, contre 0,7 mg/acre (soit 1,7 env. par ha) dans la région goîtrigène des Grands Lacs nord-américains.

En réalité, lorsqu'il s'agit d'une carence essentielle en iode, s'il doit y avoir prophylaxie de masse (1), il ne semble pas souhaitable que celle-ci soit réalisée par le truchement du réseau public de distribution d'eau.

1.12. Fluorose et carie dentaire

La démonstration expérimentale de la relation existant entre ces affections et les concentrations de fluor a été réalisée dès 1925, puis confirmée vers 1931 par les études d'endémicité fluorée chez les animaux domestiques paissant au voisinage d'industries traitant le minerai d'aluminium ou produisant des superphosphates. Mais, également vers 1930, des travaux sur la prophylaxie et la toxicologie du fluor montraient que cet élément peut aussi jouer un rôle significatif dans la prévention de la carie dentaire chez l'homme. La marge entre dose toxique et dose bénéfique apparaissait alors si faible que des essais à long terme devenaient nécessaires pour la mise en œuvre d'une fluoration corrective des eaux de distribution publique. Les observations menées en 1942 par Dean sur des enfants de 12 à 14 ans de vingt et une villes de quatre Etats des U.S.A. ont montré que la permanence dans l'eau de 1,3 p.p.m. de fluor aurait un effet protecteur et, en 1950, Hodge établissait que la relation entre la fluorose et les concentrations de fluor dans l'eau est de forme logarithmique, le point d'inflexion de la courbe étant à considérer comme représentant la concentration en fluor assurant le maximum de protection de la dentition.

Du point de vue du métabolisme du fluor et en résumé, étant donné la masse d'expérimentations relatées dans la littérature, on peut retenir ce qui suit :

- Au cours de leur absorption, les sels de fluor franchissent rapidement le tractus gastro-intestinal. C'est ainsi que chez le mouton le fluor radioactif apparaît dans le sang 5 minutes après le passage dans l'estomac. Tout est terminé en 3 heures.

  Sur le rat, avec des solutions diluées, on a constaté qu'en 90 minutes 86 % de la dose ingérée était passée dans le sang. Chez l'homme, pour des petites doses de solutions diluées non radioactives, le fluor apparaît au bout d'une heure dans le sang ; 20 à 30 % du fluor ingéré sont retrouvés dans les urines durant les 3 à 4 heures suivantes. Pour les sels insolubles de fluor, le transit est plus lent et l'on s'aperçoit que les sels de calcium et

(1) La correction individuelle utilise très souvent la solution de Lugol, par doses croissantes et décroissantes, avec périodes de repos.

d'aluminium comme les hauts régimes en graisse et le lait sont dépressifs de l'absorption intestinale, le chlorure de sodium paraissant jouer un rôle similaire.

- Au niveau de la distribution dans l'organisme, le fluor absorbé suit le sort du chlore, traversant les membranes cellulaires, y compris celles qui sont productrices des hématies. 75 % du total du fluor du sang sont dans le plasma, 5 % se lient aux solutions plasmatiques dans les conditions physiologiques. La teneur des tissus est le reflet de celle du plasma.
- Quant à l'excrétion, une proportion relative du fluor est évacuée par l'urine et les fèces ; elle est grandement variable selon la dose, la solubilité et la méthode d'ingestion. On signale cependant que la plus grande partie du fluor qui ne s'est pas fixé sur les os et les dents est éliminée par l'urine. Les sujets qui ont un régime normal et qui ne sont pas exposés à des doses inhabituelles de fluor par l'alimentation, l'eau ou l'atmosphère, excrétent 80 % du fluor qu'ils ingèrent par l'urine.

Certes, les faits expérimentaux ne sont pas contestés, mais l'unanimité est loin d'être réalisée quant à l'opportunité d'utiliser le réseau public de distribution d'eau pour assurer la prévention souhaitable contre la carie dentaire d'une population. Les deux arguments principaux contre la généralisation éventuelle d'une telle pratique sont d'une part les aléas dans la maintenance de la concentration utile face aux risques de surcharges nuisibles et, d'autre part, le fait que le traitement s'appliquant à la capitation totale, et non pas seulement au volume d'eau de boisson et à usage domestique, une grande quantité de fluor serait dispersée sans profit dans le milieu extérieur, faisant encourir un risque qui n'apparaît pas encore évalué. Parmi les autres voies de correction ordinairement utilisées sont les tablettes fluorées distribuées aux enfants et aux femmes en période de gestation et de lactation.

A contrario, il existe des régions où les eaux sont naturellement hyperfluorées et provoquent les marbrures classiques de l'émail des dents et parfois jusqu'à des atteintes osseuses marquées. Dans ces cas particuliers, il faut ou défluorer les eaux, ce qui n'est ni plus pratique ni plus sûr que la correction par addition, ou, ce qui est préférable, déplacer le point de prise chaque fois qu'il est possible de le faire.

Pour terminer, rappelons que, en matière de toxicité :

- la toxicité aiguë apparaît pour des charges de 2 000 à 5 000 mg en ion F (dose léthale),
- la toxicité chronique est observée pour des doses relativement peu élevées : de 2 à 8 mg/l fluorose dentaire ; de 20 à 80 mg/l fluorose osseuse ; à partir de 40 mg/l retard de croissance ; à partir de 70 mg/l troubles de la reproduction ; plus de 100 mg/l troubles thyroïdiens et hypophysaires, troubles rénaux.

1.13. Les maladies cardio-vasculaires

Concernent essentiellement des cardiopathies coronariennes, angor pectoris typique ou angine de poitrine, naissant brusquement à l'occasion d'un effort ou pendant le décubitus nocturne, caractérisé par une douleur rétro-sternale intense, irradiant vers le cou et le bras gauche, et dont la principale complication est l'infarctus myocardique au pronostic bien connu.

Ici s'ouvre un nouveau chapitre de la pathologie de l'eau, introduit sur la base de travaux de nos confrères, anglo-saxons notamment, sur les relations entre les oligo-éléments présents dans l'eau et les maladies cardio-vasculaires, sujet ayant donné lieu à de nombreux débats dont l'un des derniers en date était organisé à Luxembourg en mai 1975 par la Communauté Européenne de Coopération Economique.

Nous venons de voir, tant pour le goître endémique que pour la fluorose et la carie dentaire, que les constatations épidémiologiques, relativement faciles à établir surtout au cours des années de croissance, ont conduit aux remarquables études et résultats connus en matière de prévention. Les choses sont plus difficiles à élucider dans le présent chapitre car la manifestation est généralement terminale.

Le consensus général cependant n'est pas acquis quant à rapporter uniquement à l'eau les faits constatés. Il semble qu'une part serait due à l'influence de l'utilisation généralement accrue pour l'alimentation de produits manufacturés qui perdent au cours des processus de fabrication une fraction importante de nutrients en s'enrichissant de traces métalliques. Une autre part reviendrait à l'adoucissement de l'eau de boisson qui provoque des changements tant dans l'équilibre minéral des ressources auxquelles les populations sont adaptées de longue date que, par voie de conséquence, dans la physiologie cardio-vasculaire par inadaptation biochimique aux modifications rapides, d'origine technologique, de l'équilibre des eaux utilisées.

De nombreuses investigations toutefois semblent confirmer que le risque coronarien est le plus grand chez les résidents de certains secteurs géographiques tributaires d'eaux douces et, à l'occasion du symposium de Luxembourg, il a été admis qu'il existe « une association de type inverse entre la dureté de l'eau et les maladies cardio-vasculaires, c'est-à-dire qu'à une faible dureté de l'eau correspond une augmentation de la fréquence des maladies ».

De plus, des travaux récemment publiés font état de l'interaction entre les éléments traces présents dans les eaux qui, schématiquement, réagiraient les uns par rapport aux autres en s'inhibant partiellement ou qui interviendraient directement au niveau du muscle cardiaque, ou du sérum sanguin, ou de la tension artérielle, montrant que, dans certaines eaux douces, il resterait encore suffisamment de substances « bénignes » pour bloquer un processus toxique métallique éventuel. Mais bien que le ou les facteurs spécifiques restent à élucider, l'attention s'est portée sur la déficience magnésienne qui accompagne la malnutrition protéino-calorique en affectant les autres électrolytes aussi bien que le métabolisme des lipides et les phénomènes neurologiques dans l'ischémie cardiaque.

De toute évidence ce problème va encore évoluer. Mais on ne saurait accepter aujourd'hui que soient rapportés aux seules eaux douces naturelles, à l'exclusion des eaux adoucies par permutation sur résines échangeuses d'ions — comme il est dit dans une revue —, des faits dont la conséquence épidémiologique est si importante. L'Organisation Mondiale de la Santé a d'ailleurs établi des programmes étendus et approfondis les concernant.

1.14. La méthémoglobinémie infantile

Elle se manifeste par une cyanose généralisée, entérogène : syndrome caractérisé par une coloration bleue des téguments et des muqueuses, liée à des troubles dyspeptiques d'origine intestinale. A ne confondre ni avec les autres cyanoses généralisées des cardiopathies et des pneumonies ni avec les cyanoses localisées, cyanoses des extrémités ou par compression.

Cette maladie est spécialement confinée chez le nourrisson et résulte de la réduction biologique en milieu gastrique de l'ion NO^3 (nitrate) en NO^2 (nitrite) qui convertit l'hémoglobine en méthémoglobine inapte à fixer l'ogygène. Bien que l'ion NO^2 soit l'agent toxique, il ne peut exister dans les eaux en concentration importante, sauf addition volontaire ; il ne représente qu'un stade intermédiaire.

Les cas cliniques apparaissaient surtout après utilisation d'eau de puits polluée pour le coupage des biberons, l'ébullition constituant malencontreusement un facteur d'enrichissement en nitrates. Cette affection, rare, a repris de l'actualité du fait de l'accroissement continu de la concentration des eaux superficielles, destinées à la production d'eau alimentaire, en nitrates provenant de l'industrie, de l'agriculture et des eaux usées urbaines. Les enfants courent donc le risque, à l'occasion de la reconstitution du lait à partir de lait sec et également du fait que le nouveau-né reste doté un certains temps d'hémoglobine fœtale facilement convertissable en méthémoglobine. La question des nitrates vient de donner lieu à une conférence tenue à Copenhague en août 1975 où les aspects de santé ont été débattus en liaison avec les problèmes d'eutrophisation.

2. Maladies correspondant à la présence dans l'eau d'éléments toxiques ; risques à moyen terme par accumulation

Pour ces faits d'intoxication, nos renseignements de base sont classiques ; cependant, l'insidiosité des effets s'est révélée plus près de nous non seulement par des incidents collectifs tragiques, mais aussi grâce au développement de nos moyens d'investigation. D'immenses travaux ont été effectués. L'Organisation Mondiale de la Santé, les E.P.A. aux U.S.A., les Communautés européennes, les Etats eux-mêmes, ont uni leurs efforts pour dresser le bilan de l'invasion de notre environnement par les éléments inorganiques et organiques, résultant le plus souvent de l'activité industrielle, que les substances en cause relèvent des listes noire, ou grise, ou beige, figurant dans les conventions internationales aux buts différents.

En effet, après Minamata et les accidents du Bassin du Jintsu, chacun s'est préoccupé de connaître l'état de la dérive des éléments métalliques et leur devenir (1). Nous nous limiterons à quelques-uns des éléments toxiques en relation directe avec des eaux de consommation dont la teneur risque de s'accroître au sein des réseaux de distribution publics ou privés, notamment le plomb, le cadmium (1). Le mercure ne figurera dans cette revue que pour mémoire ; il représente un reliquat de la pollution comme les trois groupes suivants, pesticides organo-chlorés, pesticides organo-phosphorés, herbicides de type organo-phénoxy, qui seront également envisagés.

(1) *Cf.* notamment les travaux de l'Internationale Arbeitsgemeinschaft der Wasserwerke im Rheineinzugsgebiet, de la Commission internationale pour la protection du Rhin contre la pollution, des Laboratoires de la Préfecture de Paris dans le bassin parisien, des Agences de Bassin françaises, des Communautés européennes, etc.

2.1. Les composés métalliques

Le plomb, le cadmium et le mercure induisent une pathologie commune, bien que d'inégale valeur, au niveau de l'ingestion, de la circulation et de l'excrétion qui s'effectue normalement par les émonctoires habituels. Ces métaux se différencient cependant au niveau du franchissement de la barrière intestinale, le taux de passage étant estimé à 6 à 7 % de l'ingestat pour le plomb, 4 à 7 % pour le cadmium, 15 % pour les sels inorganiques de mercure. La circulation est assurée par le sang où les métaux se fixent sur les hématies, un pourcentage relativement faible subsistant dans le plasma. Pour ce qui regarde la rétention, elle varie d'un métal à l'autre et constitue le fait majeur de l'intoxication. Alors que le surplus non éliminé du plomb se fixe schématiquement sur les os, pour le cadmium le foie et le rein sont les lieux d'élection, pour le mercure il faut y ajouter la rate et le système nerveux central.

2.11. Intoxication saturnine

L'intoxication aiguë par le plomb est exceptionnelle ; elle provoque une tubulo-néphrite avec anurie. Elle est presque toujours chronique, rarement professionnelle actuellement, souvent alimentaire surtout à partir d'eaux agressives à l'égard des canalisations en plomb. Le diagnostic est basé sur l'un des quatre éléments cliniques et sur les deux éléments de laboratoire suivants :

- colique de plomb (crises douloureuses, vomissements, avec souvent crise hypertensive); manifestations nerveuses (paralysie radiale ou troubles psychiques); hypertension artérielle (avec insuffisance rénale); liseré gingival grisâtre de Burton ;
- anémie fréquente (avec hématies ponctuées); augmentation de la coproporphyrine urinaire jusqu'à 3 500 µgr/l.

Le rapport entre la dose de plomb et la réaction clinique induite est difficile à évaluer à partir de la dose absorbée, compte tenu des différences individuelles au niveau de l'absorption, de l'excrétion, de la fixation, de la sensibilité, etc. Il faut donc utiliser pour l'établir la concentration sanguine qui représenterait le mieux la charge métabolique active.

Les concentrations urinaire, fécale ou salivaire, également utilisables, ne présentent pas d'avantage; il en est de même pour les cheveux qui ne fournissent que des informations additionnelles. Sur cette base, Chisolm donne les effets du plomb rapportés à cinq degrés d'exposition et à cinq taux d'absorption du métal. On constate que les effets nocifs n'apparaissent qu'à partir de 80 microgrammes, sauf affection intercurrente, et que les lésions fonctionnelles se pro-

(1) Une simple mention sera faite du cuivre et du zinc.

duisent lorsque la concentration en plomb dans le sang est au-dessus de ce taux, les effets résiduels de cette atteinte persistant lorsque les concentrations en plomb dans le sang sont redevenues normales.

Il est signalé que les ions calcium protègent d'une manière générale contre les atteintes saturnines au niveau de la muqueuse intestinale.

Mais, à ma connaissance, si l'étude des 23 ménages de la région de Glasgow (1972), portant sur 71 personnes, n'a pas montré de corrélation entre la concentration du plomb dans l'eau domestique et les douleurs abdominales et arthralgiques des sujets, elle a établi par contre la corrélation hautement significative d'une part entre les concentrations de plomb dans l'eau et dans le sang et la concentration du plomb dans l'eau et l'activité de l'ALA déshydratase, d'autre part enfin entre la concentration du plomb dans l'eau et l'acide δ aminolévulinique dans l'urine, avec chez 6 personnes une augmentation de la pression sanguine. Les sujets étaient tributaires soit de réservoirs en plomb pour l'eau froide, soit de canalisations en plomb supérieures à 60 pieds de long pour les uns et inférieures à cette dimension pour les autres.

La conclusion est que l'ingestion d'une eau de boisson contaminée par le plomb provoque des anomalies biochimiques sans que les consommateurs présentent forcément les symptômes d'intoxication caractéristiques. Le saturnisme n'est pas disparu ; il est possible d'en citer encore des cas récents.

2.12. Intoxication par le cadmium (maladie d'Itaï-Itaï)

Le cadmium est très utilisé en galvanoplastie ainsi que pour les joints de canalisations. Il accompagne le zinc dans les minerais.

La pathologie des intoxications cadmiées, tant au niveau de l'absorption que de la circulation et de l'excrétion, qui toutefois est lente, est superposable à celle du plomb. Mais elle s'en différencie quant au point de concentration dans l'organisme par une atteinte élective des reins et du foie, avec troubles enzymatiques, de la phosphorylation notamment.

Cette affection présente un complexe multiforme de symptômes, selon le stade de la maladie ou le niveau de l'atteinte sont consignés les signes cliniques suivants : diarrhée, tubulonéphrite avec crises de coliques néphrétiques, hypertension, anémie, anosmie, ostéomalacie plus ou moins généralisée, éventuellement perturbations dans le domaine génital par un mécanisme se rapportant à des lésions vasculaires.

Sauf dans les cas exceptionnels, on n'atteint pas la gravité clinique de la maladie d'Itaï-Itaï où l'apport de cadmium à l'organisme était, dans cette région située en aval des déversements d'une mine de zinc, de cadmium et de plomb, 10 fois supérieur à la normale, soit 600 μ g/jour.

A l'heure actuelle, l'eau constitue un maillon de la chaîne de contamination des aliments ; elle ne contribue cependant directement que pour une faible part à l'apport quotidien de cadmium à l'organisme, de 0,2 à 20 μ g/2 litres de consommation journalière. Etant donné d'une part toutefois les incidents signalés au Japon, d'autre part le fait que l'eau douce naturelle ou l'eau déminéralisée à pH légèrement acide extrait facilement le cadmium, et qu'enfin le cadmium, sous forme de stéarates ou d'autres sels, est utilisé comme charge du chlorure de polyvinyle, la teneur en cadmium des eaux de boisson, après avoir fait l'objet d'études nombreuses, reste très surveillée. Les normes internationales sont restrictives, l'intoxication étant toujours grave, et les recommandations vont jusqu'à demander, dans toute la mesure du possible, une limitation de l'emploi du cadmium et de ses composés pour les usages mentionnés ci-dessus.

2.13. Intoxications mercurielles Hydrargirisme professionnel (maladie de Minamata)

Le mercure est répandu dans la nature, mais inégalement ; il existe en des secteurs géographiques privilégiés ; on le rencontre dans les eaux souterraines à des taux particulièrement bas, inférieurs à 0,1 μg/l. Dans les eaux superficielles non polluées, il peut atteindre cette valeur, mais lorsqu'il existe des décharges industrielles (fabrique de chlore), les concentrations s'accroissent en fonction du flux de décharge. Sauf cas de pollution des eaux brutes, le mercure ne devrait pas intéresser directement les services de distribution d'eau, et l'on ne voit pas arriver de mercure métallique aux prises d'eau.

Sur le plan du métabolisme, les sels mercuriques inorganiques sont absorbés au niveau du tractus gastro-intestinal à raison de 15 %. Mais s'ils sont éliminés en quantités à peu près égales par les reins et les fèces, l'élimination rénale, étant donné sa lenteur, réalise les conditions d'une exposition prolongée et le rein constitue le site principal de l'accumulation. La concentration sanguine ou urinaire ne présente d'ailleurs qu'une valeur limitée comme indice d'exposition par rapport à la quantité stockée dans l'organe.

Par opposition, les composés organo-mercuriels franchissent la barrière intestinale sans grande difficulté. Les composés à chaîne courte sont beaucoup plus répartis et les points de concentration maxima sont les hématies et le cerveau, le rein et le foie. L'élimination se fait de la même manière que précédemment, mais n'a pas tout à fait la même signification, les concentrations sanguine ou urinaire présentant une certaine relation avec la concentration du mercure dans le système nerveux quant à l'appréciation du taux d'accumulation. A noter, comme pour l'arsenic, l'intervention du système pileux, facteur qui peut également être utilisé. Les composés à chaîne courte franchissent aussi la barrière amniotique et provoquent des atteintes fœtales irréversibles.

Les composés organo-mercuriels à chaîne longue présentent un métabolisme intermédiaire entre celui des sels inorganiques et celui des composés à chaîne courte.

La clinique des intoxications mercurielles (mercure et dérivés organo-mercuriels) repose sur quatre groupes de signes :

- signes digestifs avec stomatite et gastro-entérite ;
- signes rénaux avec anurie ;
- signes méningés avec tremblement et, parfois, perte de l'ouïe et réduction du champ visuel ;
- troubles psychiques (encéphalopathie).

On possède très peu de renseignements sur la toxicité chronique des sels de mercure (1), à l'exception du méthylmercure. Si la seule exposition au mercure provient de l'eau, et si tout le mercure présent s'y trouve à l'état de méthylmercure au niveau où celui-ci se rencontre dans les eaux douces non polluées, le risque encouru par la consommation de cette eau est inférieur au 1/10 de celui qui est induit normalement par l'alimentation. Ceci n'implique pas, en soi, un risque d'intoxication notable, mais pour des eaux polluées, il n'en est pas de même, surtout lorsqu'il y a, comme à Minamata, participation de la chaîne alimentaire. D'autant plus que l'on sait que les sels inorganiques de mercure, au cours de leur cheminement dans la zone d'interface vase/eau, sont méthylés et s'accumulent dans le phyto et le zoo planctons où les poissons trouvent leur nourriture.

2.14. Cas de quelques autres métaux

Intoxications par le cuivre et par le zinc

La présence en excès de cuivre et de zinc dans les eaux résulte soit de la corrosion des canalisations, soit d'une addition volontaire. Pour l'un comme pour l'autre de ces métaux usuels, il apparaît que les quantités ingérées au cours de la consommation d'eau sont suffisamment éloignées des quantités nécessaires pour induire des empoisonnements chroniques.

Pour le cuivre, en dehors des expositions de type industriel, on ne connaît pas de désordre chronique dégénératif. La maladie de Wilson, caractérisée par l'association d'une rigidité avec tremblement exagéré par le mouvement, une hépatite nodulaire avec cirrhose, l'anneau vert cornéen de Kaiser-Fleischer, l'hyperaminoacidurie avec élévation du cuivre plasmatique et urinaire, est liée à un défaut d'une globuline plasmatique de transport qui véhicule normalement le cuivre et à une augmentation du cuivre libre. Elle résulte donc d'une défectuosité métabolique et non d'une ingestion d'une quantité excessive de cuivre.

Pour le zinc, ainsi qu'il a été dit plus haut, à l'état pur la marge de sécurité dépend de la composition de base du régime et, en particulier, dans l'organisme, l'absorption et l'utilisation du zinc dépendent du cuivre, du fer et du cadmium présents dans la ration. La prise en considération de la corrosion des canalisations n'est donc pas sans fondement.

Cette pathologie du cuivre et du zinc, bien que non négligeable, n'est cependant pas du même ordre que celle du plomb et du cadmium, et la question de l'opportunité de l'emploi de ces métaux recouvre actuellement de telles implications socio-économiques et provoque de si nombreux échanges de vues qu'elle ne saurait être traitée aujourd'hui.

(1) Vous avez sans doute retrouvé comme moi dans une publication britannique la relation sur les circonstances de la mort de Charles II d'Angleterre (petit-fils d'Henri IV). Il a été dosé dans ses cheveux les quantités de mercure significatives confirmant son intoxication professionnelle, ce souverain pratiquant l'alchimie.

Le même problème se pose d'ailleurs pour l'asbestose qui intéresse l'utilisation des tuyaux d'amiante-ciment et qui, pour des raisons semblables, ne sera pas abordée ici.

Arsenicisme ou intoxication arsenicale

On distingue dans l'intoxication arsenicale par voie orale :

- des formes suraiguës, cholériformes, algicides ;
- des formes aiguës avec signes digestifs, éruptions cutanées, collapsus, atteinte hépato-rénale.

On bénéficie pour le diagnostic de l'affinité de l'arsenic pour les groupements SH de la kératine, d'où sa concentration au niveau de la peau et des phanères.

La présence d'arsenic en quantités dangereuses dans une eau de distribution ne peut être que d'origine accidentelle et le tableau clinique ci-dessus n'est jamais observé.

Les eaux naturelles arsenicales, d'origine hydrominérale, sont du reste depuis toujours utilisées pour leurs propriétés médicinales.

Intoxication par le chrome

La toxicité de ce métal est admise, mais elle varie selon le degré d'oxydation. Alors que la marge de sécurité entre la dose ordinairement ingérée et celle qui induit des effets dommageables pour le chrome trivalent est relativement élevée, par contre, pour le chrome hexavalent (chromates et bichromates alcalins), les expositions conduisent à la chronicité.

Les expériences sur animal montrent que, pour une ingestion de quantités dépassant 50 p.p.m., on a des ralentissements de croissance, des atteintes hépatiques et rénales. Naturellement, de telles quantités ne peuvent procéder que de déversements incontrôlés et les procédés habituels de traitement en place dans les usines productrices d'eau paraissent capables d'assurer un abattement significatif.

L'argyrisme

En dehors de l'absorption accidentelle de nitrate d'argent qui conduit à un état de choc avec signes digestifs intenses, l'argent est introduit dans l'organisme par voie orale lors de la consommation d'eaux traitées par les procédés électrophysiques ou d'eaux embouteillées stabilisées à l'argent.

Le transfert au milieu intérieur se fait par voie gastro-intestinale. L'argent présente une affinité pour la peau où il se fixe ; l'élimination par les voies urinaires devenant négligeable, on aboutit à l'argyrisme.

L'argyrisme, caractérisé par une décoloration grise permanente et indélébile de la peau, ne se produit que pour des doses dépassant le gramme d'argent fixé dans le corps humain. Il semble peu probable, sauf surcharge accidentelle des eaux ainsi traitées, que ce stade d'accumulation soit atteint pour une durée de vie moyenne. Toutefois, compte tenu du manque d'informations quant à la toxicité chronique de l'argent pour l'homme, il est nécessaire d'observer des normes limites strictes pour les eaux alimentaires.

2.2. Les composés organiques limités aux pesticides

Ces corps sont fort nombreux et représentent une pollution résiduelle, soit au niveau de leur production, soit au niveau de leur utilisation. Les procédés usuels de traitement des eaux présentent une efficacité faible ou nulle à leur égard.

Ils peuvent être groupés en trois classes principales :

2.21. Les insecticides organo-chlorés

(types DDT, HCH, aldrine, dialdrine, chlordane, toxaphène) qui ont tendance à s'accumuler dans les tissus graisseux plus rapidement qu'ils ne sont métabolisés, entraînent du point de vue clinique des signes généraux avec céphalées, vertiges et étourdissements, réduction du champ visuel, des signes digestifs avec diarrhée, lésions hépatiques et rénales, des signes respiratoires plus ou moins tardifs de type œdème aigu du poumon, des signes nerveux enfin allant du simple engourdissement des extrémités pour les cas moyens aux paralysies avec convulsions et contractures généralisées pour les cas sévères. L'évolution se fait de l'hyperirritabilité vers le coma avec collapsus.

2.22. Les insecticides organo-phosphorés

(types parathion, malathion, phosdrine), tout aussi dangereux que les précédents lorsqu'ils sont absorbés en petites doses et pendant une longue période de temps, provoquent un disfonctionnement de la cholinestérase, accompagné de signes digestifs, diarrhée, douleurs et vomissements, de signes respiratoires plus ou moins tardifs avec œdème aigu du poumon, des signes nerveux avec agitation, convulsions, l'évolution se faisant vers un collapsus et la mort pour des doses d'imprégnation très faibles.

2.23. Les herbicides ou fongicides de type chlorophénoxy

(chlorophénoxy, acétate de soude, ancien 2-4D) sont variables selon les espèces auxquelles ils s'appliquent. Ils agissent à très petites doses en entraînant, comme les dérivés nitrés des phénols, une hyperthermie avec cyanose, des troubles digestifs accompagnés de sudation et soif intense, des troubles respiratoires à type de tachypnée et des troubles nerveux avec angoisse, délire et convulsions. Ils sont, de plus, dotés d'un pouvoir tératogène en agissant au niveau de l'embryon.

Ces tableaux cliniques répondent en fait à des intoxications progressives ou accidentelles. Dans la pratique courante, de tels processus ne peuvent naître à partir d'une distribution publique normale, car il existe des signes prémonitoires importants, empoisonnement de la faune, odeur, saveurs anormales et caractéristiques résistant à tout traitement, avec imprégnation généralisée de l'environnement pour des doses infinitésimales à peine supérieures aux concentrations maximales admissibles prises en compte dans toutes les réglementations.

Pour l'ensemble de ces corps nous sommes donc très avertis et des associations se sont créées, tel par exemple le Centre international d'étude du Lindane, constitué à Bruxelles par arrêté royal du 23 avril 1974.

Mais que penser maintenant des composés organochlorés induits à l'occasion de la chloration des eaux ?... Dans l'expectative d'une meilleure information, on se doit, sans le condamner, de modérer l'usage de la préchloration des eaux ; l'application du break-point au niveau des eaux brutes paraît devoir être remis en cause. Deux réunions internationales spécialisées auront lieu cette année sur le sujet, l'une à Cincinnati, l'autre à l'Académie des Sciences à New York.

3. Maladies correspondant à la présence dans l'eau de bactéries, de parasites ou de virus ; risques à court terme

Nous abordons ici le domaine des maladies infectieuses, « vieille terreur de l'humanité », qu'elles soient endémiques, minant les forces vives des populations, ou épidémiques, soudaines, étendues, violentes, provoquant désolation, abandon de toute observance et démoralisation des survivants.

Il en fut ainsi de la lèpre jusqu'à la fin du moyen-âge, de la peste depuis l'antiquité jusqu'à la campagne d'Egypte, du typhus et des dysenteries depuis toujours, du choléra qui occupe la période contemporaine.

La présentation du sujet est délicate ; la littérature utilise soit la division par agent causal, virus, parasite, soit le répertoire des maladies soumises à déclaration, soit encore la symptomatologie clinique envisagée ou non par appareil, soit enfin la désignation des maladies elles-mêmes. Nous suivrons cette dernière formule en ne retenant que les affections les plus importantes ayant une étiologie hydrique :

- à transmission humaine directe, sans intermédiaire, les dysenteries, les typhoparatyphoïdes, les colibacilloses, le choléra ;
- à transmission indirecte nécessitant un hôte intermédiaire, les leptospiroses, la bilharziose, les distomatoses, les filarioses, les parasitoses intestinales communes, représentées dans nos contrées par les lambliases, les ascaridioses et les trichocéphaloses où l'eau n'est pas le vecteur unique et qui sont de conséquences pathologiques bénignes ;
- les maladies à virus, poliomyélite, hépatite épidémique.

3.1. Maladies à transmission humaine directe

3.11. Les dysenteries

Quel que soit l'agent causal, les dysenteries sont caractérisées par un syndrome gastro-intestinal typique comportant :

- des douleurs abdominales avec coliques, épreintes, ténesme ;
- des expulsions de selles non fécales, nombreuses (4 à 20 par jour), glaireuses et sanguinolentes ;
- une altération de l'état général avec déshydratation, amaigrissement, adynamie.

L'examen des selles conduit à la distinction entre les dysenteries bacillaires et les dysenteries protozoaires.

Les dysenteries bacillaires, fréquentes dans nos régions européennes, sont rapportées au bacille de Shiga, les bacilles de Strong, de Hiss et Flexner, de la même famille, ayant des actions moins caractéristiques. Ces affections débutent brutalement avec une élévation de température coexistant avec l'apparition du syndrome décrit ci-dessus ; elle cède en général en une dizaine de jours, pouvant en certains cas, en l'absence d'une thérapeutique adéquate, laisser place à des complications telles que arthrites, phlébites, ou donner lieu à l'apparition de formes hypertoxiques ou cholériformes à mortalité parfois élevée. Toutefois, dans nos régions, l'origine strictement hydrique de la shigellose, tout en restant possible, est rare.

Les dysenteries à protozoaires. Si elles correspondent au syndrome dysentérique commun, elles débutent en général plus insidieusement que les précédentes et ont tendance, en se prolongeant, à passer à la chronicité, laissant aussi place parfois à des complications, ici hépatiques ou pulmonaires (abcès). C'est de ce groupe que relève la dysenterie amibienne, apanage des sujets ayant séjourné dans les pays chauds, caractérisée par la présence de l'amibe ou de ses kystes dans les selles. Mais à côté de cette amibe classique apparaissent maintenant des amibes libres du groupe Limax, genre Naegleria, fréquemment isolées dans le monde entier à partir des eaux de lacs, de piscines ou de distribution, qui, après franchissement de la muqueuse nasale, sont capables de provoquer, sans manifestations dysentériques, une méningo-encéphalite primitive rapidement mortelle. Il y a là un problème nouveau dont il faudra tenir compte en considération du réchauffement artificiel et continu des eaux superficielles à des températures supérieures à 22°, nécessaires au développement du genre et qui sont réalisées dans les piscines où ont été contractés les cas de méningite qui ont cristallisé l'attention (1).

Ce point particulier de la pathologie hydrique nous met en garde contre le fait que les amibes du groupe Limax, déjà dangereuses sous certaines conditions par elles-mêmes, se nourrissant de bactéries, peuvent véhiculer des virus ; ainsi les eaux les contenant doivent être considérées, jusqu'à meilleure information, comme impropres à la consommation.

3.12. Les typhoparatyphoïdes

Ce sont des septicémies (2) dues au bacille d'Eberth ou aux bacilles paratyphiques, caractérisées par la fièvre, le tuphos (stupeur et abattement extrême), des complications graves possibles (collapsus cardio-vasculaire, hémorragies intestinales, avec ou sans perforation).

Quatre germes de la famille des Salmonella sont isolément en cause, le bacille d'Eberth, le para A, le para B, exceptionnellement le para C. Ils ne se distinguent les uns des autres que par des réactions chimiques ou biologiques, mais ils induisent une symptomatologie unique.

Durant la première semaine, ils passent dans le sang, puis dans les urines, et se retrouvent dans les selles pendant toute la maladie et quelquefois plusieurs mois après. Il existe également des sujets dits porteurs sains qui évacuent ces germes toute leur vie et n'éprouvent aucun dommage pour leur propre état de santé.

La contamination se fait par voie digestive, à partir de l'eau contaminée par des matières fécales ou d'eaux non potables, des mains sales, des aliments souillés. Le bacille traverse la muqueuse intestinale sans la léser et va se fixer dans les ganglions mésentériques pendant l'incubation. Ensuite il envahit le système circulatoire, entraînant une septicémie ; l'élimination se fait par les voies biliaires et les selles en libérant une endotoxine neurotrope, qui lèse le système neuro-végétatif abdominal avec ulcérations intestinales et le SN Central avec troubles généraux.

Les typhoparatyphoïdes représentent les épidémies type d'origine hydrique ; elles ont certainement, bien que non isolées en tant que syndromes, fait partie des grandes dysenteries historiques. Grâce à la typhomycine (chloramphénicol), les choses ont grandement évolué, mais si l'on ne meurt plus de typhoïde dans nos pays occidentaux (1), dans d'autres contrées, moins pourvues, le danger subsiste, bien qu'à un moindre degré qu'autrefois.

Dans le cadre de la surveillance des Salmonella en 1973, publiée par l'Organisation Mondiale de la Santé le 30 juin écoulé, il n'est signalé qu'une seule épidémie associée à S. typhi, survenue dans un camp de travailleurs immigrants aux U.S.A., dont la cause a pu être rapportée à la contamination de l'approvisionnement en eau. La plupart des cas transmis à l'O.M.S. durant cette période étaient liés en général à une alimentation de qualité suspecte.

On assiste en ce moment à un regain d'intérêt pour les Salmonella du fait de l'urbanisation, de la croissance démographique et de l'augmentation concomitante des volumes d'eaux usées rejetées dans les milieux naturels réutilisés pour produire l'eau d'alimentation, du fait également de la survivance, sinon de la multiplication des Salmonella à travers les processus épuratoires mis en œuvre. De l'enquête menée par nos collègues belges (2), il résulte qu'il est possible actuellement d'obtenir, à partir d'une rivière donnée, pour 187 prélèvements effectués sur 43 sites, 38 % de résultats positifs répartis en 27 sérotypes différents (3), parmi lesquels les paratyphiques B sont en nombre assez faible (6 % sur 319 prélèvements).

Ce travail confirme les conclusions de la surveillance des Salmonella par l'O.M.S. qui retient qu'en dehors des propagations nosocomiales (4) on assiste

(1) J. Lapierre : 60 cas décrits ; sujets de 8 à 27 ans. Incubation 5 à 9 jours. Rhinopharyngite avec perturbation du goût et de l'odorat. Début brutal. Température 39-40°, malaise, céphalées violentes, nausées, vomissements. Les signes d'irritation méningée apparaissent en 2 ou 3 jours, relayés par l'atteinte encéphalitique, avec inconscience confinant au coma, quadriplégie flasque, avec parfois convulsions et signe d'ataxie cérébelleuse.

(2) Nom générique des maladies causées par l'introduction dans l'économie d'un germe infectieux qui s'y développe sans susciter de réaction locale.

(1) Restriction faite pour l'endémie signalée en Campanie en 1974, la Salmonella Wien en cause accusant une polyrésistance aux antibiotiques.

(2) J. Barbette. Institut d'Hygiène et d'Epidémiologie, Bruxelles.

(3) De Maeyer-Clumpoel (Inst. Hyg. et Ep. Bruxelles). Les sérotypes le plus fréquemment isolés ont été S. Brandebourg, S. Panama, S. Typhimurium, S. Paratyphi B.

(4) Dépend des hôpitaux.

à une diffusion de nouveaux sérotypes de par le monde à partir des denrées alimentaires, des ingrédients contaminés et des animaux familiers de l'environnement humain infestés.

3.13. Les colibacilloses

Ce terme, selon HAMBURGER, « ne répond à aucune entité morbide définie, en dehors de la pyélonéphrite colibacillaire ou des infections générales à colibacilles (septicémie à colibacilles) ».

En ce qui concerne les pyélonéphrites, sans nous y arrêter, rappelons qu'elles impliquent la présence de causes favorisantes, infection intestinale, lithiase, infection ou hypertrophie prostatique, ou toute affection pouvant apporter une gêne à l'évacuation urinaire (grossesse, fibrome, coudure urétérale).

Pour la septicémie à colibacilles, en dehors de la notion d'une porte d'entrée intestinale, appendiculaire, biliaire ou urinaire, il n'existe aucun élément de diagnostic ; l'hémoculture elle-même est d'interprétation difficile en raison de la fréquence des bactériémies simples et passagères.

Les coliformes en cause correspondent à la tribu des Escherichiae Bergey qui comprend trois genres dont celui des Escherichia (type E. Coli), le genre Aerobacter (type A. aerogenes), le genre Klebsiella (type K. pneumoniae).

L'Escherichia Coli, englobé dans les coliformes fécaux, est un saphrophyte constant du tube digestif qu'il envahit dès les premières heures de la naissance et où il mène une existence « non pas précaire, mais florissante » dans les matières fécales où il se multiplie et se trouve par milliards chez l'homme et tous les animaux. D'après Brisou, il représente 50 % de la flore intestinale.

Il peut acquérir un pouvoir pathogène chez l'homme soit en provoquant une lésion de voisinage péri-intestinale, soit plus souvent en passant dans le sang et déterminant une septicémie qui, lorsqu'elle est grave, prend l'allure de la fièvre typhoïde avec localisation possible pulmonaire, biliaire, rénale. Mais il s'en faut de beaucoup que toutes les décharges d'E. coli dans le sang déclenchent des septicémies ou des infections urinaires. Dans de très nombreux cas, la bactériémie et la bactériurie se font sans causer de désordre appréciable.

Dans quelle mesure l'eau participe-t-elle à l'apparition des colibacilloses ? Les réponses sont très modestes à cet égard. Mme DE MAEYER signale que des expériences sur volontaires ont montré que, par voie orale, une dose comportant 10^9 E. coli revivifiables, des sérotypes 0^{55} ou 0^{111}, est susceptible de provoquer chez l'homme des troubles diarrhéiques, alors que la même dose d'E. coli non sélectionnés en est incapable. On peut donc admettre sur cette base que la participation directe à la pathologie hydrique des E. coli de l'eau est aléatoire. Mais, étant donné la corrélation existant entre les coliformes fécaux auxquels appartient ce germe et les Salmonella, on ne saurait sans danger utiliser directement des eaux au sein desquelles la présence des coliformes fécaux est reconnue.

3.14. Le choléra

On sait que le choléra sévit dans l'Inde depuis toujours ; il ne quitte guère la péninsule. En 1817, l'épidémie gagne Ceylan, les îles Maurice et Bourbon, Zanzibar, la Birmanie, les Philippines. En 1819, elle apparaît au Siam, en Indochine et en Chine ; en 1821, elle atteint la Perse, puis la Caspienne où l'on boit l'eau des citernes ou celle que véhiculent les canaux des montagnes avec les eaux de lavage du linge. En 1823 elle est en Asie Mineure, Mésopotamie, Syrie, mais ne passe ni en Egypte ni en Europe. Les pays d'Occident, l'épidémie apaisée, voient disparaître leurs craintes et ne songent plus à l'hygiène (1)

En 1826, le choléra réapparaît dans le Bengale ; en 1827 il passe en Afghanistan, puis dans le Turkestan et, en 1829, à Astrakan ; il dévaste la Perse, il ravage la Russie, s'introduit en Allemagne du Nord et, en 1831, en Angleterre. En 1832 il visite Londres. Le 22 mars 1832 exactement, il fait sa première victime rue Mazarine à Paris où il sévit, ainsi qu'en Europe, jusqu'en 1837, ayant fait plus d'un million de victimes, dont 400 000 en Russie, 340 000 en Autriche, 100 000 en Espagne, 95 000 en France, partout ayant donné lieu aux scènes d'horreur propres aux grandes épidémies. Mais la leçon, si dure fût-elle, est à l'origine, tout au moins en France, d'un effort extraordinaire à l'égard des eaux. Cependant, de 1847 à 1856, une nouvelle épidémie de choléra dévaste l'Europe où elle fait plus de victimes encore que la précédente. Elle resurgit encore de 1865 à 1873, à Marseille notamment. Mais l'agent causal, le Vibrio coma, est découvert par Koch en 1884... Le choléra fait une nouvelle incursion en 1892, puis disparaît, du moins en tant que grande épidémie (2).

Pendant un siècle, il a été pour l'Europe la maladie la plus redoutée. « Toute l'organisation sanitaire des villes d'Europe et d'Amérique est fille du choléra... » En 1923, croyant en avoir terminé avec lui, on pense que le choléra perdait de son importance sur le plan de la santé publique, sauf dans quelques pays où il était endémique. Mais, en 1961, il revint en force. L'agent causal était alors le Vibrion El Tor... C'était le début de la septième pandémie qui se poursuit actuellement (3). Partie une fois de plus de l'Extrême-Orient, la maladie, après avoir gagné certaines régions européennes, s'installe en Afrique où aucun cas n'avait été signalé pendant près de cent ans. Mais aujourd'hui, le choléra, tout en restant une affection grave, importante internationalement, grâce à la vaccination et aux nouvelles techniques semble moins redoutable... Ce qui n'a pas empêché, étant donné le brassage des populations, notamment en Europe, au cours des étés, de prendre des dispositions sanitaires appropriées qui, il faut le reconnaître, à part quelques cas connus, se sont révélées efficaces.

Du point de vue clinique, le choléra est une maladie à incubation très courte, variant de quelques heures à cinq jours ; il est caractérisé par une diarrhée profuse, à grains riziformes, de plusieurs litres par jour, avec vomissements et douleurs épigastriques, l'algidité, la température s'abaissant à 35°, 34° ; ce tableau s'accompagne de crampes musculaires et d'anurie. En l'absence de l'administration de sulfamides ou autres antiseptiques intestinaux, de réchauf-

(1) A titre d'exemple, à Paris de 1806 à 1825, la construction d'égouts ne dépasse pas 500 mètres/an ; en 1824, le réseau a 37 km de long ; il se déverse dans la Seine en plein Paris.

(2) « Les épidémies et l'histoire ». A. COLNAT.

(3) Santé du Monde, juillet 1976. BARNA et THAPALYAL.

lement du malade, et surtout de réhydratation (1), la mort survient, parfois en quelques heures. On n'a pas été sans remarquer le changement de variété du vibrion cholérique entre la sixième et la septième pandémie.

Le choléra ne mériterait pas cette évocation s'il n'était l'une des maladies hydriques par excellence et bien que tous les vibrions isolés des eaux ne soient pas pathogènes (2), il est difficile de protéger la ressource en eau de cette contamination en période épidémique si l'on n'a pas pris par avance les mesures nécessaires. Pour la France, on peut noter l'efficience des instructions préventives de la Direction Générale de la Santé lors des événements récents (3).

3.2. Maladies à transmission indirecte

3.21 Les leptospiroses

Les leptospiroses ou spirochétoses sont responsables d'ictères infectieux avec atteinte rénale, réalisant des hépato-néphrites ; toutefois, de nombreuses leptospiroses ne provoquent pas d'ictère. Celles qui intéressent l'eau ont pour vecteur commun les urines des rongeurs, réservoirs de virus. On peut citer :

les leptospiroses ictéro-hémorragiques (avec Leptospira icterohemorragiae de Inada et Ido). C'est la maladie des baigneurs en rivière et des égoutiers, des pêcheurs éventuellement, la contamination s'effectuant par voie transcutanée ; vecteur : le rat.

la fièvre des marais (Leptospira grippo typhosa) ; vecteur : le campagnol.

la maladie des rizières italiennes (Leptospira bataviae); rongeurs.

Du point de vue des symptômes, la forme ictéro-hémorragique, courante en France, est à début brutal, avec syndrome méningé souvent discret, algies musculaires, herpès, congestion du visage, injection conjonctivale, atteinte rénale avec azotémie ; rechute ou reprise fébrile le 15e jour ; l'ictère flamboyant apparaît au 4e ou 5e jour.

3.22 La bilharziose ou schistosomiase

Très fréquente en Amérique du Sud, en Afrique (Egypte notamment), en Asie Mineure, en Europe méridionale, Chine, sud du Japon, Philippines, cette affection présente une extension inquiétante qui paraît consécutive aux efforts tentés pour le développement des irrigations en pays subtropicaux. L'agent pathogène, Schistosoma, est contracté parfois au cours de bains de rivières, mais aussi à l'occasion de la marche à pieds nus dans les ruisseaux des rizières et les marécages. C'est une infestation par voie transcutanée. L'hôte intermédiaire est un mollusque de genre Bulinus, le bullin d'eau douce.

Le diagnostic est difficile par la voie clinique ; on se trouve devant des états infectieux avec hépatomégalie, urticaire, hématurie dans le cas de bilharziose vésicale, diarrhée, parfois syndrome cirrhotique (bilharziose intestinale). Confirmation doit toujours être faite par une numération globulaire, qui montrerait une éosinophilie importante, et une recherche des œufs de parasites dans les selles ou les urines.

3.23. Les distomatoses hépatiques ou fascioloses

Les distomatoses sont caractérisées par la présence de douves dans les canaux biliaires extra ou intra-hépatiques. L'agent pathogène est Fasciola hepatica, parasite habituel des herbivores, les moutons surtout. L'hôte intermédiaire est une limnée. La contamination se fait principalement par la consommation de cresson infesté par les cercaires, larves de la douve. En Extrême-Orient, les douves parasitent les animaux domestiques.

Le diagnostic est basé sur l'association d'un gros foie, d'un ictère et d'une très forte éosinophilie sanguine ; il ne peut être établi que par un examen parasitologique des selles et de la bile.

Cette affection, assez fréquente, devrait conduire à l'interdiction de la commercialisation de cresson provenant d'installations non contrôlées.

3.24. Les filarioses, la filariose de Médine ou dracunculose

Observables dans les pays tropicaux, ces affections sont dues à des nématodes du genre Filaire. La filaire de Médine ou ver de Guinée est celle qui concerne cet exposé. Elle nécessite le passage par un hôte, le cyclops, petit crustacé qui, absorbé lors de la consommation de l'eau souillée, rétablit le cycle de l'infestation, l'adulte se développant chez l'homme.

Le diagnostic est basé sur des poussées de lymphangite évoluant vers des éléphantiasis localisés aux membres inférieurs.

3.25. Les parasitoses intestinales communes

La lambliase a pour vecteur des flagellés, Lamblias ou Giardias, qui parasitent le duodénum et la vésicule biliaire. Leurs kystes, expulsés par les selles, sont à l'origine de la contamination des eaux. Le diagnostic de l'infection est basé sur une diarrhée avec asthénie, amaigrissement, cholécystite parfois.

L'ascaridiose est due à des nématodes, Ascaris lombricoïdes, dont la larve, après un trajet complexe, finit par donner la forme adulte dans l'intestin, de 10 à 20 cm de long, d'où un syndrome douloureux intestinal, des troubles digestifs, neurovégétatifs, hématologiques, avec anémie et éosinophilie. La contamination s'effectue par les eaux et les légumes souillés ainsi que par les matières fécales contenant les œufs des parasites.

La trichocéphalose est provoquée par un petit nématode de 3 à 4 centimètres de long qui vit dans le coecum et l'appendice en donnant des troubles digestifs discrets avec douleurs dans la fosse iliaque droite faisant penser à l'appendicite chronique. L'infestation se fait par voie orale par absorption de l'œuf du parasite.

Pour ces trois formes d'affections, le diagnostic repose sur la recherche des œufs de parasites dans les selles. L'infestation se produit à partir d'un environnement non protégé.

(1) L'O.M.S. vient de mettre au point une réhydratation par voie orale (Santé du Monde, juillet 1976, pp. 30, 31).

(2) Gastinel Bactériologie p. 519.

(3) Dr Charbonneau, Pr Aldighieri, Dr Cassaigne.

3.3. Les maladies à virus

Dans la pathologie infectieuse, les virus occupent une place importante, surtout depuis 1949, date à laquelle Enders démontre la possibilité d'isoler les virus par culture cellulaire. Du point de vue de la pratique médicale, les virus ont été répartis schématiquement en deux groupes :

- les virus respiratoires, à transmission par voie pharyngée et conjonctivale et par contact direct, avec les sécrétions respiratoires (virus de la grippe, de la variole, de la varicelle, des oreillons, de la rougeole, de la pneumonie atypique ; les adénovirus ou virus APC (1), 16 types chez l'homme ; les REO virus, non entéro-virus, peuvent leur être associés);
- les entéro-virus, à transmission par voie digestive, où figurent les virus poliomyélitiques types I, II, III ; les virus Coxsackie 28 types (2), les Echo virus 24 types (3), le virus A de l'hépatite épidémique, le virus B de l'hépatite d'inoculation.

L'expansion du domaine des entéro-virus peut expliquer certaines confusions de cas frustes ou atypiques avec la poliomyélite, alors que le syndrome clinique est provoqué par exemple par un virus ECHO (orphelin) ou par certains virus Coxsackie... Dans les collectivités où sévissent de telles épidémies, on rencontre fréquemment des formes inapparentes, parallèles.

Nous ne traiterons ici que des maladies majeures relevant des entéro-virus, « dont l'origine hydrique est considérée comme possible » et où les virus responsables éliminés par les émonctoires sont repris par l'homme à partir des milieux de transfert, la poliomyélite, l'hépatite virale notamment.

3.31. La poliomyélite antérieure aiguë

C'est la paralysie infantile, maladie de Heine-Médin. Elle est endémo-épidémique et due au virus poliomyélitique dont on connaît actuellement trois grands types comprenant plusieurs souches chacun, parmi lesquelles Brunehilde pour le premier, Lansing pour le second, Léon (Armstrong) pour le troisième.

L'agent causal se rencontre :

- dans le rhino-pharynx et surtout dans les matières fécales du malade ; l'élimination par les selles est massive et prolongée ;
- dans les selles des convalescents et des sujets sains porteurs de germes, les cas de maladie inapparente étant excessivement nombreux : 99 % ;
- dans les eaux d'égout épurées ou non, à l'issue des fosses septiques et sur les légumes pollués par les déjections.

La contamination se fait par voie digestive surtout, par l'intermédiaire de l'eau, des aliments, sur un mode rappelant celui de la fièvre typhoïde, ainsi que plus rarement par le rhino-pharynx.

Le sujet contaminé fait soit une poliomyélite-infection, immunisante, cas fréquent, soit une poliomyélite-maladie, cas plus rare.

Le diagnostic est basé sur une symptomatologie pseudo-grippale de quelques jours et l'installation brusque des paralysies qui atteignent d'emblée tous les muscles qu'elles doivent atteindre, paralysie flasque avec abolition des réflexes et atrophie musculaire précoce sur les muscles qui resteront atteints. On ne note pas de trouble sensitif ou sphinctérien dans la forme classique, mais dans les formes épidémiques peuvent apparaître de fortes réactions méningées ou des signes encéphaliques graves.

Quant à l'évolution, en dehors des paralysies respiratoires qui nécessitent une assistance rapide, les séquelles paralytiques, avec troubles trophiques, déformations et attitudes vicieuses, peuvent être définitives. Toutefois, elles sont toujours moins étendues que l'atteinte primitive.

La vaccination, qu'elle soit pratiquée avec les vaccins vivants ou les vaccins inactivés, a totalement transformé le pronostic. Mais le danger d'imprégnation virale des populations subsiste néanmoins, surtout dans nos pays où l'hygiène est, dit-on, très développée. A cet égard, il ne paraît subsister aucun doute quant au rôle des eaux usées, épurées ou non, et de leurs milieux récepteurs et de transport dans la dispersion des virus. Par contre, au niveau de la reprise, l'inactivation de ces virus n'est encore ni infirmée, ni confirmée en dehors de la mise en œuvre d'équipements spéciaux connus. Les travaux des virologues de tous pays en matière d'isolement de petites quantités de virus, en leur état d'avancement actuel, devraient pouvoir permettre d'obtenir la réponse attendue dans les toutes prochaines années.

3.32. L'hépatite épidémique

Cette affection fait partie des hépatites virales qui représentent la cause la plus fréquente des ictères par hépatite. Deux virus sont plus particulièrement responsables, le virus A (I.H. = infection hepatitis) qui correspond à l'hépatite épidémique, le virus B (S.H. serum hepatitis) qui correspond à l'hépatite d'inoculation.

Seule l'hépatite A nous intéresse aujourd'hui.

L'hépatite épidémique débute par des troubles digestifs, avec anorexie et parfois vomissements. L'ictère apparaît progressivement ; l'hépato-mégalie, bien que fréquente, n'est pas constante. En outre, certaines hépatites sont anictériques. De très nombreux travaux sont consacrés à l'étude étiologique des hépatites virales, de grandes incertitudes persistant du fait que l'on ne dispose pas encore d'animal de laboratoire réceptif, que ni l'œuf embryonné ni les cultures de tissus n'ont pu être infectés. Ce n'est que sur des échantillons de sang, passés en série sur culture cellulaire, qu'il a été possible, après centrifugation du milieu, de constater, au microscope électronique, la présence de particules de 12 à 18 μ, absentes dans les essais témoins. Ces éléments, d'une taille moitié de celle des entéro-virus, apparaissent comme les plus petits virus connus.

L'évacuation des virus se fait par les selles et leur transfert par l'eau et les légumes souillés. Il existe des porteurs sains. Le virus est transmis par voie digestive.

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(1) APC = adénoïdal pharyngal conjonctival virus.

(2) Coxsackie : méningite lymphocytaire, herpangine, pharyngite fébrile de l'enfant, myalgie épidémique, radiculo névrite aiguë, myocardite et péricardite aiguë infantile, syndromes de type poliomyélitique.

(3) ECHO = enteric cytopathogenic human orphan viruses (virus orphelin) : syndrome grippal méningitique aseptique : diarrhées estivales des nourrissons, syndromes de types poliomyélitiques...

L'évolution de l'hépatite virale peut se faire en quelques jours ou durer jusqu'à trois mois. La guérison est habituelle dans les formes bénignes de type catarrhal, mais des complications peuvent survenir avec apparition d'œdème et d'ascite, de dyskinésies vésiculaires et insuffisances pancréatiques. La mort est toujours possible par ictère grave.

Etant donné leur sévérité, les hépatites virales font l'objet d'études statistiques très importantes. C'est ainsi que, pour Paris, on a enregistré, en 1975, sur 294 cas d'hépatite virale, 184 cas présumés A, soit 62 %, les coquillages intervenant dans 13 cas, les voyages dans 21, le contact direct dans 16 cas. La répartition est faite par tranche d'âge jusqu'à 30 ans.

4. Réflexions

Au terme de cet exposé, plusieurs questions se posent :

- *en ce qui concerne les risques à long terme par carence ou par surchage.*

Si le goître et les caries dentaires peuvent être combattus efficacement, il est plus difficile d'éviter les effets cardio-vasculaires à une époque où le souci d'un fonctionnement optimum des installations à l'intérieur des immeubles et la recherche de la satisfaction du confort des usagers conduisent à modifier volontairement la structure physico-chimique des eaux. De ce point de vue, évitant le cas particulier des eaux naturellement douces qui sont en harmonie avec les terrains du gîte aquifère et avec lesquelles le consommateur autochtone vit en symbiose, il convient d'être attentif aux conséquences du déséquilibre volontaire introduit par la pratique privée ou collective de l'adoucissement systématique des eaux de dureté moyenne. Des études épidémiologiques prospectives sur la faculté d'adaptation des populations, en étroite liaison avec les techniques de production, de distribution et d'utilisation des eaux, devraient être développées au sein de l'association.

- *en ce qui regarde les risques à moyen terme par accumulation* correspondant à la présence d'éléments toxiques dans l'eau, hormis les graves inconvénients résultant de la corrosion (saturnisme), c'est la coexistence des décharges industrielles et des prises d'eau qui est encore en jeu. Ceci ne peut trouver de solution que dans une coordination raisonnable entre d'une part la qualité des rejets industriels autorisés en amont des prises d'eau (1) et, d'autre part, la qualité des eaux extraites auxdites prises d'eau (2). Les différents modes d'expression utilisés pour ces deux données ajoutent aux difficultés fondamentales et, compte tenu des impacts économiques, une mise au point raisonnable nécessitera probablement des délais assez longs.

C'est pourquoi il appartient aux distributeurs d'eau, publics ou privés, pour la garantie de qualité de la fourniture dont ils doivent répondre envers la population desservie, d'exiger des autorités responsables, selon leurs pays respectifs, le strict respect des concentrations maximales admissibles fixées pour les corps toxiques répertoriés sur les listes internationales et dont la plupart ont été souscrites par l'industrie.

- *en ce qui regarde les risques à court terme, microbiologiques et parasitaires*

Trois facteurs concourent à l'évaluation de ces risques : la persistance des germes émis dans le milieu extérieur, les eaux destinées à l'alimentation ; la transformation de ces germes dans le même milieu ; la quantité de germes nécessaires (dose infectante) pour faire apparaître la maladie chez l'homme. On trouve difficilement dans la littérature des considérations concomitantes sur ces trois aspects de la vie des organismes agresseurs.

- *La persistance* des germes microbiens est en général fonction de l'intensité de la contamination, des facteurs physiques influençant le milieu, des facteurs nutritionnels (richesse en matières organiques) et de la concurrence vitale. Généralement, l'intensité de la contamination, la présence de matières organiques, le froid sont des facteurs favorables à la persistance ; la température, l'insolation, le pH inférieur à la neutralité, la minéralisation sont des facteurs défavorables à la longévité. Parmi les données fragmentaires recueillies, on peut noter à titre d'exemple :

- l'Eberth résisterait deux semaines à 37° et jusqu'à neuf semaines aux températures inférieures à 10°;
- l'E. coli, à 18°, persisterait huit semaines ; les Shigellas quatre semaines à la même température; les entérocoques sept à huit semaines ;
- les leptospires seraient très fragiles et ne résisteraient pas plus de deux heures à la lumière solaire, le froid maintenant leur virulence...

La remarque la plus intéressante à retenir semble être celle correspondant au vibrion cholérique, dont la survivance maximale ne dépasserait pas 8 jours en rivière et 11 jours en mer dans les conditions climatiques du Japon. Cette observation a été utilisée d'une part pour réglementer les irrigations, d'autre part comme moyen de décontamination grâce à la mise en réserve des eaux supposées contaminées deux semaines avant leur consommation. Il y aurait intérêt à préciser les connaissances dans ce domaine de la persistance.

- *La transformation des germes dans les milieux extérieurs.* La longévité des germes n'a pas pour parallèle obligatoire une même durée de leur virulence, puisque la structure antigénique des microorganismes, dont dépendent toutes les réactions de spécificité, n'est pas conservée dans le temps. Or, de cette structure résulte leur pouvoir pathogène. A titre d'exemple, le facteur VI de l'Eberth disparaît au cours du vieillissement du germe, entraînant une diminution de son pouvoir pathogène. L'inversion d'un tel processus est rare dans les conditions spontanées.

- *Dans le domaine de la dose infectante,* ou quantité de germes nécessaires pour faire apparaître un processus clinique, on ne trouve que les résultats d'expériences sur des volontaires ; ils sont donc peu nombreux, mais leur intérêt est grand. Les travaux de HORNICQ et WOVDWARD de 1966, toujours cités, montrent que pour une suspension dans du lait de 10^9 germes (S. typhi), 95 % des volontaires ont présenté des signes cliniques typiques.

(1) Exprimée en flux par produit et par rapport à la production.

(2) Exprimée en concentration.

Pris dans leur état, ces résultats conduisent à penser que les quantités nécessaires pour provoquer une épidémie ne se rencontreraient jamais dans les eaux. Or, on s'est aperçu que l'acidification gastrique, dans le cas d'utilisation orale d'une suspension dans le lait de bacilles typhiques, était excitée et que, par voie de conséquence, la quantité de germes nécessaires dans ce cas pour provoquer la maladie était plus grande que dans l'eau, dont l'absorption, en provoquant une variation positive du pH du suc gastrique, facilite le franchissement de la barrière intestinale, diminuant les défenses. Il s'ensuit que WATANABE, en 1973, présume que la dose de S. typhi infectante dans l'eau de distribution publique au cours d'une épidémie n'est pas supérieure à 100 organismes. Si l'on considère la quantité de germes éliminés par gramme de matières fécales, on doit admettre avec MOORE que l'apparition d'une épidémie dans le domaine d'une distribution publique nécessite la présence fortuite, ou accidentelle, d'une intercommunication (bypass) avec des eaux fécalement contaminées, ce qui traduirait une faute lourde contre l'hygiène. Ces éléments, relativement nouveaux, montrent le danger présumé, mais encore mal apprécié, de la double canalisation.

- *En ce qui concerne les virus*, les dernières estimations relèvent que la plus faible dose qui puisse être détectée, 1 p.f.u., est infectante, mais ne conduit pas nécessairement à des manifestations cliniques, le porteur étant capable cependant de constituer une menace pour les autres membres de la communauté. Ces précisions ont été acquises sur la base des travaux de PLOTKIN et KATZ, en 1967, qui montrent que, pour 1 p.f.u., 30 % des sujets sont infectés.

On sait aujourd'hui qu'à 1 p.f.u. correspond toujours plus d'une particule virale ; on en dénombre 40 au microscope électronique pour la vaccine. Ces particules ne sont d'ailleurs pas toutes dans le même état ; elles s'entraident en quelque sorte.

Ceci justifie les travaux poursuivis actuellement en microscopie électronique (1) sur la portée de l'atténuation naturelle des virus dans l'eau et leur capacité résiduelle d'infestation. Il s'agit d'une orientation particulièrement intéressante pour les distributeurs d'eau, qui aurait l'avantage de permettre d'apprécier l'impact sur la santé publique de décharges virales des eaux usées reprises après un certain parcours par les prises d'eau. A signaler également, dans le même ordre d'idées, l'opération inverse (1) menée sur l'adaptation du virus SF3 de la grenouille qui, à 25°, pénétrant dans les tissus humains, ne s'y multiplie pas, mais prendrait cette fonction à une température plus élevée, ce qui, sur le plan virologique, aurait une signification semblable à ce qui a été observé avec les amibes du groupe Limax.

- *En ce qui regarde l'immunité*, celle-ci intéresse les populations ou l'individu, qu'elle soit naturelle ou acquise. Une épidémie ne peut naître que s'il y a simultanément augmentation momentanée du nombre et de la virulence des germes ou des particules infectantes et diminution de l'immunité. Les porteurs sains participent donc à l'entretien du niveau de l'immunité par les affections infra-cliniques qu'ils provoquent. Ces formes inapparentes s'opposent aux formes légères et aux formes lourdes, cliniquement décelables.

Questions de dernière heure

Deux points restent à inscrire au niveau de nos réflexions :

- *la radioactivité résiduelle due au tritium*, résultant de la production d'énergie par voie nucléaire, qui n'est arrêtée à aucun niveau et dont on admet qu'elle reste supportable à un niveau relativement élevé.
- *l'hyperchloration des eaux* qui conduit à la dispersion d'organo-chlorés de seconde formation et à leur diffusion dans la population, alors que tous les efforts sont déployés pour les arrêter à la source. De ce fait, et du point de vue de la santé publique, le problème de la préchloration des eaux est à reconsidérer dès maintenant.

En conclusion, il serait souhaitable que la rédaction des fiches sanitaires en cours de création au sein des Communautés par la Direction de la Protection Sanitaire à Luxembourg soit activement poursuivie avec la participation effective des distributeurs d'eau et des industriels.

Certes, il convient de constater les progrès réalisés à l'échelle mondiale dans l'hygiène de l'eau au cours des dernières décennies en matière de distribution d'eau. Néanmoins, pour préciser les apports dus à l'eau d'alimentation dans la ration alimentaire, de nouveaux travaux sont à poursuivre.

Paris, le 19 août 1976.

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(1) Laboratoire de Pathologie cellulaire, Paris.

(1) Service de Contrôle des Eaux de la Ville de Paris.

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	O.M.S.	Les risques pour la santé dus à la présence de certaines substances dans l'eau	Chronique - Genève

1974 -	BAYER A. INR	Radiological belassung in obertheingebiet durch Kerntechnische anlagen	KFK Nachvichten R.F.A. Karksruhe
	BOUQUIAUX J.	Non organic micropollutants of the environment	C.E.E. - Luxembourg
	BUTTIAUX et Col.	Manuel de techniques bactériologiques	Flammarion - Paris
	COSKSON J.	Virus and water supply	Jawwa (water technology)
	DERNOTT J.H.M.C.	Virus problèmes and their relation to water supplies	Jawwa
	BEAN E.L.	Potable water quality goals	Jawwa
	O.M.S. (MASIRONI R.)	Relations entre oligo-éléments et maladies cardio-vasculaires	Genève
	O.M.S. (BLAUD J.B.)	La mer empoisonnée	Santé du Monde
	O.M.S. (groupe d'experts)	Méthode d'études des effets biologiques des polluants	Bureau régional - Copenhague
	O.M.S.	Tendance et perspective de la mortalité	Chronique - Genève
	O.M.S.	Les maladies transmissibles et lutte antivectorielle	Chronique - Genève
	O.M.S.	Goître et crétinisme endémique Importance en santé publique et prévention	Chronique - Genève
	O.M.S.	Prévention et traitement des maladies cardio-vasculaires	Chronique - Genève (3 fascicules)
	O.M.S.	Règlement sanitaire international	Genève
	O.M.S.	Elimination domestique des excès de fluor Fluoridisation des eaux	Centre international de référence La Haye
	O.M.S. (PASSMORE et Col.)	Manuel sur les besoins nutritionnels de l'homme	Genève
	O.M.S.	Contamination des aliments par le Hg	Chronique - Genève
	PARREINS G.	Les centrales nucléaires	PUF - Paris
1975 -	Dr ABRAMON M.	La pollution par le plomb menace le cerveau de l'enfant	Le Soir - Bruxelles
	Agence Financière de bassin Adour-Garonne	Le cadmium	Bordeaux
	Dr BOUTEILLE et Col.	Etude par microscopie électronique de la pathogénicité de virus isolés des eaux sur la cellule	DGRST - Paris
	C.E.E.	Environment and quality of life	Health protection Directorate - Luxembourg
	C.E.E.	Colloque sur les maladies cardio-vasculaires (communiqué de presse)	Direction de la protection sanitaire Luxembourg
	CD - MR	Réflexions sur l'émission télévisée FR 3 du vendredi 31.10.75 (service public)	Paris
	DRAPEAU A.J.	Formation de subtances cancérigènes au cours de la chloration de l'eau potable	Technique de l'eau - Bruxelles
	ELZENGA C.H. et Col.	Asbestos in drinking water	Water research center -Delft
	E.P.A. (JAKUBOWSKI et Col.)	Viruses and drinking water quality observations and reflexions on methods and standards	
	E.P.A. (JAKUBOWSKI et Col.)	Examination of drinking water supplies for viruses	Water supply research - Cincinnati
	FESTY B. (LHVP)	Teneurs en éléments inorganiques d'eaux superficielles prélevées au droit des prises d'eau de la Région parisienne en 1975, prélevées au niveau des compteurs ou de l'habitat (Zn, cd, Fe, Cu) en 1975	Préfecture de Paris
	FESTY B. (LHVP)	Premiers résultats des recherches systématiques d'éléments métalliques ou métalloïdiques contenus dans les eaux superficielles au droit de prises d'eau de la région parisienne	Préfecture de Paris

1975 -	FIGARO - (LE) (suite)	Méfiez-vous des enfants du parasite	19 juin
	GRUNNET R.	Salmonella in sewage and receiving waters	University of Arhus - DK
	GULLIAN C. et Col.	Essais comparatifs de méthodes de concentration du poliovirus à partir des eaux	T.S.M. n° 5
	HERDA H.J.	Die gepeinigten Menschen von Minamata	Die Stern
	INFORMATION	Lindane	International Association
	MARIE A.	Salmonella Wien = polyresistance	—
	Dr MOYEN E.N.	Documentation bibliographique. Environnement et infections bactériennes	Ministère de la Qualité de la vie - Paris
	NERI L.C. et Col.	Health aspects of hard and soft waters	Jawwa (Management)
	National Environmental Enginuring research Institut	Virus removal in sewage treatment	Technical digest Nagpur - India
	O.M.S. (MASIRONI R.)	Maladies cardio-vasculaires et eaux douces	Genève
	O.M.S.	Evolution de la mortalité et de la morbidité (1969-1972)	Chronique - Genève
	O.M.S.	Classification des pesticides	Chronique - Genève
	O.M.S.	Health effects relating to direct and indirect re-use of waste water for human consumption	International Reference Center (D. Hague)
	O.M.S. (ROOK J.)	Complications se rapportant à la préchloration	Centre international de référence (La Haye)
	O.M.S. (DREWRY W.A.)	Elimination des virus par coagulation chimique	Centre international de référence (La Haye)
	O.M.S.	Surveillance des Salmonella	Chronique - Genève
	O.M.S.	Les maladies transmissibles dans les Amériques	Chronique - Genève
	O.M.S.	Normes élémentaires provisoires de l'eau potable	Centre international de référence (La Haye)
	PICARD G.	Pourquoi pas l'eau du robinet ?	50 millions de consommateurs n° 59
	Quartely report	Determination of tritium (^3H) in groundwater	National Institut for water supply the Netherland
	BIZET M.	Epuration des eaux de surface et des eaux usées	S.L.E.E.
	BIZET M. et Col.	Etude sur station pilote du devenir de divers polluants au cours d'une réalimentation de nappe aquifère par bassin	S.L.E.E.
	RODIER J. et Col.	L'analyse de l'eau	Dunod - Paris
	Technical University of Denmark	Conference on Nitrogen as a water pollutant	Copenhague
	U.S. Départment of Health. Education and Welfare	Cumulated Inter Medicus (vol. 16)	Washington DC (consultation in O.M.S. - Genève) IAWR
	VAN DER VEEN C.	International arbeitgemeinschaft der Wasserwerk im Rheineinzugsgebiet	Arbeitotagungs - Amsterdam
	VILAGINES R. (SCEVP)	Résultats des recherches systématiques d'éléments métalliques ou métalloïdiques dans les eaux superficielles du bassin parisien	Préfecture de Paris
1976 -	ANPE	A propos des adoucisseurs d'eau	L'eau pure N° 36
	ANPE	L'eau de ville remplace l'eau minérale à l'Assistance Publique	L'eau pure N° 36
	Cahier des Ingénieurs agronomes	Contrer les dangers de l'Ingénierie génétique	N° 296

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	ETERNIT	Amiante et ses problèmes de santé	Courrier
	FROST D.V.	On being right about the elements we fear	Chemical et enginuring news
	GLEDEL M.	Coliformes fécaux et Escherichia coli et contamination des produits alimentaires	AFNOR
	GRONCELLA C.	Quelques réflexions sur l'évolution nécessaire des traitements des eaux de distribution	AGHTM - Tours
	Les grands thèmes	La vie microscopique	Laffont - Paris
	Dr LABONDE (LHVP)	Bilan des hépatites virales déclarées au cours de l'année 1975 à Paris	CDHP - Paris
	LAPIERRE J.	Méningo-encéphalite amibienne primitive	Concours médical - Paris
	LECLERC M. et Col.	Sur la qualité microbiologique du cresson et des eaux de cressonnières Autoépuration microbienne par lagunage Les eaux usées des hôpitaux et leur importance épidémiologique	Water Research
	Dr LEVET G.	Eaux d'alimentation = 81,6 % des entérobactéries isolées résistent à un ou plusieurs antibiotiques	Quotidien du médecin
	LOT F.	Les prix Nobel de Sciences 1975 : virus et malignité	Le moniteur des pharmaciens et des laboratoires
	MAZOIT L.P.	L'eau et la vie	L'eau pure n° 39 et 40
	HANEVY J.V.	Les généticiens ne joueront pas aux sorciers	L'express
	MORETTE A.	Rapport concernant la conception d'une réglementation à appliquer aux produits commercialisés nouveaux dans le cadre de la protection des eaux	Note
	NORDAN C.G.	Les eaux usées, un problème sans cesse renouvelé : synergies et antagonismes	Le moniteur des pharmaciens et des laboratoires
	O.M.S. (BARNA D. et Col.)	Nouvelle arme contre le choléra et les autres maladies diarrhéïques aiguës	Santé du Monde - Genève
	O.M.S.	L'onchocercose	Chronique - Genève
	O.M.S.	Approvisionnement et excreta	Chronique - Genève
	O.M.S. (LE BLANC R.)	Gestion des eaux au Canada	Bulletin de la qualité des eaux
	O.M.S. (HUISMAN et Col.)	Critères de qualité de l'eau	Chronique - Genève
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	O.M.S. (KRISHNAMA-CHARI K.)	Fluorure dans l'eau potable	Centre international de référence - La Haye
	O.M.S. (GERLACH S.)	Recherche MEDLINE	Genève
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	O.M.S. (LUND E.)	Manual on analysis for water pollution control (virological examination)	Bureau régional - Copenhague (en cours de fonction)
	LE CADUCEE (Ecologie et Médecine)	Pesticides organo-chlorés Pesticides organo-phosphorés Pesticides divers Métaux lourds, Hg, Cd, Zn Le plomb Facteurs cancérogènes de l'environnement	Aid. Press - Bois-Colombes (92)
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Review of waterborne diseases

Summary

The author, reviewing the pathology of hydric diseases, notes that its scope has widened with the considerable expansion of analytical investigation means now available.

As far as water supplies are concerned, the review is divided up in three parts on the basis of the hazards incurred—

- diseases related to the water structure,
- diseases related to toxic substances in water,
- diseases related to bacteria, parasites, or viruses in water.

Diseases related to the water structure; long term hazards through deficiency or excess

These diseases are essentially linked with the internal balance and the physico-chemical nature of water. Four cases only are dealt with.

Endemic goitres due to chronic thyroid tumification; considerable past and current research works have practically established a relative correlation with the iodine content in the environment.

Fluorosis and dental decay for which beneficial and hazardous limits of global daily requirements in fluorine have finaly been set up on the basis of climatic conditions.

Cardio-vascular diseases related non only to the alcaline-soils content in water, but also to traces of foreign metallic substances.

Infantile methemoglobinemia linked with gradual water enrichment in nitrates which, in the absence of precautionary measures, could result in a multiplication of occurences. In this respect, the quality of water supplies is reassuring.

Diseases related to toxic substances in water ; medium term hazards through accumulation

Until a comparative recent time, the problem of intoxications by traces of metals was only due to accidental or voluntary ingestions or to occupational diseases, although some acute intoxication cases were reported in the past. Advances in analytical investigation have shown how insidious impregnation by metallic or organo-metallic compounds in water can be for consumers. A few conventional cases only are reviewed.

Lead poisoning, considering the extensive use of lead pipes, is still a problem. While slightly mineralized water only results generally in chronic intoxications, one can fear that uncontrolled softened water, in compliance with consumers wishes, could induce more marked pathological effects.

Cadmium intoxication was drawn to our attention by spectacular accidents in Japan—which are most unlikely to occur in water utilities. However, since traces of this metal cause important enzymatic disorders, authors are very suspicious of it.

Mercury intoxication, apart from occupational intoxication, proved its existence in the Minamata case. Fortunately, organometallic mercury compounds —whether due to direct discharges or to biological methylation—are only found in small quantities in the environment. But their possible participation to the alimentary chain at a given stage should make us particularly attentive to the discharge of mercury refuses.

Other metals, copper, zinc, arsenic, chromium, silver, whether they are currently used for pipe manufacturing or for water disinfecting or result from a characterized pollution, never cause lasting pathologic effects to users, except in case of accidents or voluntary actions.

All pesticides are dangerous. However, normally operated waterworks cannot be the cause of the gradual intoxications pesticides generate. But what about organo-chlorinated compounds due to water chlorination?

Diseases related to bacteria, parasites or viruses in water ; short term hazards

The cause of the big epidemics in the past or those still dormant. They essentially concern directly or undirectly man-transmitted contagious diseases.

Dysentries, either of bacillar or protozoal origin, involve a typical gastro-intestinal syndrome, developing toward healing or chronicity. A new fact should be noted—some saprophyte amoebae in our rivers can become pathogeneous, under still unknown conditions.

Typhoparatyphoids, a speticaemia, whose symptoms are still present in our memories, should not be lethal nowadays although morbidity is not nil. Improvements in isolation techniques have shown a very large diversification of the concerned serotypes—typhoids still represent an important potential hazard, especially through alimentary toxinfections.

Colibaciloses do not correspond to any specific morbid category. The interest of the germs lies in the fact that they are normally always found in the digestive tract; this is why they are used as test germs for assesing faecal contamination in water.

Cholera has not disappeared, but it has changed and only remains epidemic in areas where it initialy occured; in our areas, it has lost it epidemic character for the time being as a result of prophylactic measures and of the specific therapeuty applied. No other disease has been more instrumental than cholera to the introduction of collective health equipment.

Leptospiroses, bilhazioses, distomatoses, filarioses as well as **common intestinal pavitoses** require special conditions to be transmitted to man and to intermediary carriers. Although some of those diseases are widely spread, they hardly concern waterworks.

Virus diseases are increasingly important in water pathology, but the epidemic concept has not in this case the same implications as with microbial germs. While poliomyelitis is now checked thanks to inoculation, epidemic hepatitis is spreading without possible laboratory checking. This raises the problem of virus pathogenity; the lack of specific therapy is a deficiency which water utilities have to counteract by high quality treatments.

The author, after having noted the worldwide advances achieved in the field of drinking water health and taking into account the latter's part in the alimentary ration, concludes that in order to improve the pathological approach of the problem of water, the following action is required—

- projective epidemologic studies should be made on the possibilities for populations to adapt themselves to changes brought about by the technology concerning water quality,

- a reasonable coordination between authorizations to industry and towns for toxic discharges upsteam of water intakes and the limit quality admissible for raw water which is drawn by waterworks intakes.

- a better consideration of problems resulting from the persistance and the life time of microbial germs and the infecting dose required for the occurence of clinically detectable symptoms.

The author wishes that the compiling of health cards now being initiated within the EEC should be speeded up in order to determine in which directions new actions should be undertaken in the field of water quality.

Relationship between the Environment and Water Development Schemes

by Richard Hazen
(U.S.A.)

A. Impact of Water Development Schemes on the Environment

The most avid environmentalists conceded that a safe and potable water supply contributes much to the environment of towns, cities and other densely populated areas and to the development of commerce and industry so necessary in our economy. The vital role of water supply, recognized down through the ages in all parts of the world, has given water development schemes a priority not enjoyed by other utilities or public and private undertakings.

In proclaiming the benefits of water development schemes, however, we must recognize that inhabitants of distant rural areas furnishing the water may feel quite differently. For them, the lowering of groundwater tables, the clearing of reservoir sites, the flooding of homes, farms, and villages are deadly blows to the region's familiar environment.

Until quite recently in the United States, objections to water supply undertakings on environmental grounds have been infrequent and of limited significance. Today, however, with many of the people and responsible officials misinformed as to water resources and their development, there is an insistent demand that traditional schemes give way to the wonders of the age: recycling, wastewater reclamation, desalting of sea water, etc. The demands are persistent and vocal, too much for most politicians to resist. In well watered parts of the country, the opposition to using land exclusively for water supply reservoirs is formidable. Even in the western arid and semi-arid states, where the federal government provides irrigation at a fraction of the cost, storage projects give way to environmental complaints.

In New York State, preliminary engineering studies of a reservoir to regulate the Hudson River for a metropolitan water supply triggered legislation barring permanently the construction of reservoirs within the Adirondack forest preserve. In Washington, D.C., millions of dollars are being spent to explore the feasibility of pumping water from the Potomac estuary, or to reclaiming a water supply from the municipal sewage treatment plant effluent in order to avoid regulation of the river. The Potomac watershed spreads over 3 states with an area of 25 100 km^2, an average discharge of 318 m^3/s and minimum monthly discharge of 24,3 m^3/s. Water demands on the river, expected to increase from approximately 21,1 to 36,9 m^3/s in the next decade, could easily be met by the construction of 2 or 3 reservoirs of modest size. Corps of Engineers studies a few years ago demonstrated more than 16 feasible and economical reservoir sites. Up to the present, however, environmental protest, state rivalry and other objections have blocked construction of all but one reservoir, so far upstream as to preclude effective regulation at the Great Falls intake.

There is a great deal of environmental activity in the United States at the present time, with thousands of federal, state, and local agencies stirring the pot. The programmes of particular interest to the water supply industry are the new water quality standards issued this year by the federal Environmental Protection Agency, and a national "clean waters" programme headed by the same agency. The latter is an enormous undertaking, costing billions of dollars. It will miss by a wide margin target dates set by Congress a few years ago. Although "better water supplies at less cost" have been cited frequently as a major objective of the clean waters programme, most of the activity and money has been directed to more and better treatment of municipal and industrial wastewater, little of which ever reaches a water supply intake.

What irritates the American water industry most is EPA's inclusion of filtration plants with municipalities and industrial plants as sources of river contamination. Under its regulations, all alum or iron sludge and filter backwash (unless recycled) must be treated before its return to the source of supply. The need for this is recognized where the wastewater is discharged to a small stream or pond, or into a dry ditch. But to require such treatment at plants taking water from large rivers, lakes or estuaries with turbidities ranging up to 500 j.u., or more, is absurd and extremely wasteful. At some works the cost of wastewater treatment facilities is estimated substantially more than required for construction of the original plant. The A.W.W.A. has formally requested modification of the regulation and elimination of wastewater treatment except where necessary. Fortunately, enforcement has been slow to date, and few facilities have been built.

B. Impact of the Environment on Water Development Schemes

Turning to impacts of the environment on water development schemes, we find many questions and many complaints. Some of these are warranted and significant. Some reflecting exaggerated environmental propaganda may have minor significance. For the real, adverse changes in the environment, however, the water manager has good reason for concern. Although most of the changes are man-made, they are usually irreversible. Some involve unknown pollutants, not always amenable to current water treatment. The changes may come quickly, without adequate time to evaluate the effects. And, as we are beginning to realize, the worst effects may go undetected for years.

The impacts of environmental change cited below are based largely on experience in the United States. Similar conditions and problems are found in many parts of the world.

1 Recreational Use of Water Supply Facilities

Barring the public from water supply reservoirs and watersheds has been debated for years. Based upon the principle that even the best of raw waters was not too good, water managers had little choice. Except for boating and fishing under permit, the rules were seldom relaxed. Most communities and water companies cannot afford to own a substantial part of their watershed, and rely on inspection patrols and health department action to reduce upstream contamination. Protective efforts are necessarily concentrated at the reservoir and near intakes.

At Portland, Maine, however, where treatment is limited to chlorination, the eastern end of Sebago Lake is used for water supply and the western portion for bathing and water sports. Sebago Lake has an area of more than 130 km², and the water supply intake is around 16 km from the recreational area. At Chicago, Lake Michigan water was used without filtration over much of the city until 1965 in spite of wide spread recreation, industrial activity and water transportation. Chief reliance was placed on long intake tunnels to deep water and chlorination. A proposal some years ago to devote parts of New York City's reservoirs to recreation made little impression on health and water supply officials. In any event, the most eager proponents were discouraged by the rocky and muddy shoreline of half-full reservoirs during a long, severe drought.

The hazards of swimming and water sports are cut dramatically by the installation of coagulation and filtration. In spite of this, most health departments and water supply officials have kept in force regulations adopted years ago to protect unfiltered supplies. This probably will change because of public demands for multi-purpose undertakings, and the trend toward superior filtration of all surface waters. Water treatment plants designed and operated to meet the new strict standards should practically eliminate the hazards of recreational contamination. Where conditions are favourable for recreation, the public ultimately may be expected to insist upon access to water supply facilities. It will be a nuisance to management, but not a disaster.

2 Deforestation of Watersheds and Agriculture

Stripping forests from watersheds by logging and pulp wood operations are likely to reduce the quantity and quality of water available, and to increase its cost. Reduction of the forest and natural ground cover quickens the storm runoff, causes erosion and gullying, and reduces the dry weather flow previously sustained by underground seepage.

The effects of these changes may show up at the water plant in a number of ways:

- —For unfiltered supplies, a drop in water quality, erratic sterilization, and potential shutdown of plant following rains; periodic flushing of water mains.
- —Siltation of reservoirs, basins, and other structures; periodic removal of silt and sand.
- —With filtered supplies, greater coagulant dosages, shorter filter runs, and some loss in filtered water quality.

Recent studies by the Soil Conservation Service (U.S. Department of Agriculture) showed for a number of river basins in southeastern United States, gross annual erosion of 12,6 tonnes per hectare from crop land, 5,04 tonnes per hectare from pasture land, and 1,26 tonnes per hectare from forest land. For large construction projects involving earth moving, erosion rates reached staggering amounts, several hundred times the figures cited, but they applied to only a small part of the watershed. The studies indicated that between 30 and 40% of the eroded soils become water borne sediment in the streams. Erosion rates vary over a wide range, depending upon local conditions and other factors too numerous to consider here. The figures cited indicate the order of magnitude.

Large scale logging is commercially feasible in only a few parts of the United States, and is restricted or prohibited in national and state forests. Pulp wood is harvested in many states, but operations are relatively local, and the effects minimized by active reforestation programmes. Although logging interests, conservationists, and federal officials quarrel a good deal over the administration of protection policies, the impact of future clearing is likely to be spotty and not of major significance to water development schemes.

The erosion of agricultural land and increased turbidity of water, and the wasting of fertilizer into waterways have been a source of complaint for years. The situation has been aggravated in recent years by the addition of non-degradable organic insecticides. In the United States, the land devoted to agriculture has decreased steadily in recent decades and the situation is not expected to change substantially. Only a few western states where irrigation projects may open land to farming is there likely to be serious trouble.

In fact, in rural and suburban counties surrounding many cities, the abandonment of farms has spread rapidly. The prices of farmland for housing has skyrocketed, and made the owners glad to sell out. Between 1955 and 1975 the farm acreage in New York State declined from 6,24 to 4,62 million hectares, or 25%; in New Jersey, the "Garden State of America", from 0,69 to 0,41 million hectares, or 41%; and in Connecticut, 0,65 to 0,20 million hectares, or 69%. These changes have had a substantial impact on water supplies in the region.

Algae problems and eutrophication of lakes have called attention to the discharge of nutrients by municipalities, industries, and agriculture. The quantities of phosphates and nitrates contributed by agriculture vary widely with the circumstances. They depend upon the character and slope of cropland, the type and dose of fertilizer, the rainfall, the frequency and intensity of storms, and the surface or groundwater course to the receiving water.

In a study of Lake Erie, the yearly agricultural discharge of phosphorous was estimated at 43,6 kg per km², or $2{,}73 \times 10^6$ kg. The nitrogen input was estimated at least 10 times this amount. The Lake Erie basin is comparatively flat with a total area of 104 300 km², one-fourth of which is occupied by the lake. The principal in-basin sources of phosphorus are municipal wastes (72%), agricultural runoff (17%), urban runoff (7%), and industrial wastes (4%). While the total agricultural contribution is enormous, its relation to nutrients in municipal sewage, urban runoff, and industrial wastes proved smaller than anticipated by many.

In studies of much smaller lakes at Madison, Wisconsin, C. N. Sawyer found nutrient contributions from fertilized farm land as follows:

	Range kg/a km²
Inorganic Nitrogen	488–715
Organic Nitrogen	181–206
Inorganic Phosphorus	6,1–10,8
Organic Phosphorus	32,4–34,9

Eutrophication is nothing new and sure to continue. Treatment of reservoirs with copper sulphate, pretreatment with roughing filters or microstrainers for algae removal, and periodic taste and odour problems are likely to be with us for years to come.

Although limited to arid states, perhaps the most critical consequence of agriculture on water supply developments has been the high dissolved solids in irrigation drainage to streams used for water supply purposes. The Bureau of Reclamation has estimated that Upper Colorado River Basin demands for irrigation water will increase from 4 660 to 7 275 million cubic metres per annum by the year 2000, and that without remedial measures, the TDS of river water at the Mexican border will increase from 850 to 1250 mg/l. Heavy drafts on the Colorado River for irrigation and water supply and their effect on water quality have been sore subjects between the United States and Mexico for years. At least partial correction is being sought by construction of a 5,27 m^3/s reverse osmosis desalting plant and an 86 km long reject water canal to the Gulf of California. Improvements in irrigation technology can reduce but not eliminate the build up of solids in drainage water. Potential solutions are costly.

3 Watershed Land Development

(a) *Residential*

The effects of residential developments on watersheds and water supply are demonstrated by many cities that have abandoned local sources in favour of new, more distant supplies. In most instances the moves have involved inadequate supply and deterioration in water quality.

Sparsely populated settlements with properly constructed household sewage disposal systems have little effect. Where settlements are such as to require public water supply and sewerage systems, the danger may be greater because of possible malfunction or non-function of small and infrequently attended wastewater treatment plants. In any event, nutrients are likely to reach the water course and reservoir. Much the best solution, of course, is a well constructed and tight sewer by-passing the reservoir to a downstream discharge.

New York's first major water supply system was completed in 1842 on the Croton River, 48 km north of the City. The 982 km^2 watershed, farmland with a few widely separated hamlets when the first reservoir and aqueduct were built, is now the fastest growing section of Westchester County. The watershed population totals more than 100 000. New York City provides wastewater treatment for villages on the watershed, but 80% of the population is non-urban, depending chiefly upon household septic tanks.

In the past 30 years the nitrates doubled from 0,13 to 0,25 mg/l, and reached 0,42 mg/l in one year; the chlorides have increased from 3,0 to 21,0 mg/l (some of this due to the use of salt for melting ice on the growing network of highways). Total solids have gone from 80 to 140 mg/l. By close control and frequent application of $CuSO_4$ in the several reservoirs, algae have been held in check, but only at the cost of periodic copper concentrations near allowable limits for drinking water.

Residential developments in urban and suburban regions differ from those in rural areas in their effect on water supply. Assuming reasonably tight sanitary sewerage away from the water course, the nutrient input may not be serious. Combined sewer overflows and non-point sources in urban areas, however, are frequently significant. The fast runoff following storms from house tops, paved streets, parking lots, etc. is likely to carry sand, silt, vegetation, litter and miscellaneous waste substances through gutters and storm water drains into the water supply. While such substances are for the most part easily removed by filtration, they are intolerable in an unfiltered supply.

(b) *Commercial and Industrial*

The harmful effect of watershed development on water supply is not only domestic sewage contamination, but more important, the potential discharge of toxic and pathogenic wastes from commercial and industrial operations. Such wastes discharged to a municipal sewerage system are frequently diluted and not noticed for a long time.

The impact of watershed development on water supplies has had a marked effect on a number of systems in the New York Metropolitan area. Pilot plant tests for selection of the most economical treatment of Croton water are near completion. Under prodding by the New Jersey Health Department, each of the 3 largest purveyors in the northern part of the state have plans underway for 5,27 m^3/s filter plants. Similar action will follow in Connecticut.

The changes in environment noted above are important in regard to surface water supplies, watersheds, and impounding reservoirs. They are less important for groundwater supplies because of protection against contamination afforded by filtration in the underground aquifer. In spite of this, wells are occasionally destroyed by toxic wastes leached from sanitary landfills, or by the disposal of industrial wastes through injection into the ground. Along the coast, salt water intrusion caused by close well spacing and over-pumping has ruined for all time many groundwater supplies.

4 Sewage and Waste Disposal in Waterways

As long as adequate water supplies could be obtained economically by tapping clean headwater streams, the discharge of sewage and industrial wastes to the rivers and waterways of the United States had little effect on water development schemes. This situation still prevails in many parts of the country. In the central portion, where large rivers and the Great Lakes provide both water supply and waste disposal, there has always been some conflict. Even there, however, early water treatment was directed toward the removal of natural turbidity rather than of harmful substances in sewage and industrial wastes.

Under Public Law 92-500, the "clean water act" adopted by the Congress in 1972, the treatment of municipal sewage and industrial wastes is scheduled to be substantially improved over the next 10 or 15 years. In some instances this will facilitate water treatment for public supply. In many others, the effects will be nominal unless parallel measures are taken to exclude particularly dangerous substances.

It has seemed to the writer that the EPA programme has overemphasized the removal of BOD and suspended solids. Even in places where there is no dissolved oxygen problem, and effluent turbidity is only a fraction of that in the receiving water, strict effluent criteria are stipulated. We worry about phosphate and nitrate reduction at plants discharging into waters with none of the characteristics requisite for algae growth.

On the other hand, although there is much said and written about the subject, we have moved slowly toward effective control of heavy metals, toxic substances, and suspected carcinogenic organics found in some industrial wastes and municipal sewages. For some of these wastes, complete removal and disposal at the plant of origin will be necessary. The quantities of dangerous waste usually are not large, making feasible and not too expensive disposal before dilution. In some cases, use of offensive substances must be prohibited by law, as with DDT in the United States a few years ago.

In any event, the responsibility and cost of keeping these potentially dangerous substances from waterways must be borne by industry. Even if water treatment methods are developed to cope with them, the cost should not be shifted to the purveyor and water customer.

The following quotation taken from a 1973 memorandum by the International Consortium on Water

Supply Undertakings in the Rhine Catchment Area (IAWR) states the issue clearly:

> For the evaluation of effluent releases from the standpoint of drinking-water supplies, the biochemical oxygen demand (BOD_5) is not the primary consideration. Numerous other pollution parameters are much more important. All unwholesome and potentially toxic substances have absolute priority in the evaluation. The release of dangerous substances into our rivers and streams must be totally prohibited.

Conclusion

The impact of environmental changes described varies from place to place, depending upon circumstances. Individuals will differ sharply as to the relative significance and importance of these changes and to the priorities that should be assigned. The listing of the several items reflects in a general way the writer's evaluation of the gravity or severity of the changes to water supply development. Thus, recreational use of water supply facilities may be permitted with a high degree of safety if suitable facilities are provided and precautions taken. Detection and exclusion of little understood but potentially dangerous chemicals, on the other hand, may prove exceedingly difficult. Although the true significance of some wastes may take years to discover, the potential hazard cannot be ignored. Environmental change is inevitable. The worst effects can be alleviated if we concentrate on the essentials.

Relations entre l'environnement et les projets de mise en valeur de l'eau

par Richard Hazen
(U.S.A.)

A. Impact sur l'environnement des projets de mise en valeur de l'eau

Les plus exigeants des environnementalistes concèdent qu'une alimentation en eau saine et potable contribue beaucoup à l'environnement des villes et autres régions très peuplées, et aux progrès du commerce et de l'industrie si nécessaires à notre économie. Le rôle vital de l'alimentation en eau, reconnu à travers les âges dans le monde entier, a fait donner aux projets de mise en valeur de l'eau une priorité dont ne bénéficient pas les autres services publics et les entreprises publiques ou privées.

En proclamant les bienfaits de ces projets, cependant, nous devons reconnaître que les habitants des régions rurales éloignées qui fournissent l'eau peuvent penser très différemment. Pour eux, l'abaissement du niveau des nappes souterraines, l'évacuation des sites de réservoirs, la submersion d'habitations, de fermes et de villages sont des atteintes mortelles à l'environnement familier de la région.

Jusqu'à une date récente, aux Etats-Unis, les oppositions aux distributions d'eau pour des motifs environnementaux ont été peu fréquentes et d'importance limitée. Mais aujourd'hui, où beaucoup de gens et d'officiels responsables sont mal informés sur les ressources en eau et leurs possibilités de mise en valeur, il y a une demande insistante pour que les usages traditionnels cèdent la place aux merveilles du siècle: le recyclage, la récupération des eaux usées, le dessalement de l'eau de mer, etc. . . .

Les demandes sont persistantes et reitérées, trop pour que la plupart des politiciens leur résistent. Dans les parties bien arrosées du pays, l'opposition à l'utilisation de terrains exclusivement pour des réservoirs d'alimentation en eau est formidable. Même dans les états de l'ouest arides ou semi-arides, où le gouvernement fédéral fournit l'irrigation au-dessous du prix de revient, les projets de réservoirs donnent lieu à des plaintes pour l'environnement.

Dans l'Etat de New-York, les premières études pour un réservoir destiné à alimenter en eau la métropole en régularisant l'Hudson ont provoqué une législation qui a interdit en permanence la construction de barrages dans la réserve forestière de l'Adirondack. A Washington, D.C., des millions de dollars sont dépensés pour explorer la possibilité de pomper l'eau de l'estuaire du Potomac, ou de traiter l'eau de la station de traitement des eaux usées municipale afin d'éviter la régularisation du fleuve. Le bassin du Potomac couvre 2 500 km^2 sur trois états et son débit mensuel moyen est de 22 700 000 m^3/j et le débit mensuel minimal de 1 740 000 m^3/j. Les besoins en eau sur le fleuve, que l'on estime devoir passer de 1 500 000 à 2 600 000 m^3/j dans la prochaine décennie, pourraient facilement être couverts par la construction de deux ou trois réservoirs de taille modeste. Les études du Corps du Génie il y a quelques années ont montré qu'il y a plus de seize sites économiquement utilisables. Mais jusqu'à présent les protestations de l'environnement, les rivalités entre Etats et autres objections ont bloqué la construction de tous les réservoirs, sauf un qui est trop en amont pour régulariser efficacement le débit à la prise de Great Falls.

Il y a actuellement une grande activité en faveur de l'environnement aux Etats-Unis, où des milliers d'organismes fédéraux, d'Etat ou locaux s'agitent en ce sens. Les programmes d'un intérêt particulier pour l'industrie de l'eau sont les nouvelles normes de qualité de l'eau publiées cette année par l'Agence fédérale pour la protection de l'environnement (E.P.A.) et un programme national "d'eaux propres" dirigé par le même organisme. L'E.P.A. est une entreprise énorme qui coûte des milliards de dollars. Elle est largement en retard sur les objectifs fixés par le Congrès il y a quelques années. Bien que "de meilleurs services d'eau à un moindre coût" aient été souvent cités comme objectif majeur du programme des eaux propres, la plupart des activités et des fonds ont été employés à traiter plus et mieux les eaux usées urbaines et industrielles, dont une faible partie atteint jamais une prise d'eau potable.

Ce qui irrite le plus l'industrie américaine de l'eau est le fait que l'E.P.A. a inclus les stations de filtration urbaines et industrielles parmi les sources de contamination des cours d'eau. D'après ses règlements, toute eau de lavage des boues de sulfate d'alumine ou de fer et des filtres doit (à moins d'être recyclée) être traitée avant rejet dans le milieu naturel. La nécessité de ce traitement est évident quand les eaux usées sont rejetées dans un petit cours d'eau, un étang ou un fossé. Mais exiger un tel traitement pour des stations prenant l'eau de fleuves importants, lacs ou estuaires dont la turbidité atteint 500 U. Jackson ou plus, est absurde et extrêmement coûteux. A certaines stations, le coût des ouvrages de traitement des eaux usées est estimé être beaucoup plus élevé que celui représenté par la construction de la station originale. L'AWWA a formellement demandé la modification de ce règlement et la suppression du traitement des eaux usées là où il n'est pas nécessaire. Heureusement la mise en oeuvre a été lente jusqu'à présent et peu d'ouvrages ont été réalisés.

B. Impact de l'environnement sur les projets de mise en valeur de l'eau

Si l'on en vient aux impacts de l'environnement sur les projets de mise en valeur de l'eau, nous trouvons beaucoup de questions et de plaintes. Certaines sont pertinentes. Certaines, reflétant une propagande environnementaliste exagérée peuvent avoir une signification mineure. Mais en ce qui concerne les changements réels nocifs dans l'environnement, le responsable de la gestion des eaux a de bonnes raisons de s'inquiéter: bien que la plupart des changements soient provoqués par l'action de l'homme, ils sont généralement irréversibles. Certains impliquent des polluants inconnus, qui ne relèvent pas toujours des traitements de l'eau courants. Les changements peuvent être rapides, ne laissant pas le temps d'en évaluer les effets. Et, comme nous commençons à la comprendre, les effets les plus dangereux peuvent ne pas être détectés pendant des années.

Les impacts des changements de l'environnement cités ci-dessous sont largement basés sur l'expérience américaine. Des situations et problèmes similaires se retrouvent en de nombreuses régions du monde.

1 Emploi pour les loisirs des ouvrages d'alimentation en eau

Interdire au public les réservoirs et bassins versants est discuté depuis de nombreuses années. En vertu du principe que même la meilleure des eaux n'est pas trop bonne, les gestionnaires de l'eau n'avaient pas beaucoup le choix. Sauf pour le canotage et la pêche avec permis, les règles étaient rarement élargies. La plupert des collectivités et des distributeurs d'eau ne peuvent pas se permettre d'acheter une part importante de leurs bassins versants, et comptent sur des inspections et sur l'action du service de santé pour réduire la contamination amont. Les efforts de protection sont nécessairement concentrés sur le réservoir et au voisinage des prises d'eau.

A Portland, Maine, cependant, où le traitement se limite à une chloration, la partie est du Lac Sebago est utilisée pour l'alimentation en eau et la partie ouest pour la baignade et les sports aquatiques. Le Lac Sebago couvre plus de 130 km² et la prise d'eau est à ± 16 km de la zone de loisirs. A Chicago, l'eau du Lac Michigan a été utilisée sans filtration pour la plus grande partie de la ville jusqu'à 1965 malgré l'extension des activités de loisir, de l'industrie et du transport. On faisait confiance essentiellement à la longueur des tunnels de captage en eau profonde et à la chloration. Une proposition faite il y a quelques années d'utiliser une partie des réservoirs de la ville de New-York pour les loisirs a laissé de marbre les responsables de la santé et du service d'eau. De toute manière, les demandeurs les plus acharnés ont été découragés par l'aspect rocheux et boueux des rives des réservoirs à moitié vides pendant une longue sécheresse.

Les dangers de la baignade et des sports nautiques sont radicalement stoppés par l'installation d'une coagulation-filtration. Malgré cela, la plupart des services de santé et des services d'eau ont maintenu en vigueur les réglements anciennement adoptés pour protéger les eaux non filtrées. Cela changera probablement en raison de la demande du public pour des installations à fins multiples, et de la tendance à une filtration poussée de toutes les eaux de surface. Les stations de traitement conçues et exploitées pour couvrir les nouvelles normes strictes doivent pratiquement éliminer les dangers de contamination dus aux activités des loisir. Là où les conditions sont favorables aux loisirs, on peut s'attendre à ce que le public insiste pour avoir accès aux ouvrages d'alimentation en eau. Ce sera une gêne pour le service, mais pas un désastre.

2 Déboisement des bassins versants et agriculture

La coupe à blanc des forêts pour fournir du bois ou de la pâte à papier peut réduire le volume et la qualité de l'eau disponible et augmenter son coût. La réduction de la couverture forestière et naturelle du sol accélère l'écoulement des orages, provoque l'érosion et le ravinement et réduit le débit de temps sec antérieurement soutenu par l'eau infiltrée.

Les effets de cas changements peuvent se manifester à la prise d'eau de différentes façons:

—si l'eau n'est pas filtrée, diminution de sa qualité, stérilisation incertaine et possibilité d'avoir à fermer la prise après la pluie; chasses périodiques des conduites d'eau.

—Envasement des réservoirs, bassins et autres ouvrages; enlèvement périodique de la vase et du sable.

—Si l'eau est filtrée, doses plus fortes de coagulant, vie plus courte des filtres et une certaine baisse de qualité de l'eau.

Des études récemment faites par le Service de Conservation du sol (Ministère fédéral de l'Agriculture) ont montré, dans un certain nombre de bassins fluviaux du sud-est des Etats-Unis, une érosion brute annuelle de 12,5 t/ha pour les terres cultivées, 5 t/ha pour les prairies et 1,25 pour les forêts. Les grands chantiers de construction qui impliquent des mouvements de terre font monter vertigineusement l'érosion, plusieurs centaines de fois les chiffres cités, mais ils n'intéressent qu'une faible part du bassin versant. Les études montrent que 30 à 40% des sols érodés deviennent des sédiments en suspension dans les cours d'eau. Les taux d'érosion varient largement, suivant les conditions locales et autres facteurs trop nombreux pour être examinés ici. Les chiffres cités donnent l'ordre de grandeur.

L'abattage de forêts à grande échelle pour produire du bois n'est commercialement réalisable que dans de rares régions des Etats-Unis; il est limité ou interdit dans les forêts nationales. On produit de la pâte à papier dans de nombreux états, mais ces opérations sont relativement locales et les effets sont minimisés par d'actifs programmes de reboisement. Bien que les producteurs de bois, les conservationnistes et les agents fédéraux discutent abondamment sur l'administration des politiques de protection, l'impact des coupes à blanc futures sera sans doute local et sans influence majeure sur les projets de mise en valeur de l'eau.

L'érosion des sols cultivés et l'augmentation de la turbidité de l'eau, ainsi que l'excès d'engrais que l'on retrouve dans les cours d'eau amènent des plaintes depuis longtemps. La situation s'est aggravée ces dernières années par l'addition d'insecticides organiques non dégradables. Aux Etats-Unis, les terres consacrées à l'agriculture ont progressivement diminué au cours des dernières décades, et il est peu probable que la situation change beaucoup. Ce n'est que dans quelques états de l'ouest où des projets d'irrigation peuvent ouvrir des terres à la culture qu'il y a risque de troubles sérieux.

En fait, dans les Comtés ruraux et suburbains entourant de nombreuses villes, l'abandon des fermes s'est propagé rapidement. Le prix des terrains pour la construction a grimpé vertigineusement, ce qui fait que les fermiers étaient heureux de vendre. Entre 1955 et 1975, les surfaces cultivées de l'Etat de New-York sont passées de 6,16 à 4,56 millions d'hectares, en diminution de 25%; dans le New Jersey, "Le Jardin de l'Amérique", de 0,68 à 0,4 millions d'ha, soit—41%; et dans le Connecticut, de 0,6 à 0,2 millions d'ha, soit—69%. Ces changements ont un impact substantiel sur les services d'eau de cette région.

Les problèmes d'algues et d'eutrophisation des lacs ont attiré l'attention sur les déversements de nutriments par les villes, les industries et l'agriculture. Les quantités de phosphates et nitrates provenant de l'agriculture varient beaucoup suivant les circonstances. Elles dépendent du caractère et de la pente des terres cultivées, du type et de la dose d'engrais, des pluies, de la fréquence et de l'intensité des orages, et du trajet de surface ou souterrain vers les eaux réceptrices.

Dans une étude du Lac Erié, les apports annuels de l'agriculture en phosphore ont été estimés à 43,5 kg/km² ou 27 000 tonnes. L'apport d'azote est estimé au moins à 10 fois ce chiffre. Le bassin versant du Lac Erié est relativement plat; sa surface est de 104 000 km², dont un quart est occupé par le lac. Les principales sources de phosphore du bassin versant sont les eaux d'égout (72%), le ruissellement agricole (17%), le ruissellement urbain (7%) et les rejets industriels (4%). La contribution agricole totale est énorme, mais par rapport aux nutriments apportés par les eaux d'égout, le ruissellement urbain et l'industrie, sa part est plus petite qu'on imaginait.

Dans une étude de lacs bien plus petits à Madison, Wisconsin, C. N. Sawyer a trouvé que les apports de nutriments venant des terrains cultivés étaient les suivants en kg/an.km^2:

Azote minéral	486 à 712
Azote organique	180 à 205
Phosphore minéral	6 à 24
Phosphore organique	32,5 à 77

L'eutrophisation n'a rien de nouveau et se poursuivra sûrement. Le traitement des réservoirs au sulfate de cuivre, le prétraitement par filtres dégrossisseurs ou microtamis pour enlever les algues et les problèmes périodiques de goûts et odeurs se retrouveront dans les années à venir.

Bien que limitée aux Etats arides, la conséquence la plus critique de l'agriculture sur la mise en valeur de l'eau pour l'alimentation est peut-être la forte teneur en sels dissous des eaux de colature renvoyées aux cours d'eau utilisés pour l'alimentation en eau. Le Bureau of Reclamation a estimé que la demande en eau d'irrigation dans le bassin supérieur du Colorado passera de 4,6 à 7,3 milliards de m^3/an en 2000 et que si l'on n'y remédie pas, la masse de sels dissous dans l'eau du fleuve à la frontière du Mexique passera de 850 à 1 250 mg/l. Les lourdes ponctions sur le Colorado pour l'irrigation et l'alimentation en eau et leur effet sur la qualité sont depuis des années une pomme de discorde entre les Etats-Unis et le Mexique. On cherche un correctif au moins partiel par la construction d'une usine de dessalement par osmose inverse de 380 000 m^3/j et un canal de fuite de 85 km vers le Golfe de Californie. Les améliorations dans la technologie de l'irrigation peuvent réduire, mais non éliminer l'augmentation des sels dissous dans l'eau. Les solutions potentielles sont coûteuses.

3 Occupation des bassins versants

(a) *Résidentielle*

L'effet du développement des résidences sur les bassins versants et l'alimentation en eau est démontré par les nombreuses villes qui ont abandonné des captages locaux en faveur de captages nouveaux, plus éloignés. Dans la plupart des cas, ces changements impliquaient une insuffisance de l'alimentation en eau et une détérioration de la qualité de l'eau.

Les lotissement très dispersés avec des systèmes d'élimination des eaux usées domestiques bien construits ont peu d'effet. Lorsque ces lotissements exigent des réseaux de distribution d'eau et d'assainissement, le danger peut être plus grand en raison du mauvais fonctionnement possible ou du non fonctionnement de stations d'épuration d'eaux usées petites et mal surveillées. En tous cas, des nutriments sont susceptibles d'atteindre le cours d'eau et le réservoir. De loin, la meilleure solution est un égout bien construit et imperméable qui by-passe le réservoir jusqu'à un cours d'eau en aval.

Le premier grand captage de New-York fut réalisé en 1842 sur la Croton River, à 48 km au nord de la ville. Le bassin versant de 976 km^2 qui était couvert de cultures et de quelques villages très espacés quand le premier réservoir et aqueduc a été construit est maintenant la région du Comté de Westchester qui se développe le plus vite. La population du bassin versant compte plus de 100 000 âmes. Le ville de New-York assure le traitement des eaux d'égouts des villages du bassin versant, mais 80% de la population est non-urbaine et dépend essentiellement de fosses septiques domestiques.

Depuis 30 ans, les nitrates ont doublé, de 0,13 à 0,25 mg/l et ont atteint une année 0,42 mg/l; les chlorures sont passés de 3,0 à 21,0 mg/l (une partie provient du le sel employé pour faire fondre la neige sur le réseau croissant de grandes routes). Les sels dissous sont passés de 80 à 140 mg/l. Grâce à une étroite surveillance et à de fréquentes applications de sulfate de cuivre dans les divers réservoirs, les algues ont été contenues, mais au prix de concentrations périodiques en cuivre proches des limites admises pour l'eau potable.

Les développements résidentiels dans les régions urbaines et suburbaines diffèrent de ceux dans les régions rurales quant à leurs effets sur les distributions d'eau. Si l'on admet un réseau d'égout raisonnablement étanche éloigné du cours d'eau, l'apport de nutriments ne peut pas être très important. Mais les déversoirs d'orage des égouts unitaires et les sources non ponctuelles dans les régions urbaines sont fréquemment significatives. Les écoulements rapides provenant des toits, des rues revêtues, des parkings, etc. . . . après un orage sont susceptibles d'amener au cours d'eau, par l'intermédiaire des caniveaux et des égouts pluviaux, sable, boue, végétation, déchets et autres ordures. Ces substances sont pour la plus grande partie facilement éliminées par la filtration, mais elles sont intolérables si l'eau n'est pas filtrée.

(b) *Commerciale et Industrielle*

Les effets dangereux de l'occupation des bassins versants sur l'alimentation en eau ne viennent pas seulement de la contamination par les eaux d'égouts domestiques, mais, plus encore, du rejet potentiel des déchets toxiques et pathogènes des opérations commerciales et industrielles. Ces déchets quand ils sont rejetés dans un réseau d'égout urbain sont fréquemment dilués et passent longtemps inaperçus.

L'impact de l'occupation des bassins versants sur les distributions d'eau a eu un effet marqué sur un certain nombre de réseaux dans le Grand New-York. Des essais en pilote pour le choix du traitement le plus économique de l'eau du Croton sont en cours d'achèvement. Sous l'impulsion du service de santé du New-Jersey, chacun des trois grands fournisseurs de la partie nord de l'Etat ont à l'étude des plans pour filtrer 378 500 m^3/j. Des opérations semblables suivront dans le Connecticut.

Les modifications dans l'environnement notées ci-dessus sont importantes pour des alimentations en eau de surface, les bassins versants et les barrages réservoirs. Elles ont moins d'importance pour les eaux souterraines en raison de la protection assurée par la filtration dans le sol. Malgré quoi il arrive parfois que des puits sont mis hors service par les résidus toxiques rejetés sur les dépôts contrôlés, ou par le rejet de résidus industriels par injection dans le sol. Le long des côtes, l'intrusion d'eau salée provoquée par le surpompage de puits placés trop près ont de tout temps ruiné de nombreux captages d'eau souterraine.

4 Rejet des eaux usées dans les cours d'eau

Aussi longtemps que l'on a pu capter économiquement les eaux propres des cours supérieurs des fleuves, le rejet dans les cours d'eau des Etats-Unis ont eu peu d'effet sur le projet de mise en valeur de l'eau. Cette situation se retrouve encore en de nombreuses parties du pays. Dans le Centre, où les fleuves et les grands lacs fournissent à la fois l'eau potable et un exutoire pour les égouts, il y a toujours eu des conflits. Mais même là, le premier traitement de l'eau visait à éliminer la turbidité naturelle plutôt que les substances nuisibles qui se trouvent dans les eaux usées urbaines et industrielles.

La loi publique 92—500, dite "loi de l'eau propre", votée par le Congrès en 1972 prévoit que le traitement des eaux usées urbaines et industrielles sera sensiblement amélioré au cours des 10 à 15 prochaines années. En certains cas cela facilitera le traitement de l'eau pour les distributions publiques. En beaucoup d'autres, les effets seront symboliques si l'on ne prend pas des mesures parallèles pour exclure les substances particulièrement dangereuses. Il semble au rapporteur que l'EPA a trop mis l'accent sur l'élimination de la DBO et des solides en suspension. Même dans les endroits où il n'y a pas de problèmes d'oxygène dissous, et où la turbidité de l'effluent n'est qu'une fraction de celle du milieu récepteur, de stricts critères d'effluent sont stipulés. Nous nous faisons du souci pour la réduction du phosphate et du nitrate à des stations qui rejettent dans des eaux qui n'ont aucun des caractères nécessaires pour la poussée des algues.

D'un autre côté, bien que beaucoup ait été dit et écrit sur le sujet, nous avons fait peu de progrès vers la lutte efficace contre les métaux lourds, les substances toxiques et les matières organiques suspectées d'être carcinogènes que l'on trouve dans certaines eaux usées industrielles et urbaines. Il sera nécessaire d'éliminer certains de ces résidus à l'usine d'origine. La quantité de résidus dangereux n'est généralement pas grande, ce qui rend possible et pas trop coûteux de les éliminer avant dilution. En certains cas, l'usage de substances nuisibles doit être prohibé par la loi, comme nous l'avons fait pour le DDT aux Etats-Unis il y a quelques années.

En tout cas, la responsabilité et le coût de maintenir ces substances dangereuses hors des cours d'eau doivent être supportés par l'industrie. Même si l'on met au point des méthodes pour les traiter, ce coût ne doit pas être transféré au distributeur d'eau et au consommateur.

La situation suivante, extraite d'un mémorandum publié en 1973 par l'Association internationale des services de distributions d'eau du bassin du Rhin (I.A.W.R.) énonce clairement ce point:

"Pour juger les effluents déversés du point de vue des distributions d'eau, la demande biologique d'oxygêne (DBO_5) n'est pas la considération primaire. De nombreux autres paramètres de pollution sont beaucoup plus importants. Toutes les substances dangereuses et potentiellement toxiques doivent avoir une priorité absolue dans cette évaluation. Le déversement de ces substances dans nos cours d'eau doit être totalement interdit."

Conclusion

L'impact des modifications de l'environnement varie de place en place suivant les circonstances. Les opinions diffèrent beaucoup en ce qui concerne la signification relative et l'importance de ces changements et des priorités qui doivent être dressées. La liste des divers points cités reflète d'une façon générale l'estimation par le rapporteur de la gravité ou de la sévérité de ces changements pour les progrès de l'alimentation en eau. Ainsi, l'utilisation des réservoirs pour les loisirs peut être autorisée sans grand danger si les installations convenables existent et si certaines précautions sont prises. D'un autre côté, la détection et l'exclusion de certains produits chimiques mal connus mais potentiellement dangereux peut se montrer extrêmement difficile. Bien que l'importance réelle de certains déchets puisse prendre des années à se manifester, le danger potentiel ne peut pas être ignoré. Les changements dans l'environnement sont inévitables. Leurs pires effets peuvent être évités si nous nous concentrons sur l'essentiel.

Recent advances in water disinfection

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1 Alternative methods of water disinfection

1.1 Classification

The great diversity of techniques and means of water disinfection calls for a revision of the existing classification which divides methods into chemical and physical ones. A new classification of disinfection methods has been proposed by the Leningrad Civil-Engineering Institute Chair of Water Supply, based on an analysis of more than 500 published communications. The classification comprises:

(a) non-electrical chemical methods, based on the use of bactericidal or neutralising agents: chlorine, iodine, or bromine application, coagulation and flocculation;

(b) electrochemical methods, in which electrical energy is used to form the bactericidal or neutralising agent: ozonisation, treatment with silver ions, electrolysis, electroflotation;

(c) physical methods, using the effect of mechanical energy: ultrafiltration, heat treatment, ultraviolet or gamma radiation, ultrasonic or magnetic fields.

(d) methods of electrical treatment, in which is included, in addition to the effects mentioned above, the power effect on bacterial bodies polarised or with a hard dipole: electrophoresis, electrocoagulation, electric discharge, VHF fields.

The proposed classification starts from a colloid-chemical standpoint, the contaminated water being considered as a biodispersed system relatively monodispersed and stable. This approach permits the classification of methods on the basis of the disinfecting agents introduced, the change of the system into an unstable state and the isolation of the aggregated biophase from the medium. It must be said that the creation of a universal classification for water-supply specialists is scarcely possible—each of them would wish a classification based on his own preference. The applicability of each method depends on many factors and local conditions, such as quality of the raw water, treatment processes, availability of reagents and facilities, capacity of the station, presence or absence of a plot of land, length of the distribution system, tastes of the consumers, traditions and so on. Often the simplicity of a method, considered an advantage for a small installation proves to be a drawback for a large capacity station. In the selection of a method of disinfection in preference to all others, one should be guided by considerations of technique and economy, taking into account the shortcomings inherent in each of them.

1.2 Methods in practical use

Chemical methods of disinfection are most popular in water supply, and chlorination is at the top of the list. Iodine and bromine are used as a rule mainly for water disinfection in swimming pools. Their widespread use for the disinfection of potable water is hindered by the biological activity of these reagents which present a danger for public health, and by their high cost. Coagulation and flocculation are partially used for intensifying the sedimentation of bacteria and viruses on floc particles.

Among electrochemical methods two are now used in the water industry: ozonisation and electrolysis of chlorine compounds. Their high bactericidal effect, the easy availability of electric energy and the technical progress achieved in the design of equipment assure the increasing general acceptance of these methods.

The use of physical methods for some time to come will be restricted to UV-radiation, which exercises a nearly instant effect on the genes of the cells and does not require the introduction of foreign matter into the water. B. Nietsch (BRD 1973) notes that the molecule of desoxyribonucleic acid with a size of 265 nm contained in the gene, has a maximum spectral absorption for radiation with a wave length of 265 nm. The other methods are still confined to laboratory research.

In the USSR UV-radiation is used for underground water disinfection, provided the *E. coli* index is below 1000 per litre. Treatment units developed by the Academy of Municipal Economy have various capacities for both pressure and gravity installations.

All the methods mentioned in the classification may be of interest to specialists and will, doubtless, be developed for special purposes. But it may be said with certainty that chlorination will remain for a long time the basic method for disinfection of drinking water. Although there are still many unresolved questions which will be discussed below, the method has many advantages: high efficiency, simplicity of laboratory and industrial control, automatic feeding and control equipment, low cost of reagent, long-term disinfection effect. There is scarcely one other method which possesses all of these advantages.

In the USSR chlorination is considered the most reliable and practical method of disinfection for municipal systems of water supply, although in recent years chlorination of drinking water has been supplemented by ozonisation.

2 A review of the present status of chlorination

2.1 The theoretical bases of chlorination

The chlorination of drinking water was introduced some 80 years ago and the chlorination of sewage more than a century ago. Nevertheless, we have as yet no exhaustive concept of the mechanism of the bactericidal effect of chlorine and the chemistry of chlorination. Scientific research in this direction is continuing. In the General Report presented at the 5th Congress of the IWSA, a summary was given of results obtained in the development of theoretical concepts and their practical application. Considerable progress had been achieved in the improvement of control methods. Most attention in the report was given to the use of chlorine for water disinfection, because of the universal acceptance of this method as almost the only one used for water disinfection. Its economic, technical and sanitary advantages are thoroughly known.

The method, however, has its inherent substantial shortcomings; the two most important of these are impairment of water palatability, and the possibility of only indirect control of the reliability of disinfection by the amount of residual chlorine. Some researchers are of the opinion that the fixing of a residual chlorine quantity may be misleading, because the quantity of chlorine in the water is not equivalent to its quality. Specialists are still discussing the question of the precise meaning of the terms "free" and "combined" residual chlorine. No control of the efficacy of chlorination by the amount of residual chlorine will be satisfactory unless the form of residual chlorine is specified. But, as already said, the term "residual chlorine" is vague and inaccurate. It must be emphasised that the effect of disinfection depends on the composition of residual chlorine rather than on the process of chlorination *per se*. Therefore, in current chlorination practice it is required to specify whether combined or free residual chlorine is used as the indicator of the reliability of disinfection. The USSR standards require 0,3–0,5 mg/l free or 0,8–1,2 combined residual chlorine.

2.2 Advances in chlorination

There is no unanimous opinion on the progress achieved in chlorination. According to the report [1] of the AWWA Committee on water disinfection (1975), in the last two decades no important improvements have been introduced into the methods of chlorination of drinking water. The discovery of the break point (1939), the development of an amperometric titrator (1942), and of a recording device for the determination of residual chlorine (1952), the investigations of Feyer, Morris, Chang at Harvard (1944), and the explanation of the reaction of break point by Granstrom (1954) are all the result of profound scientific research carried out in the 'forties and early 'fifties. In the following years research was concerned mainly with the disinfection of waste water and recirculated water. In this context some problems arose, associated with the disinfection of drinking water, which called for new investigations. It was found that chlorination beyond break point of waste waters with a low content of ammoniacal nitrogen leads to the formation of an extremely resistant dichloramine in the presence of free chlorine in the total residual chlorine. This fraction is assumed to be a compound of organic nitrogen with monochloroamine and free chlorine, which titrates like dichloramine and has almost no bactericidal power. This means that new and better analytical methods have to be developed to differentiate between monochloroamine and other chlorine compounds contained in residual chlorine. Meanwhile the method most commonly used in the USA for measuring residual chlorine is the orthotolidine method which does not permit differentiation between free chlorine and combined chlorine. The discovery of objectionable volatile organic chlorine compounds raised the problem of preventing their formation. For this purpose gas-chromatography and mass-spectrography may be used.

In Great Britain chlorination has been the object of thorough investigations at the National Water Council's Water Research Centre (formerly Water Research Association), and has been accepted as a standard method of water disinfection. The investigations, the results of which were first published more than 30 years ago, have been repeated in recognition of the new practical problems of odours and after-tastes, reaction of chlorine with other disinfectants, virus removal and disinfection of distribution systems. Nevertheless the practice of chlorination is still empirical in many respects and needs optimization to assure the economy and safety of the method and to solve the problem of odours and after-tastes caused by superchlorination. Great importance is attached to analytical methods for the determination of residual chlorine. In England, the colorimetric method DRD has gained acceptance in recent years. This method makes it possible to differentiate between all the fractions of residual chlorine.

In the USSR chlorination is the principal standard method owing to the ready availability and efficacy of chlorine. Besides the achievements mentioned above some advance has been observed in the use of electrolytic chlorine.

2.3 Proof of the reliability of disinfection

Bacteriologists do not consider coliform a reliable indicator of the innocuity of drinking water disinfected by chlorination. It has been proved that the resistance of coliforms to chlorine is far lower than the resistance of viruses. A new indicator had to be found and an intensive search began for a suitable organism. In the USA a group of *Klebsiella* capsule organisms was investigated, as discovered in the distributing system of Chicago [1]. The *Klebsiella* species is the organism which causes enteritis in children, and pneumonia, diseases of the upper respiratory tracts, sepsis, meningitis, peritonitis and inflammations of the urinary tract in adults. *Klebsiella* organisms were found in the residual slime covering the inner surface of water pipes, having survived all the stages of water purification and disinfection. Thus it was assumed that the species may be used as a conventional indicator organism. This conclusion is consistent with the viewpoint of Bauman and Ludvich (1962) that the elimination of the conventional organism may be controlled by the relationship

$$a = c \cdot t$$

where c is the amount of residual chlorine in mg/l at the end of the time of contact,
and t the time of contact in minutes.

The constant value "a" is determined by treating the organism with varying doses of chlorine, and recording the amount of residual chlorine relative to the time of contact at constant pH and pre-set temperature.

Carrol Morris [2] (1971) holds the view that the principal propositions established and checked under laboratory conditions are not applicable under operating conditions. Discussing the data of Palin on the reaction of chlorine with ammoniacal nitrogen in different proportions, the author points out that coliforms lose their indicator value in reactions beyond the break point. This is accounted for by the presence of a high quantity of hypochloric acid which is notorious for ensuring a negative result of analyses of bacterial growth. In this case a more resistant organism is needed. Tennisen and Johnson suggested the use of spores as indicators, but a thorough investigation showed that any artificially introduced indicator organism caused a change in the chemical parameters, and hence could not be regarded as a reliable indicator of the efficacy of disinfection. At the same time we know that qualitative antiviral disinfection has a near sterilising effect, so that it seems possible to replace the analysis of the growth of coliforms by plate counts of the total number of bacteria including spores, which should not exceed one unit per 1 ml or, better, per 10 ml.

In the Soviet Union too, researchers are looking for a new indicator organism answering the following requirements: easy detection, chlorine resistance equal to that of viruses, constant occurrence in water. The analytical procedure should be simple and not time-consuming. Enterococci, enteric phages, spore-forming bacteria may be considered as possible substitutes for coliforms.

Another approach to the problem of checking the effectiveness of disinfection is the attempt of Hall [3] to calculate the level of disinfection by means of mathematical equations. The author used equations to describe the kinetics of disinfection, usage of chlorine, effect of binding substances on degree of disinfection and demonstrated the possibility of indicating the level of disinfection by the concentration of the disinfectant. The author proceeded from the assumption that the inactivation of microorganisms and the reaction of a chemical substance with the disinfectant, at rates proportional to concentration, obeyed the equation

$$\mathrm{Log}_{10}\left[\frac{N_i}{N_{i,0}}\right] = \frac{k\ .\ D}{2{,}303}$$

where

$N_{i,0}$ = the concentration of organisms at time $t=0$;
N_i = the concentration of organisms at the current time;
D = Integrated concentration of the disinfectant at the pre-set time;
k = constant value of the time rate of disinfection for a given species.

In reality there are far more parameters upon which the chemical reactions depend, and more combinations of species of microorganisms with the chemical properties of water than can be expressed by a mathematical equation. Therefore, it seems hardly probable that attempts to determine the degree of disinfection by quantitative estimates of chemical reactions, rather than by the results of bacteriological analyses, would gain the support of sanitary organisations. Moreover, the practical application of Hall's method presents purely technical difficulties: it would be necessary to detect and identify the microorganisms, to calculate the constants of the rate of disinfection for each species of microorganism in water of each given quality. This procedure would take so much time that the calculated process (chlorination) would lag behind the new conditions. Still the method might prove useful in the assessment of new sources of water supply and in the design of the process of disinfection for new stations.

2.4 The harmful effect of by-products in chlorine treatment of polluted surface waters

In many countries a growing need is felt for the reuse of water resources in different branches of industry. The associated problem of water quality is usually approached in isolation from the standpoint of the particular user. Thus water supply workers consider the problem of by-products only in regard to drinking water, ignoring its relation to surface waters polluted by hundreds of substances and compounds from industrial waste waters. There can be no doubt that some of these products have properties harmful to human health, and some of them are carcinogenic. Thus, for instance, in analysing the quality of treated sewage effluents the scientists of the USA Commission on Atomic Energy discovered in them carcinogenic organic chlorine compounds. A communication by Rook, (published in the English Journal "Water Research") notes the formation of haloforms in the process of chlorination of humic waters such as chloroform, tetra-chloromethane, dichlorobromomethane, bromoform, etc. Without depreciating the importance of examining chlorine compounds and determining their threat to public health, it must be stated that this is a task for chemists and public health microbiologists. Specialists in water supply should approach the problem of by-product formation from the standpoint of general water turnover, taking into account the trophic relationships. A town situated further downstream will always receive the diluted wastewaters of an upstream town. The processes of self-purification in a river will always be disturbed by a discharge of chlorinated municipal and industrial effluents. The initial river water will always contain more harmful chlorine compounds than the amount formed upon chlorination of drinking water. The doses of chlorine used in the treatment of waste waters reach 10 mg/l and more, the doses for drinking water 5–6 mg/l. Specialists of municipal sanitation in the USSR recommend an initial dose of 3 mg/l of chlorine and a 30 minute contact time for the disinfection of biologically treated sewage. Residual chlorine should be maintained at a level of 1–2 mg/l before discharge. It has been established from experience of the biological treatment of waste waters in Moscow, that about 30–40 tons of combined chlorine formed by chemical reaction with substances contained in the effluents may be introduced daily into the R. Moscow. It is most important to know how the procedure of waste water disinfection affects the ecological relationships in natural bodies of water. It must be added that quality standards for treated waste waters (if any) are less rigorous than the standards set for the quality of water in the sources of water supply. In most cases it would be erroneous to expect that an open body of water could render innocuous, by processes of self-purification, the pollutants introduced into it.

Unless radical changes are made in the present situation, water supply undertakings and their staff will be faced permanently with the problem of preventing the formation of harmful chlorine by-products. There is an urgent need to supplement the biological treatment of waste waters by physical and chemical methods of purification with complete disinfection. In this respect ozonisation seems a very promising method for the destruction of organic compounds in the water.

2.5 Water disinfection with electrolytic sodium hypochlorite

The first experiments on the electrolysis of chloride solutions date from 1850. The first patents appeared in the USA as early as 1887 and 1898. In Russia the first industrial plant for the electrolysis of common salt began work in Petrograd in 1916 [4]. In the last decade water disinfection by the method of electrolysis of chloride solutions (common salt, sea water, underground saline water) has gained wide acceptance in the USSR, USA, England, Italy, Japan and other countries. The sodium hypochlorite used for water treatment is prepared on the spot. A new development is the method of direct electrolysis, i.e. the electrolysis of drinking water containing the required amount of chlorides. The sphere of application of electrolysis is ever widening due to its many indisputable advantages: safety, prospect of operational control of the disinfection effect by the value of residual chlorine, simplicity of construction and operation of the equipment. A whole series of metallic anodes with an active coating made of a mixture of oxides of different metals has been developed, which makes it possible to intensify considerably the process of electrolysis with a corresponding reduction of energy consumed. The first task in the further development and perfection of the electrochemical method of water disinfection, is to test under operating conditions the efficiency of different electrode materials, resistance of coating, service life and stability of operating indices.

In the USSR small electrolysis units of 1 to 100 kg/d of active chlorine are widely used in small towns and rural districts. In particular, these units are supplied with factory built packaged plants of 25 to 800 m^3/d capacity for treating surface waters.

2.6 Difficulties in maintaining the required level of residual chlorine in pipelines of great length

After treatment, the residual contaminants are inevitably carried with the water into the distribution system. It may be said *a priori*, that at a daily rate of 0,01 mg/l of suspended solids (much below standards for turbidity), the distribution system of a large town with a water consumption of 1 000 000 m^3/d will receive 10 kg of contaminants per day, or 3650 kg per year. Actually, the standards for turbidity set the value of the index at 0,5–1,5 mg/l. A part of the suspended solids reach the consumer, the rest is deposited in the pipes. Thus the distribution system becomes a kind of accumulating reservoir where the quality of the water can be no better than at its outflow from the treatment plant. In Moscow, in August 1975, the mean monthly value of turbidity was 1,0 mg/l on leaving the water treatment plant, and reached the maximum of 1,4 mg/l in the distribution system. The presence of deposits in the pipes creates an additional chlorine demand in the distribution system. Due to this the average monthly value of free residual chlorine in the distribution system at a distance of 7 km from the station was 0,17, and after water treatment 0,57 mg/l. This phenomenon depends also on the temperature of the water at the water source. If the water is supplied to the town from several plants, feed zones of each of them are formed. Consequently the consumers at the end of the zone will be supplied with water with a different chlorine content compared to consumers at its beginning. If the amount of residual chlorine reaching the consumer at the end point is considered as the control dose (control of the chlorine absorption capacity of the water along the distribution system), then organoleptic problems are likely to arise near the station, caused by overdoses of chlorine. The only way out of this situation is to improve the quality of water treatment at the station, with a simultaneous improvement of the condition of distribution systems. Only then will it become possible to maintain the required chlorine level at every point of supply in distribution systems running over great distances

3 The use of ozone for water disinfection

Ozonisation of water for disinfection purposes was considered to be a method showing much promise more than 80 years ago. The first experiments were performed in France in 1886 by de Merintence. In Russia at the 5th Water Supply Congress in 1901 a decision was taken to test the method under industrial conditions. In 1911 the first large filtration-ozonisation plant in the world was built in Petersburg. During the last decade an increasing interest in ozone has been observed, prompted by new engineering developments in the production of equipment and by many investigations showing the advantages of ozone and its particularities. Thus, for instance, Feiner and Ingols (1958) state that 99% of *E. coli* present in water are destroyed by 0,4–0,5 mg/l doses of ozone at 1 minute of contact, compared with doses of 0,25–0,3 mg/l and 16 minutes of contact for disinfection with chlorine. Kessel *et al.* (1943) provided evidence of the higher antiviral activity of ozone compared with chlorine. However, according to Clark and Chang (1959), in this case combined chlorine was used for comparison, and Hetchey (1953) indicates that the antiviral effect of ozone is but slightly stronger than that of chlorine. Suchkov [5] has shown, illustrated by the decrease of permanganate oxidability with cooling, that during the first 5 minutes ozone is expended on the oxidation of readily oxidizable organic substances, its viricidal effect coming into action only on the 6–7th minute of ozonisation, at the moment of the appearance of residual ozone in the water. The data of Suchkov are analogous to the findings of Coyne *et al.* (1964) that the polio virus is inactivated within 4 minutes with a dose of free ozone of not less than 0,3 mg/l.

Häufele and Sprockhoff [6] (1973) have shown that plant bacteria, *Candida albinas*, and the strain *Penicillium* are highly sensitive to ozone and may be inactivated by doses ranging from 0,05 to 0,7 mg/l, depending on the density of the test-strains and duration of contact. Staphyllococci require doses of 3–5 mg/l, because their conglomeration interferes with the contact of the cells with ozone. Contrary to the results obtained by Brigmann the spores of *Bac. cerus*, *Bac. subtilis* and *Bac. anthracis* proved to be particularly resistant to ozone.

The spores of *Bac. globigi* lose their germinating capacity only after 15 minutes of contact with 5 mg/l of ozone. It is significant that various mould bacillus strains require different doses of ozone to obtain apathogenicity. Virulent strain No 139 (LD–50 10^{-10}) requires more ozone for the apathogenic effect than virulent strain No 138 (LDD 50 10^{-6}). Limiting values for ozone doses to inactivate Anthrax bacillus spores, determined by laboratory means, should not be applied to industrial plants. Doses of ozone under working conditions will depend upon the quality of raw water, methods of treatment and mixing air-ozone mixture with water, contact time and other conditions. Limiting values for inactivation of three types of phages, *E. Coli* T_1, T_2, T_3, were 2 mg/l of ozone. This is also sufficient to kill 10^2 to 10^6 infection units of a bird plaque virus in one millilitre. Limiting ozone doses of 2 mg, obtained by Häufele and Sprockhoff during the investigations of Coxsackie and Polio viruses are the same as the results obtained by other investigators.

Although dozens of towns are now using ozonisation, and much scientific research has been devoted to the problem, an objective analysis of the method is not possible because of the great diversity of technological schemes and trends of research. It seems that the time has come to analyse all of the work that has been carried out, using a single classification according to the types of bacteria and viruses and conditions of inactivation. Only a large research centre having at its disposal all the requisite modern equipment and a staff of qualified virologists and chemists will be able to cope with this task. The organisation of such a centre may be sponsored by the International Institute of Ozone created in USA in 1973.

4 The antiviral efficiency of disinfection

The problem of disinfection of water infected by pathogenic viruses resistant to external effects has not yet been satisfactorily solved. However, it is most pressing as more than one hundred widely occurring viruses are known, shed by the fecal route, and only partly eliminated by sewage treatment [7]. The presence of these viruses in the environment and, in particular, in bodies of water is a source of constant danger of water-borne infection carried through water distribution systems to the consumers. It is natural that the problem of virus removal from drinking water is increasingly attracting the close attention of researchers and water supply specialists. Table 1, compiled by Clark (1971) and supplemented by more recent data, shows the geography and trends of research into this problem.

TABLE 1

Author and institution	Process investigated	Comment
J. Cookson, University of Maryland	Adsorption	Kinetics of adsorption of activated carbon
W. A. Druary, University of Tennessee	Coagulation Chlorination Adsorption	Concentration effect adsorption on carbon and ion exchange
R. C. Engelbrecht, M. Chodehuri	Coagulation Diatomaceous earth	Layer with polyelectrolyte and added activated carbon
D. H. Forster, University of Illinois	Filtration	Investigation of adsorption on silica-gel, iron oxide, complex phosphates
I. M. Folicue, Institute of hygiene, Nancy, France	Coagulation	Sludge blanket clarifier
E. M. Nupen, National Institute of Water Examination, Pretoria, South Africa	pH	Time of survival at pH up to 11,2
V. P. Olivieri, J. Hopkins univ.	Chlorination	The role of RNA and protein determination, life cycle after destruction
C. F. B. Pointer, M. W. B. Labs, London, England	Filtration	Slow sand filters
H. Shuval, Hebrew univ. Jerusalem, Israel	Ozonisation	Efficiency of disinfection and measurement of ozone in the water
O. I. Sprole, Maine univ.	Ozonisation	Kinetics and study of mass transport
D. V. York, W. A. Druary, USA	Coagulation	Chemical coagulation
E. L. Lovtsevich, USSR	Chlorination Ozonisation	Dosage, time of contact, survival
L. A. Kulsky, L. I. Globa, USSR	Adhesion Adsorption	Natural clay minerals

Without discussing in detail the data of the table the results obtained may be summed up as follows:

(a) Water disinfection may be achieved only after good clarification,

(b) Complete non-specific inactivation of viral particles is reached at a redox potential of 550–600 mV.

On the basis of investigations performed under experimental and pilot plant conditions, chlorine and ozone are considered effective disinfectants acceptable for use in municipal water works. The mechanism of action consists of the destruction of the protein envelope of enteric viruses. The antiviral effect of free chlorine forms is about 50–100 times greater than the effect of combined forms. Actually the viricidal effect is produced by the amount of free chlorine left after the required time of contact. Table 2 gives the results of enteric virus inactivation by chlorine and ozone.

All of the authors are of the opinion that the effectiveness of water disinfection by chlorine and ozone in virus removal depends on the initial concentration of viruses, degree of pollution by organic substances,

TABLE 2

Virus	Inactivation effect, %	Time of contact (min)	Residual reagent (mg/l)	Author
Chlorine inactivation (Cl_2)				
Infectious hepatitis virus	99,93	30	0,3	Chang, 1968, USA
Poliovirus, type 1	99,46	60	0,3	Lovtsevich, 1973, USSR
Ozone inactivation (O_3)				
Poliovirus, type 1	complete	30 4	0,4 0,3	Coin, 1964, 1967, France
Enteric group	99–99,9	10	0,1	Chang, 1968, USA
Poliovirus, type 1	99,99	2,5 5 10	0,23 0,18 0,14	Schaffernot, 1970, USA
Poliovirus, type 1	complete	16	0,4	Ryabchenko, Lovtsevich, 1973, USSR
Poliovirus, type 1	complete	5	1,5	Katsnelson, Kletinger, 1974, USA

amount of the free agent in the water and time of contact. Under definite conditions the antiviral effect of ozone is stronger than that of chlorine. Furthermore, ozone has the additional advantage of improving the organoleptic properties of the water. Nevertheless, chlorination in the world's water-supply practice is more widely used, whereas ozonisation is only beginning to be applied on a comparable scale at new water-treatment plants now being built. It is hardly possible to compare the antiviral effect of chlorine and ozone under the operating conditions of an operational plant, owing to the lack of a common system of evaluation even for laboratory work. If drinking water is not sufficiently well clarified, then ozone may be expended on the chemical oxidation of organic substances, rather than on virus destruction. But if the water is well clarified by additional treatment with activated carbon, the effect of virus inactivation may be obtained with the aid of non-dissociated hypochloric acid without additional ozonisation.

To decide which of the two processes of chlorination or ozonisation is the most efficient, the antiviral effect of these processes must be evaluated under operating conditions for different schemes of treatment, depending on the quality of water. But such an evaluation is impeded by the fact that direct virological control of drinking water quality presents as yet many difficulties. Essential for a successful virological analysis is the development of a standard method of isolation (concentration) and detection of enteric viruses. In this respect the development of a method for quantitative definition of viral water contaminations with the aid of electron microscopy seems especially promising. As both the virological control of drinking water and the evaluation of sources of drinking water supply by the criterion of virological contamination must be performed by qualified specialists, and require costly equipment and high general expenditure, the task should be entrusted to special virological centres.

Despite the unquestionable advances in the study of virus removal in the process of water treatment and availability of certain little used aids (such as clay and other natural sorbents), it should be always borne in mind that the anthropogeneous character of virus contamination necessitates the prevention of the pollution of water sources of domestic sewage, and the development of measures for virus destruction at sewage treatment plants.

5 Disinfection of newly built and repaired mains

Disinfection is the last stage in the potable water treatment process. After disinfection the water acquires standard properties, and after sanitary control it becomes a marketable product of the water plant. It is a specific product, if only because it is supplied to the consumer at any time and in sufficient quantity by the same water plant through a distribution system. Usually there are no intermediaries in the scheme "water system—consumer". Furthermore, the product must neither lose the qualities set by the standard, nor acquire new ones which are not specified. And lastly, there are no technical means for a deliberate modification of water quality in the distribution system. In this case the amount of residual chlorine is considered. Therefore, bacterial contamination must be prevented within the distribution system.

The problem of public health in water supply involves many factors and preventive measures which must be taken into consideration. The quality of water is permanently dependent on the length of the mains, the material of pipes, time of operation, physical condition of pipes, protective coating, number of dead end mains, deposits in the distribution systems, proper maintenance and other factors. Most important is the degree of contamination of the ground in which the mains are laid from the water treatment plant to the distribution systems: the presence of badly laid out distribution districts, industrial plant sites, polluted ravines and water courses. In cases of bursts with pipe rupture or leakage, the bacterial purity of the water will depend on the cleanliness of the ground at the point of the damaged pipe and on proper repairs. Thus the elimination of bacterial contamination will be more rapid and effective in localities where sanitary preventive practice had been previously well organised along the route of the main.

Investigations of the bacterial condition of water mains have shown the existence of a bacterial flora on the inner surface of the pipes covered by a film of corrosion products, iron bacteria, calcium and magnesium salts, residual aluminium etc. The quantitative and qualitative composition of the bacterial flora depends on the physical and chemical composition of water at its source, method and degree of purification and disinfection at the water treatment plant and the condition of the main in its broadest sense. In water mains of southern towns, using water from surface sources, representatives of zoobenthos such as oligochates, nematodes, chironomids and crustaceans may be encountered. It is noteworthy that in these cases bacteriological analyses for pathogens give negative results, so that the staff is given no imperative information calling for action. Besides, many authors point out, viruses may be encountered in the intestinal tract of representatives of zoobenthic fauna. Under these conditions the standard requirement, that "water should contain no microorganisms visible to the naked eye" does not guarantee the distribution system against microbial contamination. Cases of blue-green algae occurrences in the mains have been reported. These circumstances may be accounted for by a bias towards high rates of filtration, up to 10–12 m/hour, and the unsatisfactory state of the filter media. In the USSR, after the analysis of the work of hundreds of surface water treatment plants, it was found advisable to reduce the rate of filtration in accordance with the quality of water at its source. Such information is given in table 3.

Nevertheless, despite measures taken to improve water treatment through all the links of the technological process, water entering the distribution system may still contain pathogenic micro-organisms. Moreover, bacterial contamination may originate directly within the mains from pollution during installation, leaking joints, small flaws or cross connections with industrial water systems.

TABLE 3

Type of rapid filters	Normal rate of filtration, m/hour	
	1962	1974
Single-layer media various grain sizes	6–10	5,5–8
Dual media	10	8–10
Biflow filters	12	10–12

Disinfection of new water mains is a far easier task and is performed in most countries according to the instructions. The methods used have much in common in many countries. Becker [8] states that the disinfection of new mains consisting of pipes of 300 mm and larger is obligatory. In repair works the need for disinfection is determined by the executive. Doses of 100 mg/l and contact of minimum 48 hours are used in the chlorination of new mains. After flushing the main to the value of residual chlorine, two samples are taken at 20 minute intervals from fixed points. Samples may be considered to be representative when collected from a hydrant continuously flushed with strong chlorine solution. Samples collected from the same hydrant after it had been closed for 24 hours may yield erroneous results. Residual chlorine is determined by the iodine-starch method. With a 0,1128 N sodium thiosulphite solution and a sample of 100 ml, 1 ml of solution after titration will give a chlorine concentration of 40 mg/l. During repairs one has to rely on a thorough flushing of the pipes and the presence of residual chlorine in water.

Rossum (USA) reports that in California before 1935 only large diameter pipelines were disinfected. According to an instruction of 1935 all new mains were to be treated with a chlorine solution of 50 mg/l for 24 hours. Unsatisfactory results from disinfection were observed with the introduction of mechanical couplings, which form annular dead spaces at pipe joints. The problem was solved when chlorine tablets appeared. These were placed in the annular space of the coupling and glued to the inner surface of the pipes. It is impossible to flush the pipelines before chlorination, and because of this greater care must be taken to keep the mains clean during construction. The tablet method cannot be used in repair work.

Hutchinson [9, 10] comments upon a problem which arose in 1940, associated with the growth of bacteria similar to coliforms in jute gaskets for pipe connections. Rather than use superdoses of some hundreds of mg/l of chlorine, the problem was solved by replacing jute by synthetic materials. The use of soap tampons in flushing, and the introduction of hypochlorite tablets glued to the upper part of the pipe, has meant progress in the disinfection of new mains. The use of couplings with an annular dead space may be the cause of unsatisfactory bacterial analyses. The bacterial flora in the annular spaces of the couplings are unaffected by even large doses of chlorine, which are absorbed by the soap lubricant in the annular space. The only methods of control that may be recommended under these circumstances are the observation of sanitary precautions and chlorination prior to hydraulic structural tests.

In the USSR the current instruction for the disinfection of new mains, approved by the USSR Ministry of Public Health, prescribes the disinfection of all new mains by doses of active chlorine ranging from 75 to 100 mg/l, ensuring 50% dose at the end point of the pipeline. The minimum time of contact is 6 hours. At the end of the flushing period the dose of residual chlorine should be 0,3–0,5 mg/l. Satisfactory analyses obtained for 2 samples collected at intervals of 20 minutes from each fixed point are evidence of effective disinfection. During mains repairs the doses of active chlorine are either 40 mg/l with a 24 hour contact or 75–100 mg/l and a 6 hour contact.

Pipes of 900 mm diameter and larger must be examined from inside by a representative of the operating personnel for cleanliness. The observance of this instruction ensures satisfactory disinfection and rarely is a second attempt needed. The acceptance report is usually drawn up in the presence of a representative of the public health service.

The disinfection of an operating main after repair is far more complicated in practice. Special attention should be given to the flushing and disinfection of dead ends, where different kinds of contamination may take place. During the preventive flushings of 274 dead ends, performed in 1975 in Russian cities, nematodes were discovered in three lines, despite favourable bacteriological analysis. Data on analyses of water from surface sources are presented in Table 4.

TABLE 4

Year of construction	Diameter mm	Length m	Analyses			Date
			Bacter.	Chem.	Nematode organisms in 5 l	
1914	125	110	satisf.	satisf.	8	11,06
			satisf.	satisf.	none	12,06
1934	150	340	satisf.	satisf.	3	25,07
			Coli index 14 per l	satisf.	9	25,08
			satisf.	satisf.	none	29,08
1962	200	180	dense growth	turbidity 5,6 mg/l	4	8,07
			satisf.	satisf.	none	10,08

In all cases an abundant development of benthos organisms was observed in the surface sources. The suspended solids deposited in the pipes form a favourable substrate for their propagation.

6 Development of control and laboratory analyses

Methods of laboratory analysis known today, such as iodometric method, amperometric titration, orthotolidine method, leuco crystal violet, methyl orange and syringaldazine procedures have a common shortcoming. The reagents employed alter the pH of the medium and give no separable determination of free and combined chlorine. Even the well established orthotolidine method does not allow for a separate determination of free and combined chlorine. This may be obtained only by cooling the water sample to 1°C in order to ensure the reactions of chloramine formation. The orthotolidine arsenite used (OTA) produces a slow reaction of arsenite with chloramine, which leads to overestimation of the true value of free chlorine. All of these methods involve the acidification of the water sample which disturbs the equilibrium of the chlorine-chloramine system. Therefore an additional stage was introduced, consisting of the addition of neutral orthotolidine (NORT), stabilised neutral orthotolidine (SNORT) and amperometric titration. Then the need was felt for simplifying the analyses of chlorine compounds by reducing the number of reagents.

Among the latest investigations of control and laboratory analyses the work of Palin [11] is of special interest. After a long search the author decided on using NN-Diethyl-p-phenylene diamine (DPD) instead of NORT for chlorine determinations. Following the first publication, which appeared in the JAWWA in 1957, the method was developed and supplemented by procedures for determining components of different mixtures of disinfectants. Great progress in this regard was the use of glycine for the determination of chlorine dioxide and combinations of ozone and chlorine. Glycine combines rapidly with chlorine and instantly destroys ozone. The DPD method in its present form allows for the differentiated determination of halogens, chlorine dioxide and ozone in water and can be applied under operational conditions too, because the reagent may be used in the form of tablets and powder. In our opinion the DPD method shows promise for the design of automatic devices for measuring residual chlorine in water. The equipment must consist of separate apparatus for differentiated analyses and be controlled by digital computers. The construction of the apparatus sets will depend on the reagents used for water treatment and disinfection. The equipment for the DPD method is expected to become available in 1980.

7 Discussion

Those publications reviewed for this paper (about 150 covering the last three years) have much in common in that they were all written by analytical chemists, microbiologists, virologists, working in large scientific research centres or laboratories. Their work, carried out with the aid of complicated and costly equipment, is concerned mainly with the mechanism of chlorination and the bactericidal and viricidal effect of various oxidants. This research gives us a deeper insight into the processes going on in the invisible world of microbes, but it does not provide us with practical recommendations. For years the procedure of disinfection has been managed by operators using automatic chlorine feeders and residual chlorine analysers. The results of investigations have not influenced the essence of the chlorination process in water treatment plants and distribution systems. This statement reflects the attitude of a water supply specialist, who expected to obtain something positive to apply in the practice of chlorination, but succeeded only in improving his knowledge of the subject. All the investigations only prove the old truth, that if water is well treated and the distribution system kept clean, the effect of disinfection is ensured. This extreme conclusion is expressed deliberately in order to draw attention to the gap between scientific achievements and the limitations of technical methods for controlling the processes of disinfection. Whilst speaking of the progress achieved in water disinfection, it is necessary to state our needs for the future:

(a) development of automatic devices for chemical quality control with computers and for estimation of chlorination (or ozonisation) performance.

(b) the development of computerised devices for chlorine (or ozone) dosage.

Nothing can be said as yet of prospects for the creation of automatic bacteriological control devices since all bacteriological analyses are based on visual observations involving the isolation and identification of microorganisms, but we may point out the urgent need for simple, rapid and reliable methods of bacterial control to be used by the personnel of water treatment plants.

Another requirement for the future is the evaluation of the properties of ozone and the determination of its place in the technological scheme of water treatment and disinfection.

And, finally, it is necessary to combine investigations in the field of sewage and potable water disinfection, considering it as a single technological process of natural water circulation.

In future chlorination will remain the major process for water disinfection. Extension of knowledge in the field of water disinfection creates new problems, which may be solved by improving chlorination, combined with ozonisation.

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Progrès récents dans la désinfection de l'eau

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1 Diverses méthodes de désinfection de l'eau

1.1 Classification

La grande diversité des techniques et des moyens de désinfection de l'eau oblige à réviser la classification actuelle qui divise toutes les méthodes en physiques ou chimiques. Une nouvelle classification des méthodes de désinfection a été proposée par la Chaire d'alimentation en eau de l'Institut de génie civil de Léningrad. Elle est basée sur l'analyse de plus de 500 publications. Cette classification distingue:

(a) Les méthodes chimiques, basées sur l'emploi d'agents bactéricides ou neutres d'une façon non-électrique: chlore, iode, brome, coagulation et floculation.

(b) Les méthodes électro-chimiques, dans lesquelles on emploie de l'énergie électrique pour former l'agent bactéricide ou neutre: ozonation, traitement aux ions d'argent, électrolyse, électroflottation.

(c) Les méthodes physiques, utilisant l'effet d'une énergie mécanique: ultrafiltration, traitement par la chaleur, irridiation par ultraviolets ou rayons gamma, champs ultra-sonores et magnétiques.

(d) Méthodes de traitement électriques, dont le mécanisme d'action comprend, en-dehors des effets déjà mentionnés, l'action sur les corps bactériens polarisés ou ayant un bipôle dur: électrophorèse, électrocoagulation, décharge électrique, champs de très haute fréquence.

La classification proposée part d'un point de vue colloïdo-chimique, l'eau contaminée étant considérée comme un système biodispersé relativement monodispersé et stable. Cette approche permet de classifier les méthodes en se basant sur l'agent désinfectant introduit, la transformation du système en état instable et l'isolation de la biophase aggrégée du milieu. Il faut dire que la création d'une classification universelle pour les spécialistes de l'eau est presque impossible, chacun d'eux l'ayant inévitablement envisagée d'un certain point de vue. La possibilité d'appliquer chaque méthode dépend de nombreux facteurs et conditions locales, comme la qualité de l'eau initiale, les perspectives de sa modification, la disponibilité de réactifs et d'installations, la capacité de la station, la présence ou l'absence de terrain utilisable, la longueur du réseau de distribution, les goûts des consommateurs, les traditions, etc. ... Souvent la simplicité d'une méthode, considérée comme un avantage pour une petite station, se montre défavorable pour une grande station. Dans le choix d'une méthode de désinfection plutôt que d'autres, il faut être guidé par des considérations de technique et d'économie, en tenant compte des insuffisances inhérentes à chacune d'elles.

1.2 Méthodes pratiquement utilisées

Les méthodes de désinfection chimiques sont très répandues dans les services d'eau et d'abord la chloration. On emploie en général l'iode et le brome surtout pour les piscines. L'extension de leur emploi pour la désinfection de l'eau potable est gênée par l'activité biologique de ces réactifs qui présente un danger pour la santé publique, et par leur coût éléve. La coagulation et la floculation sont partiellement utilisées pour intensifier la sédimentation des bactéries et des virus sur les particules de floc.

Parmi les méthodes électro-chimiques, deux sont maintenant utilisées dans l'industrie de l'eau: l'ozonation et l'electrolyse de composés chlorés. Leur effet bactéricide puissant, la facilité de disposer d'énergie électrique et les progrès techniques réalisés dans la conception de l'équipement font que ces méthodes sont de plus en plus généralement utilisées.

L'emploi de méthodes physiques sera restreint pendant un certain temps aux rayons ultra-violets qui exercent un effet presque instantané sur les gènes des cellules et n'exigent pas l'introduction dans l'eau de corps étrangers. B. NIETSCH (RFA 1973) note que la molécule d'acide désoxyribonucléique, de 265 nm de taille, contenue dans le gène, a une absorption spectrale maximale pour une radiation de longueur d'onde 265 nm. Les autres méthodes en sont toujours aux recherches de laboratoire.

En URSS, les rayons UV sont utilisés pour la désinfection de l'eau souterraine quand l'index *E. coli* est inférieur à 1 000 par litre. Les appareils mis au point par l'Académie d'administration municipale ont diverses capacités et valent pour des installations sous pression ou gravitaires.

Toutes les méthodes mentionnées dans la classification peuvent intéresser le spécialiste et seront, sans doute, mises au point pour des buts spéciaux. Mais on peut dire avec certitude que la chloration restera pendant longtemps la méthode fondamentale de désinfection de l'eau potable dans les réseaux de distribution d'eau. Bien qu'il y ait encore beaucoup de questions non résolues qui seront discutées ci-dessous, cette méthode a de nombreux avantages: rendement élevé, simplicité du contrôle, faible coût du réactif, effet de désinfection à long terme. Il n'y a guère de méthode qui possède tous ces avantages.

En URSS la chloration est considérée comme la méthode la plus sûre et la plus pratique de désinfection pour les réseaux de distribution d'eau. Au cours des dernières années, la chloration de l'eau potable a été complétée par l'ozonation.

2 Revue de l'état actuel de la chloration

2.1 Bases théoriques de la chloration

La chloration de l'eau potable a été employée pour la première fois il y a quelques 80 ans, et celle des eaux d'égout il y a plus d'un siècle. Malgré cela, nous n'avons pas encore une conception exhaustive du mécanisme de l'effet bactéricide du chlore et de la chimie de la chloration. Les recherches scientifiques en ces domaines continuent. Dans le rapport général présenté au 5ème Congrès de l'AIDE ont été résumés les résultats atteints dans la mise au point de conceptions théoriques et leur application pratique. Des progrès considérables avaient été obtenus dans l'amélioration des méthodes de contrôle. Une grande attention était accordée dans le rapport à l'emploi du chlore pour la désinfection de l'eau en raison de l'acceptation presque universelle de cette méthode comme à peu près la seule utilisée pour la désinfection de l'eau. Ses avantages économiques, techniques et sanitaires sont parfaitement connus.

Mais cette méthode a ses inconvénients inhérents substantiels; le principal est la détérioration du goût de l'eau et la possibilité seulement indirecte de contrôler la sécurité de la désinfection par la teneur en chlore résiduel. Quelques chercheurs estiment que fixer la quantité de chlore résiduel peut induire en erreur, car la quantité de chlore dans l'eau n'est pas équivalente à sa qualité. Les spécialistes discutent encore de la signification précise des termes chlore résiduel "libre" et "combiné". Aucun contrôle de l'efficacité de la chloration par la teneur en chlore résiduel ne sera satisfaisant si la forme du chlore résiduel n'est pas précisée. Mais, comme il a déjà été dit, le terme "chlore résiduel" est vague et imprécis; il faut souligner que l'effet de la désinfection dépend de la composition du chlore résiduel plutôt que du processus de chloration lui-même. Aujourd'hui donc, en pratique, il est exigé de spécifier si l'on utilise le chlore combiné ou libre comme indicateur de l'efficacité de la désinfection. Les normes soviétiques exigent 0,3–0,5 mg/l de chlore libre ou 0,8–1,2 de chlore combiné.

2.2 Progrés dans la chloration

Il n'y a pas unanimité d'opinion sur les progrès réalisés en matière de chloration. Selon le rapport [1] du Comité sur la désinfection de l'eau (1975) de l'American Water Works Association, il n'y a pas eu de progrès importants depuis deux décennies dans les méthodes de chloration de l'eau potable. La découverte du break point (1939), la mise au point du titrateur ampérométrique (1942) et d'un enregistreur de chlore résiduel (1952), les recherches de Feyer, Morris et Chang à Harvard (1944) et l'explication de la réaction du break point par Granstrom (1954) sont tous le résultat d'une recherche scientifique approfondie menée dans les années 40 et au début des années 50. Au cours des années suivantes, la recherche a visé surtout la désinfection des eaux d'égout et de l'eau recyclée. Dans ce contexte, certains problèmes associés à la désinfection de l'eau potable ont surgi, appelant de nouvelles recherches. On a trouvé que la chloration au-delà du break point des eaux d'égout à faible teneur en azote ammoniacal mène à la formation d'une dichloramine extrêmement résistante en présence de chlore libre dans le chlore résiduel total. On estime que cette fraction est un composé d'azote organique avec la monochloramine et le chlore libre, qui titre comme la dichloramine mais n'a presque pas de pouvoir bactéricide. Cela veut dire qu'il faut mettre au point de nouvelles méthodes d'analyse pour différencier la monochloramine et les autres composés chlorés compris dans le chlore résiduel. Entretemps, la méthode la plus couramment utilisée aux Etats-Unis pour mesurer le chlore résiduel est l'orthotolidine qui ne permet pas de distinguer le chlore libre du chlore combiné. La découverte de composées chlorés organiques volatiles dangereux a soulevé le problème d'empêcher leur formation. On peut dans ce but utiliser la chromatographie gazeuse et la spectrographie de masse.

En Grande-Bretagne, la chloration a fait l'objet de recherches approfondies au Centre scientifique de la recherche sur l'eau, et a été reconnue comme méthode normale de désinfection de l'eau. Les recherches, dont les résultats ont été primitivement publiés il y a plus de 30 ans, ont été répétées pour tenir compte des nouveaux problèmes pratiques d'odeur et de goût, de réaction du chlore avec d'autres désinfectants, de destruction des virus et de désinfection des réseaux de distribution. Cependant la pratique de la chloration est toujours empirique à bien des points de vue et exigerait une optimisation pour assurer l'économie et la sécurité de la méthode et pour résoudre le problème des odeurs et des goûts provoqués par la super-chloration. Une grande importance est attachée aux méthodes analytiques pour la détermination du chlore résiduel. En Angleterre, ces dernières années, la méthode colorimètrique DRD a gagné du terrain. Elle permet de différencier toutes les fractions du chlore résiduel.

En URSS, la chloration est la méthode la plus répandue, en raison de la disponibilité et de l'efficacité du chlore. Outre les progrès mentionnés ci-dessus, on observe un développement de l'emploi du chlore électrolytique.

2.3 Preuve de la fiabilité de la désinfection

Les bactériologistes ne considèrent pas que les coliformes sont une preuve valable de l'inocuité de l'eau potable désinfectée par chloration. Il a été prouvé que la résistance des coliformes au chlore est bien moindre que la résistance des virus. Il fallait trouver un nouvel indicateur, et des recherches approfondies ont commencé pour trouver un organisme convenable. Aux Etats-Unis, on a étudié un groupe d'organismes Klebsellia à capsule, qui a été découvert dans le réseau de Chicago [1]. Klebsellia est la cause d'entérite chez les enfants et de pneumonie, de maladie du tractus respiratoire supérieur, de septicémie, de méningite, de péritonite et d'inflammation du tractus urinaire chez les adultes. On a trouvé des Klebsellia dans l'enduit qui couvre la paroi interne des tuyaux; ils avaient survécu à toutes les étapes de purification et désinfection de l'eau. On a donc admis que l'espèce pouvait être utilisée comme organisme indicateur conventionnel. Cette conclusion est en accord avec le point de vue de Bauman et Ludvich (1962) que l'élimination de l'organisme conventionnel peut être contrôlée par la relation

$$a = c \cdot t,$$

dans laquelle c est la teneur en chlore résiduel en mg/l à la fin du temps de contact et t le temps de contact en minutes.
La constante a est déterminée en traitant l'organisme avec diverses doses de chlore et en enregistrant la teneur en chlore résiduel pour le temps de contact à pH constant et pour une température fixée.

Carrol Morris [2] (1971) est d'avis que les principales propositions établies et vérifiées au laboratoire ne sont pas applicables en exploitation. Discutant les chiffres de Palin sur la réaction du chlore avec l'azote ammoniacal en différentes proportions, cet auteur souligne que les coliformes perdent leur valeur d'indicateur dans les réactions au-delà du break point. Cela est attribué à la présence d'une grande quantité d'acide hypochlorique dont on sait qu'il donne un résultat négatif lors des analyses de croissance bactériologique. Il faut en ce cas un organisme plus résistant. Tennisen et Johnson ont suggéré l'emploi des spores comme indicateurs, mais des recherches approfondies ont montré que tout organisme indicateur artificiellement introduit provoque un changement des paramètres chimiques et ne peut donc pas être regardé comme indicateur fiable de l'efficacité de la désinfection. En même temps, nous savons que la désinfection antivirale qualitative possède un effet presque stérilisant, de sorte qu'il semble possible de remplacer l'analyse de la croissance de coliformes par un comptage sur plaque du nombre total des bactéries comprenant des spores ne qui, devraient pas dépasser 1 par ml ou mieux par 10 ml.

En URSS également, des recherches sont en cours pour trouver un nouvel organisme indicateur répondant aux exigences suivantes: facilité de détection, résistance au chlore égale à celle des virus, présence constante dans l'eau. La procédure analytique devrait être simple et rapide. Les entérocoques, les phages entéritiques, les bactéries sporulantes peuvent être considérés comme des substituts possibles aux coliformes.

Il y a une autre approche du problème de vérifier l'efficacité de la désinfection. Hall [3] a essayé de calculer le niveau de désinfection au moyen d'équations mathématiques. Il utilise des équations mathématiques pour décrire la cinétique de la désinfection, l'utilisation du chlore, l'effet des substances agrégeantes sur le degré de désinfection, et il a montré la possibilité d'indiquer le niveau de désinfection par la concentration du désinfectant. Il procède de l'assomption que l'inactivation des microorganismes et la réaction de la substance avec le désinfectant, à des taux proportionnels à la concentration, obéit à la relation:

$$\log_{10} \frac{N_i}{N_{i,0}} = \frac{k \cdot D}{2{,}303}$$

où:

$N_{i,0}$ = Concentration en organisme au tempts t = 0;
N_i = Concentration en organisme au moment actuel;
D = Concentration intégrée du désinfectant au temps pré-fixé;
k = Constante de l'évolution dans le temps de la désinfection pour une espèce donnée.

En réalité, les réactions chimiques dépendent de beaucoup plus de paramètres et il y a plus de combinaisons d'espèces de microorganismes avec les propriétés chimiques de l'eau que ne peut en exprimer une équation mathématique. Il semble donc peu probable que les essais pour déterminer le degré de désinfection par des estimations quantitatives de réactions chimiques plutôt que par les résultats d'analyses bactériologiques puissent obtenir l'accord des organismes sanitaires. De plus, l'application pratique de la méthode de Hall présente des difficultés purement techniques: il serait nécessaire de détecter et d'identifier les microorganismes, de calculer les constantes de la vitesse de désinfection pour chaque espèce de microorganisme dans l'eau de chacune des qualités données. Cette procédure prendrait tellement de temps que la chloration calculée serait en retard sur la situation nouvelle. Cette méthode peut cependant se montrer utile pour étudier les nouveaux captages et pour concevoir la procédure de désinfection pour les stations nouvelles.

2.4 Effet nocif des sous-produits du traitement par le chlore des eaux de surface polluées

Dans beaucoup de pays, on ressent un besoin croissant de recycler l'eau de différentes branches d'industrie. Le problème de qualité que cela implique est habituellement envisagé du point de vue d'une seule d'entre elles. Ainsi les distributeurs d'eau ne considèrent le problème des sous-produits que du point de vue de l'eau potable, ignorant ses relations avec les eaux de surface polluées par des centaines de substances et de composés des eaux usées industrielles. Il n'est pas douteux qu'une partie de ces produits ait des propriétés nocives pour la santé humaine, et que certains d'entre eux soient cancérogènes. Ainsi, par exemple, en analysant la qualité des effluents d'égout traités, les chercheurs de la commission américaine de l'énergie atomique y ont trouvé des composés organiques chlorés cancérogènes. Une communication de Rook, publiée dans la revue anglaise "Water Research", note la formation d'haloformes lors de la chloration des eaux humiques: chloroforme, tétra-chlorométhane, dichlorobromométhane, bromoforme, etc.... Sans vouloir déprécier l'importance d'examiner les composés chlorés concrets et de déterminer la menace qu'ils présentent pour la santé publique, il faut admettre que c'est une tâche pour les chimistes et les bactériologistes de la santé publique; les spécialistes en distribution d'eau doivent approcher le problème de la formation de sous-produits du point de vue du cycle général de l'eau, tenant compte des relations trophiques. Une ville située en aval recevra toujours les eaux usées diluées d'une ville en amont. Le processus d'auto-purification dans une rivière sera toujours troublé par le rejet d'effluents municipaux et industriels chlorés. L'eau de rivière initiale contiendra toujours plus de composés chlorés nocifs qu'il n'en sera formé lors de la chloration de l'eau potable. Les doses de chlore utilisées dans le traitement des eaux usées atteignent 10 mg/l et plus, les doses pour l'eau potable 5 à 6 mg/l. Les spécialistes en assainissement urbain en URSS recommandent une dose initiale de 3 mg/l de chlore et 30 mn de contact pour la désinfection des eaux d'égout traitées biologiquement. Le chlore résiduel doit être maintenu à 1–2 mg/l avant rejet. Il a été établi, d'après l'exemple du traitement biologique des eaux d'égout de Moscou, que 30 à 40 t de composés chlorés formés par réaction chimique avec les substances contenues dans les effluents peuvent être rejetés chaque jour dans la Moscowa. Il est très important de savoir comment ce procédé de désinfection des eaux d'égout affecte les relations écologiques dans les eaux réceptrices naturelles. Il faut ajouter que les normes de qualité pour les eaux d'égout traitées, quand elles existent, sont moins rigoureuses que les normes fixées pour la qualité de l'eau captée pour l'alimentation en eau. Dans beaucoup de cas, ce serait une erreur de s'attendre à ce qu'une eau superficielle puisse détruitre par autopurification les polluants introduits.

Si l'on n'apporte pas de modifications radicales à la situation actuelle, les stations de traitement d'eau potable seront confrontées en permanence au problème d'empêcher la formation de sous-produits chlorés nocifs. Il est urgent d'ajouter au traitement biologique des eaux usées des méthodes physico-chimiques de purification amenant une désinfection complète. Sous cet aspect, l'ozone semble une méthode bien plus prometteuse pour la destruction des composés organiques dans l'eau.

2.5 Désinfection de l'eau par l'hypochlorite de sodium

Les premières expériences sur l'électrolyse des solutions de chlorures datent de 1850. Les premiers brevets sont apparus aux Etats-Unis dès 1887 et 1898. En Russie, la première usine industrielle pour l'électrolyse du sel de cuisine commença à fonctionner à Petrograd en 1916 [4]. Au cours de la dernière décennie, la désinfection de l'eau par électrolyse de solutions de chlorure (sel marin, eau de mer, eau des salines souterraines) a été de plus en plus utilisée en URSS, aux Etats-Unis, en Angleterre, Italie, Japon et autres pays. L'hypochlorite de sodium utilisé pour le traitement de l'eau est préparé sur place. Un nouveau progrès est l'électrolyse directe, c'est-à-dire l'électrolyse d'eau potable contenant la quantité de chlorure nécessaire. Le champ d'application de l'électrolyse s'élargit sans cesse en raison de ses avantages indiscutables: sécurité, possibilité de contrôle en exploitation de l'effet désinfectant par la teneur en chlore résiduel, simplicité de construction et d'exploitation de l'équipement. Tout une série d'anodes métalliques ayant un revêtement actif d'un mélange d'oxydes de différents métaux ont été mis au point, ce qui rend possible d'intensifier considérablement le processus d'électrolyse avec réduction correspondante de la dépense en énergie. Le premier travail pour développer encore et perfectionner la méthode électrochimique de désinfection de l'eau sera de tester dans les conditions d'exploitation le rendement des diverses électrodes, la résistance des revêtements, la durée de vie, la stabilité des index d'exploitation.

En URSS de petites unités de 1 à 100 kg/j de chlore actif sont largement utilisées dans les petites villes et les régions rurales. En particulier, ces unités sont fournies avec des stations préfabriquées de 25 à 800 m^3/j de capacité pour le traitement des eaux de surface.

2.6 Difficultés pour maintenir le niveau requis de chlore résiduel dans les conduites de grande longueur

Après le traitement, les contaminants sont inévitablement entraînés avec l'eau dans le réseau de distribution. On peut dire à priori que pour une teneur moyenne de 0,01 mg/l de matières en suspension (bien inférieure aux normes pour la turbidité), le réseau d'une grande ville dont la consommation en eau est d'un million de m^3/j recevra 10 kg/j de contaminants, ou 3 650 kg/an. En fait, la norme pour la turbidité tient compte d'un indice de 0,5–1,5 mg/l. Une partie des solides en suspension atteint le consommateur, le reste se dépose dans les conduites. Ainsi le réseau devient une sorte de réservoir d'accumulation où la qualité de l'eau ne peut pas être meilleure qu'à son départ de la station. A Moscou, en août 1975, la valeur moyenne mensuelle de la turbidité était 1,0 mg/l dans le réseau de distribution. La présence de dépôts dans les conduites crée une demande en chlore supplémentaire dans le réseau de distribution. De ce fait, la teneur moyenne en chlore résiduel libre à 7 km de la station était de 0,17 au lieu de 0,57 après le traitement. Ce phénomène dépend aussi de la température de l'eau à la prise d'eau. Si la ville est alimentée par plusieurs stations, il se forme des zones desservies par chacune d'elles. En conséquence, les consommateurs à la limite des zones recevront de l'eau dont la teneur en chlore sera différente de celle reçue par les consommateurs du début de la zone. Si l'on considère la teneur en chlore résiduel atteignant le consommateur à la limite de la zone comme teneur de contrôle (contrôle de la capacité d'absorption en chlore de l'eau dans le réseau), il pourra surgir des problèmes de goût à proximité de la station en raison des doses excessives de chlore. La seule façon de résoudre ce problème est d'améliorer la qualité du traitement de l'eau à la station, ce qui améliore simultanément la situation dans le réseau. Ce n'est que de cette façon qu'il sera possible de maintenir le résiduel de chlore requis en tout point de la distribution, pour les réseaux de grande longueur.

3 Emploi de l'ozone pour la désinfection de l'eau

L'ozonation de l'eau pour la désinfection était considérée comme une méthode très prometteuse il y a plus de 80 ans. Les premiers essais ont été réalisés en France en 1886 par de Merintence. En Russie, lors du 5ème Congrès des distributions d'eau en 1901, la décision fut prise d'essayer la méthode à l'échelle industrielle. En 1911, la première grande station du monde de traitement par filtration-ozonation fut construite à Petersburg. Au cours de la dernière décennie, un intérêt croissant pour l'ozone s'est manifesté, appuyé par de nouvelles réalisations techniques dans la production d'équipement et par de nombreuses recherches montrant les avantages de l'ozone et ses particularités. Ainsi, par exemple, Feiner et Ingols (1958) établissent que 99 % des *E. coli* présents dans l'eau sont détruits par 0,4–0,5 mg/l d'ozone en une minute de contact, alors qu'il faut 0,25–0,3 mg/l de chlore et 16 mn de contact pour le même résultat. Kessel *et al* (1943) donnent la preuve de la plus grande activité antivirale de l'ozone par rapport au chlore. Mais, selon Clark et Chang (1959) dans ce cas on utilisait du chlore combiné pour comparaison, et Hetchey (1953) indique que l'effet antiviral de l'ozone n'est que légèrement supérieur à celui du chlore. Suchkov [5] a montré, à l'exemple de la décroissance de l'oxydabilité au permanganate avec la température, que pendant les cinq premières minutes, l'ozone est utilisé pour détruire les substances organiques facilement oxydables, son effet virulicide n'intervenant qu'après 6–7 mn d'ozonation, au moment de l'apparition d'ozone résiduel dans l'eau. Les chiffres de Suchkov sont analogues aux résultats obtenus par Coin *et al* (1964) que le poliovirus est inactivé dans les 4 mn par une dose d'ozone libre au moins égale à 0,3 mg/l.

Häufele et Sprockhoff [6] (1973) ont montré qu'une bactérie *Candida albinas* et la souche *Penicillium* sont très sensibles à l'ozone et peuvent être inactivées par des doses de 0,5–0,7 mg/l suivant la densité des souches-tests et la durée du contact. Les staphyllocoques exigent des doses de 3–5 mg/l car leur conglomération interfère lors du contact des cellules et de l'ozone. Contrairement aux résultats obtenus par Brigmann, les spores de *B. subtilis*, *B. cerus* et *B. anthracis* se sont montrés particulièrement résistants à l'ozone.

Les spores de *B. globigi* ne perdent leur faculté germinative qu'après 15 mn de contact avec 5 mg/l d'ozone. Il est significatif que diverses souches de moisissures bacillaires exigent différentes doses d'ozone pour leur inactivation. La souche virulente n^0 139 (LD-50 10^{-10}) exige plus d'ozone pour son inactivation que la souche 138 (LDD 50 10^{-6}). Les valeurs limites des doses d'ozone pour inactiver des spores du bacile *Anthrax*, déterminées au laboratoire, ne doivent pas être appliquées dans les installations industrielles. Les doses d'ozone dans les conditions pratiques dépendent de la qualité de l'eau brute, des méthodes de traitement et de mélange de l'air ozoné à l'eau, du temps de contact et des autres conditions. Les valeurs limites pour l'inactivation de trois types de phages, *E. coli* T_1, T_2 et T_3 étaient de 2 mg/l d'ozone. Elles sont suffisantes pour tuer 10^2 à 10^6 unités d'infection d'un virus plaque d'oiseau dans 1 ml. Les doses limites de 2 mg obtenues par Haüfele et Sprockhoff pendant les études sur coxsakies et sur poliovirus sont les mêmes que celles obtenues par d'autres chercheurs.

Bien que des dizaines de villes utilisent actuellement l'ozone et qu'un travail scientifique important ait maintenant été consacré au problème, il n'est pas possible d'analyser objectivement la méthode en raison de la grande diversité des techniques et des tendances de recherches. Il semble que la temps soit venu d'analyser tout le travail réalisé, en utilisant une classification unique, selon les types de bactéries et de virus et les conditions de l'inactivation. Seul un grand centre de recherche disposant de tout l'équipment moderne requis et d'une équipe de virologistes et chimistes qualifiés pourra entreprendre cette tâche. L'organisation d'un tel centre peut être patronée par l'Institut international de l'ozone créé aux Etats-Unis en 1973.

4 Efficacité antivirale de la désinfection

Le problème de la désinfection de l'eau contenant des virus pathogènes résistants aux effets extérieurs n'a pas encore été résolu de façon satisfaisante, mais il est très urgent, car plus d'une centaine de virus très communs sont connus, propagés par la voie fécale et partiellement éliminés seulement par le traitement des eaux d'égout [7]. La présence de ces virus dans l'environnement et, en particulier, dans les eaux naturelles, est une source de danger constant d'infection hydrique apportée jusqu'au consommateur par le réseau de distribution. Il n'est que naturel que le problème de l'élimination des virus de l'eau potable attire de plus en plus l'attention des chercheurs et des spécialistes de l'eau potable.

Le tableau 1, compilé par Clark (1971) et complété pour les années récentes, montre la géographie et les tendances de ce problème.

TABLEAU 1

Auteur et institution	Procédé utilisé	Remarques
Cookson J. Université du Maryland	Adsorption	Cinétique de l'adsorption par le charbon actif
Druary W. A. Université du Tenessee	Coagulation Chloration Adsorption.	Effet de concentration. Adsorption sur charbon et échangeur d'ions
Engelbrecht R. C. M. Chodchuri	Coagulation Diatomite	Couche de polyélectrolyte et ajout de charbon actif
Forster D. H. Université de l'Illinois	Filtration	Recherche de l'adsorption sur gel de silice, oxyde de fer, phosphates complexes
Foliguet J. M. Institut d'hygiène, Nancy, France	Coagulation	Clarificateur à voile de boue
Nupen E. M. National Institute of water examination, Pretoria, Afrique du Sud.	pH	Temps de survie jusqu'à pH 11,2
Olivieri V. P. J. Hopkins univ.	Chloration	Rôle du RNA et détermination de la protéine, cycle vital après destruction
Pointer C. F. B. M.W.B. Labs. Londres, Grande-Bretagne	Filtration	Filtres à sable lent
Shuval H. Hebrew Univ. Jérusalem, Israël	Ozonation	Efficacité de la désinfection et mesure de l'ozone dans l'eau
Sprole O. I.	Ozonation	Cinétique et étude des transports de masse
York D. V. W. A. Druary USA	Coagulation	Coagulation chimique
Lovtsevich E. L. URSS	Chloration Ozonation	Dosage, temps de contact, survie
Kulsky L. A. Globa L. I. URSS	Adhésion Adsorption	Argiles, naturelles

Sans discuter en détail les résultats de ce tableau, on peut les résumer comme suit:

(a) on ne peut obtenir une désinfection de l'eau qu'après une bonne clarification;

(b) l'inactivation non spécifique complète des particules virales est atteinte pour un potentiel redox de 550–600 mV.

D'après les recherches accomplies en laboratoire et en installation pilote, on considère que le chlore et l'ozone sont des désinfectants efficaces, acceptables pour l'utilisation en station de traitement, dont le mécanisme d'action est une dénaturation de l'enveloppe de protéine des virus entéritiques. L'effet anti-viral du chlore libre est environ 50 à 100 fois plus grand que l'effet du chlore combiné. L'effet virulicide est en fait produit par la quantité de chlore libre subsistant après le temps de contact requis. Le tableau 2 donne les résultats de l'inactivation des virus entéritiques par le chlore et l'ozone.

TABLEAU 2

Virus	Effet d'inactivation en %	Temps de contact en mn	Reactif residuel mg/l	Auteur
Inactivation au chlore (Cl_2)				
Hepatite infectieuse	99,93	30	0,3	Chang, 1968, USA
Poliovirus type 1	99,46	60	0,3	Lovtsevich, 1973, URSS
Inactivation à l'ozone (O_3)				
Poliovirus type 1	Complet	30 4	0,4 0,3	Coin 1964, 1967, France
Groupe entéritique	99–99,9	10	0,1	Chang, 1968, USA
Poliovirus type 1	99,99	2,5 5 10	0,23 0,18 0,14	Schaffernot, 1970, USA
Poliovirus type 1	Complet	16	0,4	Ryabchenko, Lovtsevich, 1973, URSS
Poliovirus type 1	Complet	5	1,5	Katsnelson, Kletinger, 1974, USA

Tous ces auteurs sont d'avis que l'efficacité de la désinfection de l'eau par le chlore et l'ozone dépend de la concentration des virus, du degré de pollution par des substances organiques, de la teneur en reactif libre dans l'eau et du temps de contact. Dans des conditions définies, l'effet antiviral de l'ozone est plus fort que celui du chlore. En outre, l'ozone a l'avantage supplémentaire d'améliorer les qualités organoleptiques de l'eau. Cependant, la chloration est plus largement utilisée en pratique dans le monde entier, alors que l'ozonation commence seulement à être appliquée sur une échelle notable dans les nouvelles stations de traitement en construction. Il est difficile de comparer l'effet antiviral du chlore et de l'ozone dans les conditions de l'exploitation en raison du manque d'un système commun d'appréciation même au laboratoire. Si l'eau potable n'est relativement pas assez clarifiée, l'ozone introduit en excès peut être absorbé par l'oxydation chimique de substances organiques plutôt que par la destruction de virus. Mais si l'eau est bien clarifiée par un traitement supplémentaire au charbon actif, l'inactivation des virus peut être obtenue à l'aide d'acide hypochlorique non dissocié sans ozonation additionnelle.

Pour décider de la plus grande efficacité et de la préférence pour le chlore ou l'ozone, il faut mesurer l'efficacité antivirale de ces corps dans les conditions d'exploitation pour différents schémas technologiques de traitement, suivant la qualité de l'eau. Mais une telle mesure est gênée par le fait que le contrôle virologique direct de l'eau potable présente encore beaucoup de difficultés. Il serait essentiel pour une analyse virologique efficace de mettre au point une nouvelle méthode normalisée d'isolation, concentration et détection des virus entéritiques. A cet égard, la mise au point d'une méthode pour la définition quantitative des contaminations virales de l'eau par microscope électronique semble spécialement prometteuse. Comme le contrôle virologique de l'eau potable ainsi que l'évaluation des ressources en eau potable d'après des critères virologiques doivent être réalisées par des spécialistes qualifiés et exigent des équipements coûteux et des dépenses élevées, cette tâche devrait être confiée à centres des virologiques spéciaux.

Malgré les progrès indiscutables dans l'étude de l'élimination des virus lors du traitement de l'eau et l'existence de certains adjuvants assez peu utilisés (comme l'argile et autres adsorbants naturels) il faut toujours se souvenir que le caractère anthropogène de la contamination virale exige la prévention de la pollution des ressources en eau par les eaux d'égout et l'élaboration de mesures pour la destruction des virus aux stations d'épuration des eaux d'égout.

5 Désinfection des conduites neuves et réparées

La désinfection est le dernier stade dans le traitement de l'eau potable. Après désinfection, l'eau acquiert des propriétés normalisées et, après contrôle sanitaire, elle devient le produit marchand de la station de traitment. C'est un produit spécifique, ne serait-ce que par ce qu'elle est fournie au consommateur à tout moment en quantité suffisante par la même station de traitement et par l'intermédiaire d'un réseau de distribution. Habituellement, il n'y a pas d'intermédiaires dans le schéma "réseau—consommateur". En outre, le produit ne doit ni perdre les qualités fixées par la norme, ni en acquérir de nouvelles, qui ne sont pas spécifiées. Et enfin il n'y a pas de moyens techniques de modifier volontairement la qualité de l'eau dans le réseau de distribution. Dans ce cas, on tient compte du chlore résiduel. Donc, la seule chose à faire est de prévenir la contamination bactérienne à l'intérieur du réseau.

Les problèmes de santé publique dans une distribution d'eau impliquent les nombreux facteurs et mesures préventives à prendre en compte. La qualité de l'eau dépend en permanence de la longueur des conduites, du matériau dont elles sont faites, du temps d'exploitation, de la condition physique des conduites, des revêtements protecteurs, du nombre de conduites en cul de sac, des dépôts dans le réseau, de l'entretien convenable et d'autres facteurs. Très important est l'état sanitaire du terrain que traversent les conduites entre la station de traitement et le réseau de distribution: présence de terrains mal consolidés, sites industriels, ravins et cours d'eau pollués. En cas de rupture ou de fuite, du dommage et de la bonne exécution des réparations. Ainsi l'élimination des contaminations bactériennes sera plus rapide et efficace dans les localités où les mesures sanitaires préventives ont été antérieurement bien réalisées le long du trajet des conduites.

La recherche des bactéries dans les conduites a montré l'existence d'une flore bactérienne à leur surface interne qui est couverte d'un film de produits de corrosion, de bactéries ferrugineuses, de sels de calcium et de magnésium, d'aluminium résiduaire, etc. . . . La composition quantitative et qualitative de la flore bactérienne dépend de la composition physique et chimique de l'eau à sa source, de la méthode et du degré de purification et de désinfection à la station de traitement et de la condition des conduites au sens le plus large. Dans les conduites des villes méridionales utilisant des eaux de surface, on peut trouver un zoobenthos composé d'oligochètes, nématodes, chironomides et crustacés. Il est remarquable qu'en ce cas la recherche bactériologique de germes pathogènes donne des résultats négatifs, de sorte que l'attention du personnel n'est pas attirée par des informations impératives. En outre beaucoup d'auteurs soulignent que des virus peuvent être trouvés dans le tractus intestinal de la faune benthique. Dans ces conditions, l'exigence des normes que "l'eau ne doit pas contenir de microorganismes visibles à l'oeil nu" ne garantit pas le réseau contre la contamination microbienne. On a signalé des cas d'algues bleu-vertes trouvés dans les conduites. Cela peut être dû à une tendance à filtrer trop rapidement, jusqu'à 10–12 m/h, et au mauvais état des filtres. En URSS, après l'analyse des résultats obtenus dans des centaines de stations de traitement d'eau de surface, on a conseillé de réduire la vitesse de filtration pour tenir compte de la qualité de l'eau puisée. Le tableau 3 donne le résultat de cette étude.

TABLEAU 3

Type de filtre rapide	Vitesse normale de filtration m/h	
	1962	1974
Couche unique, tailles de grain diverses	6–10	5,5–8
Double couche	10	8–10
Double écoulement	12	10–12

Cependant, malgré les mesures prises pour améliorer le traitement de l'eau par tous les moyens technologiques, l'eau pénétrant dans le réseau peut encore contenir des microorganismes pathogènes. De plus, la pollution bactérienne peut se produire directement dans les conduites en raison de pollution pendant la pose, ou par des joints fuyards, des petits défauts ou des interconnections avec des installations industrielles. La désinfection des conduites neuves est facile et elle se réalise dans la plupart des pays. Les méthodes utilisées sont les mêmes dans beaucoup de pays. Becker [8] écrit que la désinfection des conduites neuves à partir de 300 mm est obligatoire. Lors de travaux de réparation, la nécessité d'une désinfection est appréciée par les responsables. On utilise pour désinfecter les conduites neuves des doses de 100 mg/l de chlore et un temps de contact de 48 h. Après rinçage de la conduite jusqu'à atteindre la teneur en chlore résiduel, on prend deux échantillons à 20 mn d'intervalle en deux points fixes. Les échantillons peuvent être considérés comme représentatifs si on les prend à une bouche d'eau nettoyée en continu par une forte solution de chlore. Ces échantillons, pris après que la bouche d'eau a été fermée pendant 24 h, peuvent donner des résultats erronés. Le chlore résiduel est mesuré par la méthode à l'amidon iodé. Avec une solution 0 1128 N de thiosulfite de sodium et un échantillon de 100 ml 1 ml de solution après titration donnera une concentration en chlore de 40 mg/l. Après les réparations, on doit se fier à un rinçage soigneux des conduites et à la présence de chlore résiduel dans l'eau.

Rossum (Etats-Unis) indique qu'en Californie, avant 1935, on ne désinfectait que les conduites de grand diamètre. Selon les instructions de 1935, toute nouvelle conduite posée doit être traitée par une solution de chlore à 50 mg/l pendant 24 h. Des résultats insuffisants lors de la désinfection furent observés lors de l'adoption des joints mécaniques qui forment un espace mort annulaire entre les tuyaux. Le problème fut résolu lorsqu'apparurent les tablettes de chlore: on en plaça dans l'espace annulaire des joints et on en colla à l'intérieur des tuyaux. Il est alors impossible de nettoyer les conduites par une chasse avant chloration, et il faut donc prendre des précautions accrues pour garder les tuyaux propres pendant la pose. La méthode des tablettes ne peut pas être utilisée pour les travaux de réparation.

Hutchinson [9, 10] parle du problème lié à la croissance de bactéries similaires aux coliformes dans les bourrages de jute des joints constaté en 1940. Plutôt que d'utiliser des superdoses de plusieurs centaines de mg/l de chlore, on résolut ce problème en remplaçant le jute par des matériaux synthétiques. L'emploi de tampons de savon lors des chasses et la mise en service de tablettes d'hypochlorite collées à la partie supérieure des tuyaux ont représenté des progrès dans la désinfection des conduites neuves. L'utilisation de joints à espace mort annulaire peut être la cause d'analyses bactériologiques non satisfaisantes. La flore bactérienne dans cet espace mort n'est pas affectée même par de très fortes doses de chlore qui sont absorbées par le lubrifiant du joint. La seule méthode de contrôle à recommander en ce cas est l'observation de précautions sanitaires et la chloration avant les essais de pression.

En URSS, les instructions en vigueur pour la désinfection des conduites neuves approuvées par le Ministère de la santé publique prescrivent la désinfection de toutes les conduites neuves par des doses de chlore actif allant de 75 à 100 mg/l, avec 50% de cette dose à l'extrémité de la conduite. Le temps de contact minimal est 6 h. Après rinçage, la dose de chlore doit être 0,3–0,5 mg/l. Des analyses satisfaisantes obtenues sur deux échantillons prélevés à 20 mn d'intervalle à chaque point fixe sont une preuve de l'efficacité de la désinfection. Lors des réparations de conduite, les teneurs en chlore actif sont soit 40 mg/l pendant 24 h, soit 75–100 mg/l pendant 6 h. La propreté des conduites de plus de 900 mm de diamètre doit être examinée de l'intérieur par un responsable de l'exploitation. L'observation de ces instructions assure une désinfection satisfaisante et il est rare qu'il faille un second essai. Le rapport de conformité est généralement rédigé en présence d'un représentant du service de santé.

La désinfection après réparation d'une conduite en service est en pratique beaucoup plus compliquée. Il faut veiller spécialement au rinçage et à la désinfection des bouts morts, où l'on peut trouver diverses sortes de contamination. Pendant les rinçages préventifs de 224 bouts morts effectué en 1975 dans des villes russes, on a découvert des nématodes dans trois conduites, malgré des analyses bactériologiques favorables. Le tableau 4 donne les résultats d'analyses d'eau de surface.

TABLEAU 4

Année de construction	Diamétre mm	Longueur m	Analyses			Date
			Bact.	Chim.	Nematodes dans 5 l	
1914	125	110	satisf.	satisf.	8	11,06
			satisf.	satisf.	Aucun	12,06
1934	150	340	satisf.	satisf.	3	25,07
			Indice Coli 14 par l	satisf.	9	25,08
			satisf.	satisf.	Aucun	29,08
1962	200	180	Croissance dense	turbidité 5,6 mg/l	4	8,07
			satisf.	satisf.	Aucun	10,08

Dans tous les cas, on a observé un abondant développement d'organismes benthiques dans les eaux de surface puisées. Les matières en suspension déposées dans les conduites forment un substrat favorable pour leur propagation.

6 Perfectionnement des analyses de contrôle et de laboratoire

Les méthodes d'analyse de laboratoire utilisées de nos jours, comme la méthode iodométrique, la titration ampérométrique l'orthotolidine le leuco cristal violet, le méthyl-orange, la syringaldazine ont des inconvénients communs. Les réactifs utilisés altèrent le pH du milieu et ne donnent pas de déterminations séparables du chlore libre et combiné. Même la méthode courante à l'orthotolidine ne permet pas de différencier le chlore libre et combiné. Cela ne peut être obtenu qu'en refroidissant l'échantillon à 1°C pour assurer la formation des réactions de la chloramine. L'arsénite orthotolidine employé produit une lente réaction de l'arsénite avec la chloramine qui mène à surestimer la valuer vraie du chlore libre. Toutes ces méthodes impliquent une acidification de l'eau qui trouble l'équilibre du système chlore-chloramine. On a donc introduit une étape supplémentaire, l'addition d'orthotolidine neutre (NORT), d'orthotolidine neutre stabilisée (SNORT) et la titration ampérométrique. On a senti alors le besoin de simplifier l'analyse des composés du chlore en réduisant le nombre des réactifs.

Parmi les dernières recherches sur les analyses de contrôle et de laboratoire, les travaux de Palin [11] ont un intérêt spécial. Après de longues recherches, l'auteur a décidé d'utiliser la diéthyl-p-phenylène diamine (DPD) au lieu de la NORT. Après sa première publication dans le JAWWA en 1957, la méthode a été développée et complétée par des procédures pour la détermination des composants de divers mélanges de désinfectants. Un grand progrès à cet égard a été l'emploi de glycine pour la mesure du bioxyde de chlore et des combinaisons de chlore et d'ozone. La glycine se combine rapidement avec le chlore et détruit instantanément l'ozone. La méthode à la DPD sous sa forme actuelle permet de différencier les halogènes, le bioxyde de chlore et l'ozone et peut être également employée en exploitation, car le réactif peut être utilisé sous forme de tablettes et de poudre. A notre avis, la méthode à la DPD permet d'espérer la réalisation d'un appareil automatique pour la mesure du chlore résiduel dans l'eau. L'équipement pourrait comprendre un appareil séparé pour les différentes analyses et être contrôlé par un ordinateur avec un programme logique. La fabrication des divers ensembles dépendra des réactifs utilisés pour le traitement et la désinfection de l'eau. On peut espérer voir cet équipment pour la méthode à la DPD disponible en 1980.

Discussion

Les publications passées en revue, 150 environ pour les trois dernières années, ont beaucoup en commun en ce sens qu'elles ont été écrites par des chimistes analytiques, microbiologistes et virologistes travaillant dans de grands centres de recherche ou laboratoires. Leurs travaux, réalisés à l'aide d'un équipement sophistiqué et coûteux, ont porté principalement sur le mécanisme de la chloration et sur les effets bactéricides et virulicides de divers oxydants. Cette recherche nous donne un meilleur aperçu des phénomènes qui se produisent dans le monde invisible des microbes, mais elle ne nous fournit pas de recommendations pratiques. Pendant des années, la désinfection a été gérée par des exploitants utilisant des chlorateurs automatiques et des analyseurs de chlore résiduel. Les résultats des recherches n'influencent pas l'essence du processus de chloration dans les stations de traitement et les réseaux de distribution. Cette déclaration reflète l'opinion d'un spécialiste de la distribution de l'eau qui espérait obtenir quelque chose à introduire dans la pratique de la chloration, mais n'a réussi qu'à améliorer sa connaissance du sujet. Toutes ces recherches n'ont fait que confirmer le vieil adage que si l'eau est bien traitée et le réseau maintenu propre, l'effet de la désinfection est assuré. Cette conclusion extrême est exprimée délibérément pour attirer l'attention sur l'écart entre les découvertes scientifiques et les limitations des moyens techniques pour contrôler les processus de désinfection. Si l'on parle des progrès accomplis dans la désinfection de l'eau, il faut signaler le plus important, qui est la possibilité de formuler, d'après notre meilleure connaissance des processus de désinfection, nos besoins pour la mise au point future de dispositifs techniques:

(a) mise au point d'appareils automatiques pour le contrôle chimique de la qualité à l'aide d'ordinateurs et pour l'estimation du rendement de la chloration ou de l'ozonation;
(b) mise au point d'appareils automatiques de dosage du chlore ou de l'ozone avec un ordinateur de contrôle.

On ne peut encore rien dire sur les perspectives de création d'un appareil de contrôle bactériologique automatique car toutes les analyses bactériologiques sont basées sur des observations visuelles qui impliquent l'isolement et l'identification de microorganismes, mais nous devons faire ressortir le besoin urgent de méthodes simples, rapides et fiables de contrôle bactérien à utiliser par le personnel des stations de traitement.

Un autre besoin pour l'avenir est l'estimation des propriétés de l'ozone et la détermination de sa place dans le schéme technologique du traitement et de la désinfection de l'eau.

Et finalement il est nécessaire de combiner les recherches dans le domaine de la désinfection des eaux d'égout et de l'eau potable en la considérant comme un processus technologique unique dans la circulation de l'eau dans la nature.

Pour l'avenir, la chloration restera le procédé majeur de désinfection de l'eau. L'extension des connaissances en matière de désinfection de l'eau crée de nouveaux problèmes, qui peuvent être résolus en améliorant la chloration, combinée à l'ozonation.

Remerciements

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Peak load waterworks

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Introduction

In most types of production the producer or supplier has to meet very high demands on certain occasions. This is also valid for a waterworks. The question which is raised in several cases is: "Do we have to make an extra investment, which only will be used occasionally or perhaps not even in five to ten years time"? The producer would prefer to take care of peak demands by "extra" high operation costs or labour costs on such occasions. In normal industrial production it is, in many cases, possible to use 16-hours production instead of 8-hours. In water supply this is not possible since one is normally talking about 24-hour use. Shorter high demands lasting a few hours are taken care of by using watertowers or special reservoirs. In this paper some views and solutions will be given with respect to peak load waterworks, but to start with some definitions and data will be presented.

Peak loads at waterworks

At a waterworks one talks normally about minimum, average and maximum flow in water production. In most cases the maximum flow is equal to peak flow and will also be used in that respect in this paper.

Peak load has to be considered from three points of view.

(a) Peak load on the distribution system, i.e. peak demand from consumers.

(b) Peak load in the treatment plant to cater for the demands of (a).

(c) Peak load on raw water abstraction from rivers and impounding reservoirs which does not necessarily coincide with the times of (a) and (b).

Under (a) the following peaks will mostly be taken care of by reservoirs or other capacity within the distribution system.

— Peaks of consumption within a 24-hour period

— Peak day consumption

Peaks at (b) are associated with longer periods of high demand, e.g. peak week consumption, and at these times the increased demand has to be met from treatment works and/or sources. It can also be a question of peaks over a period of up to three to four weeks. For periods longer than four weeks one should consider a higher average flow and expand the plant.

The amplitude of the peak loads for different countries have normally been shown to be in the same range. For Sweden, France and Great Britain the peak load is 10 to 40 per cent higher than the average flow with an average value of 25–30%. These figures are valid for a one-day peak load. The corresponding value for the one-week peak load is about 10–20 per cent. It should be noted that the peaks will be higher in cases of proportionately less industrial demand and proportionately greater domestic demand.

For the design of waterworks it is not particularly appropriate to discuss the diurnal variations although these are as high as a factor of at least 2 and 3 during the 24-hour period. These fluctuations have to be taken care of within the distribution system by using reservoirs. There is a need for a reservoir capacity of 30–50% of the daily production with larger reservoirs for smaller cities. The peaks which influence the design capacity of the waterworks are those which are due to seasonal effects, e.g. many summer houses or an extremely warm and dry spell of weather lasting a few days in summer time (in Sweden usually June).

Future water consumption

From many years ago there have been prognoses done for the future use of water up to the year 2000. Those have in most cases predicted a rapid rise in water consumption. Figure 1 shows water consumption as it was

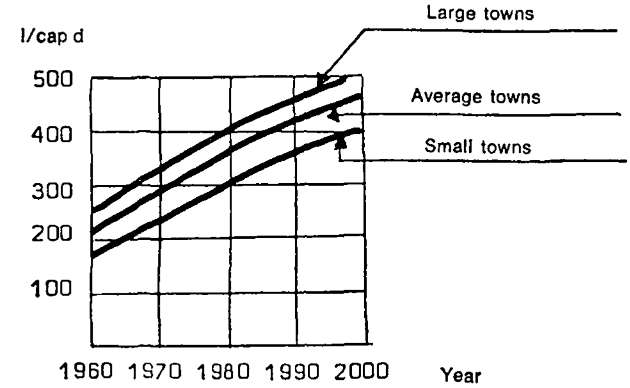

Figure 1—The increase of water consumpiton in Sweden according to a forecast in 1965.

estimated in 1965 by the Swedish authorities. From 1960 to year 2000 the domestic consumption was estimated to increase by a factor of 2 and the industrial consumption by a factor of 3.

It is, however, interesting to note that the actual water consumption has virtually stabilized at a certain level since 1970 and has in some cases even dropped during the last 7–10 years. This is valid for both domestic and industrial consumers. The normal consumption in a household during 1975 was about 205–225 l/cap d (See figure 2).

The reason for the "stagnation" in water consumption may depend on different factors, among others;

— the costs of water have risen sharply over the last few years and are, in Sweden (1976), nearly 400 per cent higher than in 1960, but only about 75% if inflation is accounted for.

— the use of water can only be raised to a certain level, i.e. that at which the "standard" is fulfilled in every household.

The same standstill in water consumption will probably occur or is already present in some other countries in Europe.

The standstill in water consumption will not have any significant effect on the number and heights of the peak loads. It is, however, likely that an expansion of the capacity will be sufficient for a longer time with a lower rise in water consumption.

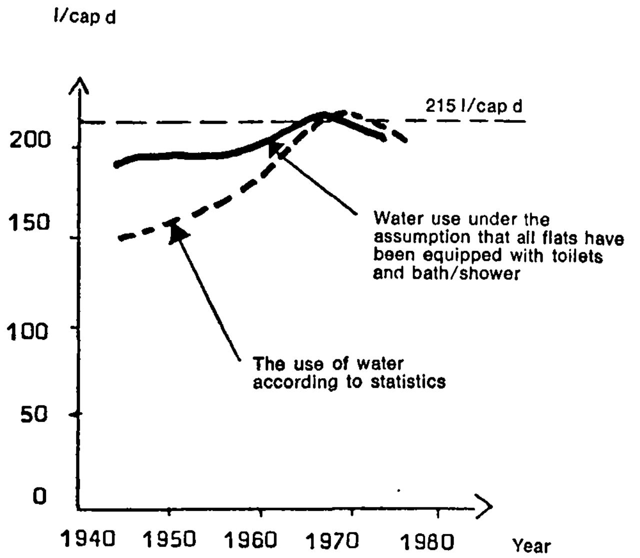

Figure 2—The development of the specific use of water within the household during 1944-1976.

At each expansion of a waterworks the additional capacity will be at least 10 per cent.

This means that the plant will have adequate capacity for between 10 and 20 years after expansion. As shown in figure 3 the maximum capacity can be covered by expansion of the plant to an over-capacity at year X to fulfil the peak load at year X + t. It is also possible to expand with a lower capacity and use the money saved for occasional additional treatment effort if a peak should occur. In figure 3 one can also see that there is a very high overcapacity when the expanded plant is first put into operation.

During the late 1960's the situation in Stockholm was that the capacity of the waterworks was starting to become insufficient. It was decided, therefore, to expand one plant in order to take care of the expected increase of consumption as well as the peak loads (up to about 35% above average flow during a few days per year). The plant, which needed an investment of 100 million Swedish Crowns, was put into operation in 1971. Because of lower water consumption than expected from the prognosis the plant has only been used occasionally. If one could find a technical solution for taking care of the peak loads by "extra" running costs one could in the above mentioned case save up to ten million SCr per year.

If one assumes normal expansion for water consumption of 2–4 per cent per year one will find that the "economic" expansion interval of the plant will be between 10 and 15 years. Longer expansion intervals will give higher costs for the investment compared with shorter expansion periods.

Without occasional additional treatment effort the expansion to Q_{max} (see fig. 3) will need an investment of K[SCr/m³/d]. The total annual cost (K_{year}) will then be,

$$K_{year} = K/\alpha + K_{kem} \times 365 \times \bar{Q} \text{ [SCr/year]}$$

where

α	= loan charges + maintenance + supervision
K/α	= "fixed" costs
K_{kem}	= costs for chemicals and electricity [SCr/m³]
$K_{kem} \times 365 \times \bar{Q}$	= "variable" costs [SCr/year]
$\bar{Q}$	= average consumption [m³/d]

$$\text{Treatment cost} = \frac{K/\alpha}{\bar{Q} \times 365} + K_{kem} \text{ [SCr/m}^3\text{]}$$

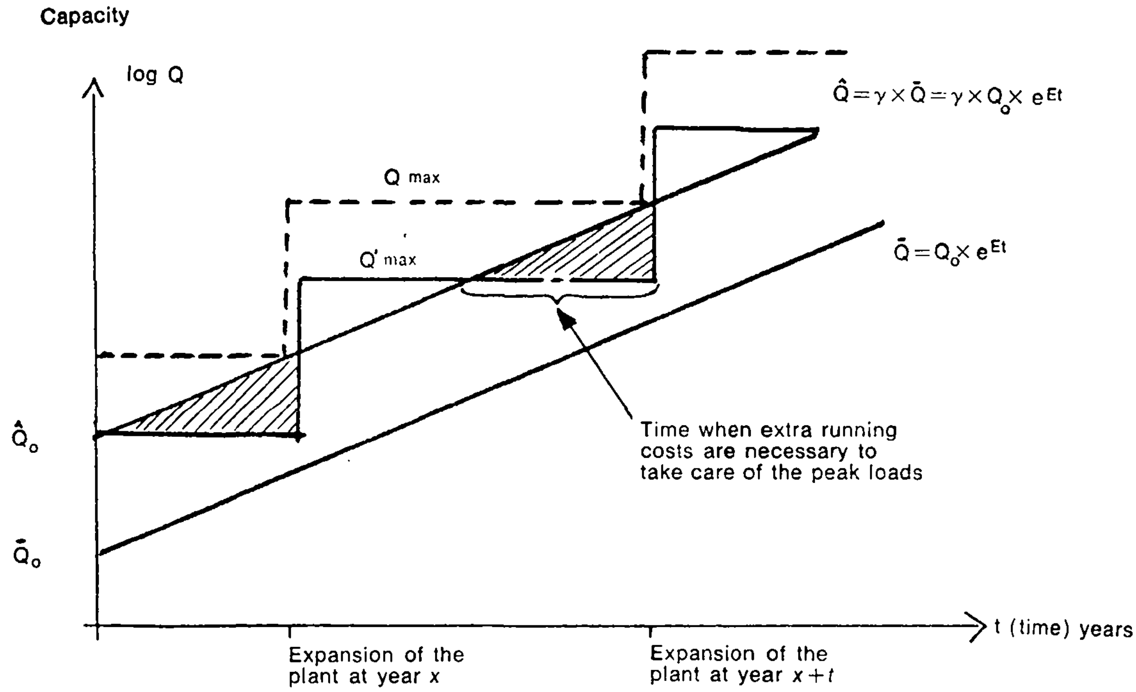

Figure 3—Capacity as a function of the expansion of the plant with or without occasional additional effort.

Where:

$\bar{Q}$	= average consumption [m³/d]
$\hat{Q}$	= maximum consumption (peak flow) [m³/d]
Q_{max}	= maximum capacity without occasional additional efforts [m³/d]
Q'_{max}	= maximum capacity with occasional additional efforts [m³/d]
γ	= peak amplitude $\hat{Q}/\bar{Q}$

With occasional additional treatment effort the corresponding investment will be lower and some "extra" running costs have to be added. The "extra" running costs will only occur at a certain time of the year and will increase according to the rise in water consumption. The plant must, however, be expanded so the extra effort can meet peak flow demand $\hat{Q}$.

The costs have to be integrated during an expansion period of time and be compared with the corresponding costs without temporary efforts which can be expected. When the curves cut each other it could be time for an expansion of the plant after a thorough examination of the costs and expected increase in water consumption.

Alternatives for supplying the demand for water during peak load

There are many different solutions available for taking care of a peak load. The main alternatives are as follows:

(1) Reduction of the demand (e.g. by increasing the price, by prohibiting irrigation at certain times or by installation of separate irrigation water systems for large consumers such as e.g. golf-links).

(2) Installation of reservoirs for treated (or partially treated) water or in some cases by using special water supply sources.

(3) Design of the treatment stages with such over-capacity that peak-load flow can be produced within the existing water plants.

(4) Installation of a separate "peak-load plant" that only operates during peak load and as a stand-by unit.

For water sources where the maximum flow is already exploited, only alternatives 1 or 2 can be used.

Alternative 1 is of course the cheapest measure for the waterworks but normally it does not solve the problem completely. Thus it has to be used in combination with other measures. The effect of prohibition of irrigation is very much dependent upon the loyalty of the consumers. This loyalty is normally too weak when expensive fruit-trees and beautiful flowers and lawns are fading. One possibility would be to distribute irrigation water in tanks if the consumers were willing to pay a high cost.

Pricing policies do seem to have an effect on water consumption. In Sweden, water consumption stagnated or even decreased during recent years as shown above in fig. 2. The situation now is that the cost of water is no longer subsidised but that the entire cost of water and wastewater treatment is charged on households and industries. The cost of water in Sweden is normally about 3 Swedish Crowns/m³ ($0,7 m³) which includes treatment and distribution of drinking water as well as collection and treatment of wastewater. With this price policy unnecessary consumption of water is reduced. Due to the stagnation of water consumption the expansion of water plants have often been postponed.

Alternative 2 is attractive only when natural water reservoirs are available since the volumes have to be very large. A Swedish city of 100 000 persons normally uses about 40 000 m³ drinking water per day. If the peak load is assumed to be 50 000 m³/d (i.e. 25% peak amplitude) during 7 consecutive days a storage volume of 70 000 m³ is needed.

The city of Stockholm uses this alternative in a somewhat modified way. The lake Bornsjön contains water of such high quality that it can be used as drinking water after only filtration and chlorination. This water source is mainly used during peak load and as a stand-by since the annual abstraction is limited.

Other normal sources include groundwater sources which can be used easily to satisfy even very high peak loads.

Alternative 3 is probably the alternative that is most often used in Sweden. However, it gives a significant increase in the investment cost since the plant has to be designed for the peak flow rather than the average flow. A surface water plant normally comprises pH-correction, coagulation with aluminium sulphate, sedimentation, sand filtration, chlorination and pH-correction.

The investment cost for this type of plant is normally about 800 Swedish Crowns/m³/d ($175/m³/d) at a capacity of 10 000 m³/d and 450 Swedish Crowns/m³/d ($100/m³/d) at a capacity of 100 000 m³/d. The operating costs (chemicals, labour etc.) will be about 0,10 Swedish Crowns/m³ ($0,02/m³). If a plant is designed for a peak load amplitude of 25% and this capacity is used during 14 days per year the calculated cost will be as shown in table 1.

TABLE 1
CALCULATED COSTS OF PEAK LOAD WATER PRODUCED IN A CONVENTIONAL PLANT

Cost Breakdown		Design Capacity	
		10 000 m³/d	100 000 m³/d
Peak load amplitude (25% of average flow)	(m³/d)	2 500	25 000
Investment cost for the peak load capacity	(Sw.Cr.)	2×10^6	$11,3 \times 10^6$
Volume of peak load water produced (14 days/year)	(m³/year)	35 000	350 000
Capital costs (15% annual interest rate)	(Sw.Cr./year)	300 000	$1,7 \times 10^6$
Operating costs	(Sw.Cr./year)	4 000	40 000
Annual cost of peak load water	(Sw.Cr./year)	~300 000	$\sim 1,7 \times 10^6$
Cost of peak load water	(Sw.Cr./m³)	8,6	4,9

Alternative 4 will be attractive if a separate "peak load water plant" can produce water at a significantly lower cost than a conventional treatment plant. The characteristics desired for a peak load plant would be:

(a) low investment cost since the capital cost is the dominating item at short operating times

(b) short start-up time; simple operation

(c) compactness—if the plant can be included in existing water treatment plants this would be the most practical solution since the costs of process building and external piping are often essential.

(d) reasonably good water quality. Water of somewhat lower quality than standard can be tolerated for pH, colour, taste and turbidity since it is used only temporarily. Furthermore the water is diluted with water that has undergone complete treatment and the blended water may still meet the required standards. The bacteriological standards should be satisfied before dilution. Quality standards vary in different countries, but are in most cases based on WHO standards. Each country should look at its own standards to see if there is any scope for using water treated to lesser standards at peak load times.

The construction of a peak load water plant does of course depend on the quality of the raw water. For well water, pumping and chlorination will often be sufficient while additional removal of suspended and colloidal solids will often be necessary for surface waters. In conventional surface water plants this is done by coagulation followed by sedimentation and sand filtration. For a peak load water plant, however, a much more compact and simple process would be desirable since it is important to minimize the investment costs.

Treatment of surface water by diatomite filtration and chlorination may be an interesting process for production of peak load water. This process is extremely compact—the retention time is normally only about one minute in the filter. Thus it is often possible to erect a diatomite filter within the existing water plant. Furthermore the investment cost is much less than for a conventional plant since the coagulation and sedimentation stages can be omitted. The purchase cost of a 7000 m^3/d diatomite filter is about 300 000 Sw.Cr. ($70 000) [1]. A peak load plant for 1100 m^3/d using diatomite filtration (filter area 28,8 m^2) and chlorination of surface water has been built in Liverpool, England. The cycle time is about 6 h and the consumption of diatomite about 80 g/m^3 [1].

The cost of an installed diatomite filter plant comprising automatic filters and slurry tank for diatomite will be about 50% of the cost of a conventional plant of the same size [2]. Diatomite filtration instead of sand filtration is seldom used in Europe due to the higher operating costs (mainly the cost of diatomite and pumping) but is considered to be competitive in the USA where diatomite is cheaper. For peak load applications the higher operating costs for diatomite filtration will not be important because of the short operating time. A cost analysis for a separate peak load plant using diatomite filtration and chlorination is shown in table 2.

TABLE 2

CALCULATED COSTS OF PEAK LOAD WATER PRODUCED IN A DIATOMITE FILTRATION PLANT

Cost Breakdown		Design Capacity	
		10 000 m^3/d	100 000 m^3/d
Peak load amplitude 25% of average flow)	(m^3/d)	2 500	25 000
Investment cost for the peak load capacity	(Sw.Cr.)	200 000	1×10^6
Volume of peak load water produced	(m^3)	35 000	350 000
Capital costs	(Sw.Cr./year)	30 000	150 000
Operating costs	(Sw.Cr./year)	10 000	70 000
Total annual cost	(Sw.Cr./year)	40 000	220 000
Total cost of peak load water	(Sw.Cr./m^3)	1,2	0,6

Diatomite filtration normally gives good removal of turbidity but less effective removal of colour and taste. One way of improving the product quality with respect to these paramenters is to use pulverized activated carbon as a filter aid instead of, or in combination with, diatomite. This process is used in Marmora, Ontario, USA, where a 1140 m^3/d plant with carbon and diatomite addition, diatomite filtration (18,6 m^2 filter area) and chlorination was built in 1961 at a cost of $42 000 [4].

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Ouvrages de pointe

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Introduction

Dans la plupart des types de production, le producteur ou livreur doit satisfaire des demandes très importantes en certaines occasions. Cela vaut aussi pour un service d'eau. La question qui en certains cas se pose est la suivante: "Devons-nous faire un investissement supplémentaire qui ne sera utilisé qu'occasionnellement et peut-être même pas du tout pendant cinq à dix ans?". Le producteur aimerait mieux couvrir les demandes de pointe par des coûts élevés "extraordinaires" d'exploitation ou de main-d'oeuvre dans ces occasions. En production industrielle normale, il est souvent possible de travailler 16 heures au lieu de huit. En distribution d'eau, cela n'est pas possible car on parle normalement d'utilisation sur 24 h. Les demandes élevées qui durent quelques heures sont couvertes grâce à des châteaux d'eau ou des réservoirs spéciaux. Dans le présent rapport, nous donnerons quelques idées et solutions concernant les ouvrages de distribution d'eau de pointe, mais pour commencer nous présenterons quelques définitions et données.

Charge de pointe des distributions d'eau

Dans un service d'eau, on parle généralement de débit minimal, moyen et maximal de production de l'eau. Dans la plupart des cas, le débit maximal est égal au débit de pointe; il sera donc utilisé ici de cette façon.

La charge de pointe doit être considérée de trois points de vue:

(a) — pointe pour le réseau de distribution, c'est-à-dire pointe de demande des consommateurs.

(b) — pointe à la station de traitement pour couvrir la demande de (a).

(c) — pointe de puisage d'eau brute dans les rivières, barrages réservoirs qui ne coïncide pas forcément dans le temps avec (a) et (b).

Sous (a), les pointes suivantes sont généralement prises en charge par des réservoirs ou autres capacités à l'intérieur du réseau de distribution:

- pointes de consommation pendant une période de 24 h;
- pointe de consommation journalière.

Les pointes (b) sont associées à des périodes plus longues de demande élevée, par ex. pointes de consommation hebdomadaires. A ces époques, la demande accrue doit être couverte par les ouvrages de traitement et/ou les sources. Il peut être aussi question de pointes durant trois à quatre semaines. Il faut noter qu'après cela il faut étudier le relèvement du débit moyen et agrandir la station.

L'amplitude des pointes de charge pour différents pays se révèle normalement du même ordre de grandeur. Pour la Suède, la France et la Grande-Bretagne, la pointe est supérieure de 10 à 40% au débit moyen avec une valeur moyenne de 25–30%. Ces chiffres valent pour la pointe journalière. Les valeurs correspondantes pour la pointe hebdomadaire sont environ 10 à 20%. On peut noter que les pointes seront plus fortes s'il y a moins d'industrie et plus d'habitations individuelles.

Pour l'étude d'une distribution d'eau, il est sans intérêt de discuter des variations journalières, car celles-ci peuvent comporter un facteur 2 ou 3 au moins pendant la période de 24 h. Ces fluctuations doivent être prises en charge par des réservoirs à l'intérieur du réseau. Il faut 30 à 50% de la capacité de production journalière et des réservoirs plus importants pour les villes plus petites. Les pointes qui influencent la capacité d'étude des distributions d'eau sont celles dues à des effets saisonniers, par ex. nombreuses résidences de vacances ou temps extrêmement chaud et sec pendant quelques jours en été (en Suède, habituellement juin).

Consommation en eau future

Depuis de nombreuses années, on a fait des prévisions pour la consommation en eau future jusqu'à l'an 2000. Ces prévisions montrent la plupart du temps une augmentation rapide de la consommation. La fig 1 montre

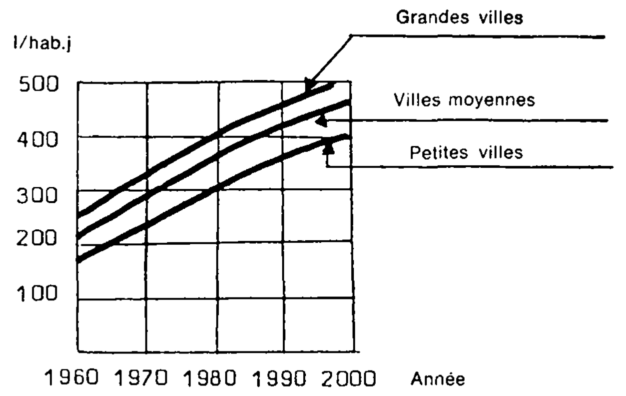

Figure 1—Augmentation de la consommation en eau en Suède selon les prévisions faites en 1965.

la consommation qui était estimée en 1965 par les autorités suédoises. De 1960 à 2000, la consommation domestique devait doubler et la consommation industrielle tripler.

Il est cependant intéressant de noter que la consommation réelle s'est stabilisée à un certain niveau depuis 1970 et qu'en certains cas elle a même diminué depuis 7 à 10 ans. Cela vaut pour les consommations domestiques et industrielles. La consommation normale d'un foyer en 1975 était d'environ 205–225 l/j.hab. (voir fig. 2).

La raison de cette "stagnation" de la consommation en eau incombe à différentes causes, et notamment:

- le prix de l'eau s'est beaucoup élevé ces dernières années. En Suède il est en 1976 4 fois plus élevé qu'en 1960, mais seulement 0,75 fois si l'on tient compte de l'inflation.
- l'utilisation de l'eau ne s'élève qu'à un certain niveau quand un certain standard est atteint dans chaque foyer.

La même stagnation de la consommation en eau surviendra probablement ou est déjà atteinte dans certains autres pays européens. Cette stagnation n'aura guère d'effet significatif sur le nombre et l'importance des pointes de charge. Il est cependant probable que l'extension de la capacité sera suffisante pendant plus longtemps si la consommation augmente plus lentement.

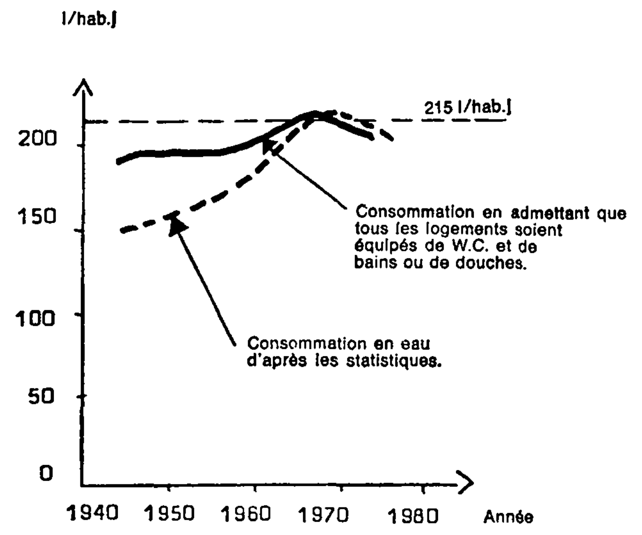

Figure 2—Développement des consommations spécifiques en eau dans les logements de 1944 à 1974.

A chaque extension d'un service d'eau, la capacité supplémentaire sera au moins de 10%. Cela signifie que la station aura une capacité suffisante pour 10 à 20 ans après l'extension. Comme le montre la fig. 3, la capacité maximale peut être couverte par une extension de la station à une sur-capacité en l'an x pour couvrir la pointe de charge en l'an $x + t$. Il est également possible de limiter l'extension à une capacité moindre et d'utiliser l'argent économisé pour des efforts occasionnels quand une pointe se produira. La fig. 3 montre qu'il y a une très forte sur-capacité quand l'extension de la station est mise en route.

A la fin des années 60, à Stockholm, la capacité du service d'eau commençait à être insuffisante. Il fut donc décidé d'agrandir une usine pour couvrir l'augmentation de demande prévue en même temps que les pointes de charge, jusqu'à environ 35% au-dessus du débit moyen pendant quelques jours par an. L'usine, qui a coûté 100 millions de couronnes suédoises, a été mise en service en 1971. Du fait que la consommation avait augmenté moins vite que prévu, cette usine ne fonctionne que temporairement. Si l'on avait pu trouver une solution technique pour prendre en charge les pointes de charge grâce à des frais d'exploitation extraordinaires, on aurait pu dans le cas présent économiser jusqu'à 10 millions de SKr par an.

Si l'on admet que l'augmentation de la consommation en eau est normalement de 2 à 4% par an, on trouve que l'intervalle "économique" pour les agrandissements des ouvrages est compris entre 10 et 15 ans. Des intervalles d'extension plus longs amèneront des dépenses d'investissement plus élevées par rapport aux intervalles plus courts.

En négligeant les efforts exceptionnels, l'extension à Q_{max} demandera un investissement de K [SKr/m³.j]. La dépense annuelle totale [K_{an}] sera alors

$$K_{an} = K \cdot \alpha + K_{kem} \cdot 365 \cdot \bar{Q}$$

où

α = annuité + entretien + gestion.

$K \cdot \alpha$ = coûts fixes.

K_{kem} = coûts des produits chimiques et de l'électricité [SKr/m³]

$K_{kem} \cdot 365 \cdot \bar{Q}$ = coûts variables [SKr/an]

Le coût du traitement est [$K \cdot \alpha/\bar{Q} \cdot 365$] + K_{kem} [SKr/m³].

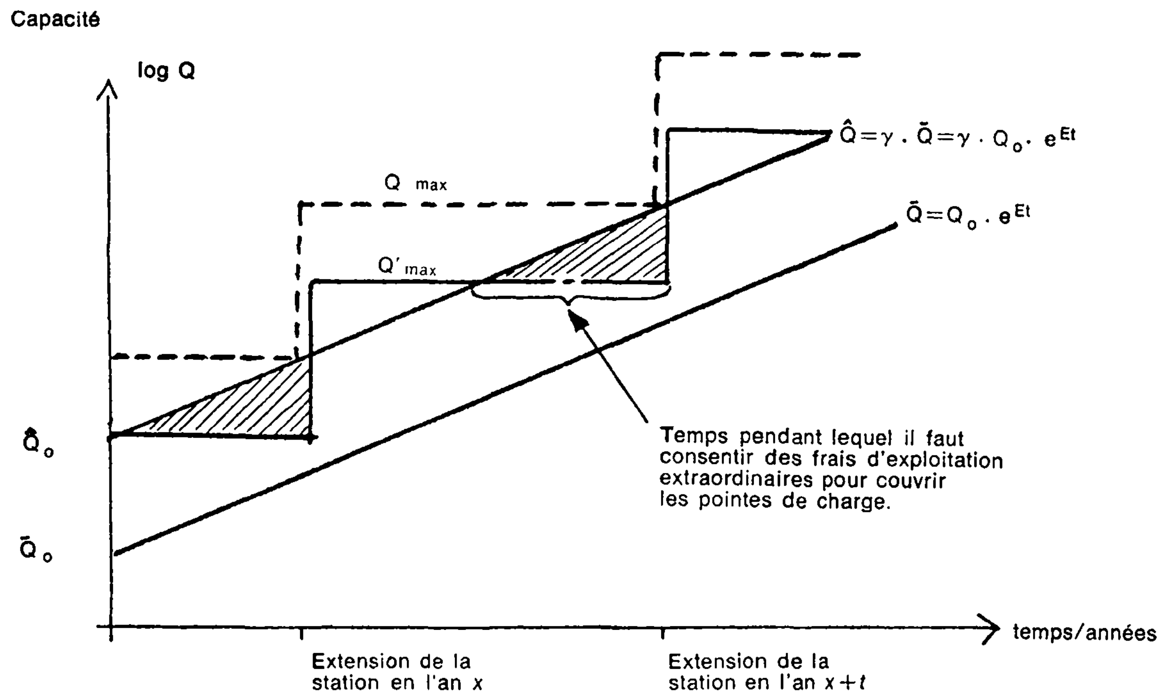

Figure 3—Capacité en function de l'expansion de la station avec ou sans efforts exceptionnels.

$\bar{Q}$ = Consommation moyenne (m³/j).

$\hat{Q}$ = Consommation maximale (débit de pointe) (m³/j)

Q_{max} = Capacité maximale sans efforts occasionnels (m³/j)

γ = Amplitude de la pointe $\hat{Q}/\bar{Q}$

Q'_{max} = Capacité maximale avec des efforts occasionnels (m³/j).

Si l'on accepte quelques efforts occasionnels, les investissements correspondants seront plus faibles, mais il faudra ajouter quelques coûts d'exploitation extraordinaires. Ces dépenses extraordinaires ne se produiront que pendant un certain temps chaque année, temps qui augmentera avec l'élévation de la consommation. Mais la station doit être agrandie de façon que les efforts occasionnels puissent couvrir le débit de pointe ($\hat{Q}$).

Les coûts doivent être intégrés pendant une période d'extension et être comparés avec les coûts correspondants sans les efforts occasionnels que l'on peut prévoir. Quand les courbes se coupent, il peut être temps d'agrandir la station après un examen attentif des coûts et de l'augmentation prévue de la consommation en eau.

Alternatives pour couvrir les besoins en eau pendant les pointes

Il y a de nombreuses solutions possibles pour couvrir les pointes de charge. Les principales alternatives sont:

(1) — Réduction de la demande, par ex., en augmentant les prix, en interdisant l'arrosage à certains moments ou en installant un réseau d'arrosage séparé pour les grands consommateurs comme les golfs.

(2) — Installation de réservoir d'eau traitée (ou partiellement traitée) et en certains cas utilisation de captages spéciaux.

(3) — Conception des étapes de traitement avec une sur-capacité telle que le débit de pointe pourra être produit dans les stations existantes.

(4) — Installation d'une station de pointe séparée qui ne fonctionnera que pendant les périodes de pointe et en secours.

Pour les captages dont le débit maximal est déjà exploité, seules les solutions 1 ou 2 peuvent être utilisées.

L'alternative 1 est naturellement la plus économique pour le service d'eau mais normalement elle ne résoud pas complètement le problème. Il faut donc la combiner avec d'autres mesures. L'effet d'une interdiction d'arrosage dépend beaucoup de l'esprit civique des consommateurs. Cet esprit civique est normalement trop faible quand de coûteux arbres fruitiers, de magnifiques fleurs ou des pelouses s'étiolent. Une possibilité serait de distribuer l'eau d'irrigation dans des citernes quand les consommateurs seraient prêts à payer un prix élevé.

La politique des prix semble avoir effet sur la consommation en eau. En Suède, comme nous l'avons vu ci-dessus, la consommation a stagné ou même décru ces dernières années. En effet, l'eau n'est plus subventionnée, et le coût total de l'eau et du traitement des eaux usées est facturé aux consommateurs domestiques et industriels. Le prix de l'eau en Suède est normalement 3 SKr/m^3 (0,7 US dollar/m^3) ce qui comprend le traitement et la distribution de l'eau potable ainsi que l'évacuation et le traitement des eaux d'égout. Cette politique de prix réduit les consommations inutiles. Grâce à la stagnation de la consommation en eau, l'extension des usines de traitement a souvent pu être retardé.

L'alternative 2 n'est attrayante que lorsque des réservoirs naturels sont utilisables, car les volumes doivent être très grands. Une ville suédoise de 100 000 âmes utilise normalement environ 40 000 m^3/j. Si l'on estime la pointe de charge à 50 000 m^3/j (amplitude de la pointe, 25 %) pendant 7 jours consécutifs, il faut un volume de réservoir de 70 000 m^3.

La ville de Stockholm utilise cette solution d'une façon légèrement différente. Le lac Bornsjö contient de l'eau d'une telle qualité qu'elle peut être utilisée comme eau potable près une simple filtration et chloration. Cette ressource est généralement utilisée en période de pointe et comme secours car le prélèvement annuel autorisé est limité.

Une autre réserve normale est l'eau souterraine qui peut être facilement utilisée et couvrir même des pointes très élevées.

L'alternative 3 est probablement la plus utilisée en Suède. Mais elle aboutit à une augmentation significative des dépenses de premier établissement puisque la station doit être conçue pour le débit de pointe plutôt que pour le débit moyen. Une station de traitement d'eau de surface comprend normalement correction du pH, coagulation au sulfate d'alumine, décantation, filtration sur sable, chloration et correction du pH.

Le coût de premier établissement d'une telle station est normalement de 800 SKr/m^3.j (175 US dollars/m^3.j) pour 10 000 m^3/j et 450 SKr/m^3.j (100 US dollars/m^3.j) pour 100 000 m^3/j. Les frais d'exploitation (produits chimiques, main-d'oeuvre, etc. ...) sont environ 0,10 SKr/m^3 (0,02 US dollar/m^3). Si l'usine est conçue pour une pointe de 25 % et si cette capacité est utilisée 14 jours par an, le prix de revient résultera du tableau 1.

TABLEAU 1

CALCUL DU PRIX DE REVIENT DE L'EAU PRODUITE EN POINTE DANS UNE STATION CLASSIQUE

Capacité nominale		10 000 m^3/j	100 000 m^3/j
Amplitude de la pointe	(m^3/j)	2 500	25 000
Premier établissement pour la capacité de pointe	(SKr)	2 . 10^6	11,3 . 10^6
Volume d'eau de pointe produit (14 jours par an)	(m^3/an)	35 000	350 000
Coûts du capital (15% d'annuité)	(S.Kr/an)	300 000	1,7 . 10^6
Coûts d'exploitation	(S.Kr/an)	4 000	40 000
Coût annuel de l'eau de pointe	(S.Kr/an)	300 000	1,7 . 10^6
Coût de l'eau de pointe	(S.Kr/m^3)	8,6	4,9

L'alternative 4 sera intéressante si une station de pointe séparée peut produire de l'eau à un coût significativement inférieur à celui d'une station classique. Les caractéristiques souhaitables pour une station de pointe sont:

(a) — faible coût de premier établissement car les dépenses en capital sont le point dominant pour une durée d'exploitation réduite;

(b) — temps de mise en route réduit; exploitation simple;

(c) — compacité: si la station peut être incluse dans les stations de traitement déjà existantes, ce serait la solution la plus pratique car le coût des bâtiments et des conduites extérieures sont souvent essentiels.

(d) — eau de qualité raisonnablement bonne. On peut tolérer une qualité un peu inférieure aux normes pour le pH, la couleur, le goût et la turbidité puisque cette eau n'est utilisée que temporairement. En outre, cette eau est diluée par de l'eau qui a subi un traitement complet et il est possible que le mélange réponde aux normes. Les normes sanitaires) doivent probablement avoir été satisfaites déjà avant la dilution. Les normes de qualité varient d'un pays à l'autre, mais elles sont généralement basées sur les normes de l'O.M.S. Chacun doit se référer à ses propres normes pour voir s'il y a une marge de manoeuvre permettant d'utiliser en période de pointe de l'eau plus sommairement traitée.

La construction d'une station de pointe dépend évidemment de la qualité de l'eau disponible. Pour de l'eau souterraine, pompage et chloration seront souvent suffisants, alors que pour une eau de surface il sera en outre nécessaire d'enlever les matières en suspension ou colloïdales. Dans les stations classiques de traitement d'eau de surface, cela s'obtient par coagulation suivi d'une décantation et d'une filtration. Mais pour un traitement des pointes, il faudrait un procédé plus compact et plus simple car il est important de minimiser les investissements.

Le traitement des eaux de surface par filtration à la diatomite et chloration peut être un procédé intéressant en vue de la production d'eau pour les pointes. Ce procédé est extrêmement compact: le temps de séjour dans le filtre n'est normalement que d'une minute. Il est donc souvent possible d'installer un filtre à diatomite à l'intérieur des bâtiments existants. En outre, les investissements sont bien plus faibles que pour un traitement classique car on peut se dispenser des étapes coagulation et filtration. Le prix d'achat d'un filtre à diatomite de 7 000 m³/j est d'environ 300 000 SKr (70 000 US dollars) (1). Une station de pointes pour 1 100 m³/j traitant par filtration à la diatomite (surface filtrante 28,8 m²) et chloration d'une eau de surface a été construite par le service des eaux de Liverpool. Le cycle de fonctionnement est d'environ 6 h et la consommation de diatomite 80 g/m³ (1).

Le coût d'une installation filtrante à la diatomite comprenant des filtres automatiques et une cuve de préparation de la bouillie de diatomite représente environ 50% du coût d'une installation classique de même capacité (2). On utilise rarement en Europe la filtration à la diatomite en raison de ses coûts d'exploitation élevés (essentiellement achat de la diatomite et pompage), mais elle est considérée comme compétitive aux Etats Unis où la diatomite est meilleur marché. Pour un emploi en période de pointe, son coût d'exploitation plus élevé n'a guère d'importance étant donné la faible durée de ces pointes. Le tableau 2 donne le prix de revient d'une station utilisant pour les pointes filtration à la diatomite et chloration.

La filtration à la diatomite aboutit normalement à une bonne élimination de la turbidité, mais elle est moins efficace pour l'élimination de la couleur et du goût. Une façon d'améliorer la situation en ce qui concerne ces paramètres est d'utiliser du charbon actif en poudre comme adjuvant de filtration en plus de ou en combinaison avec la diatomite. Ce procédé est appliqué à Marmora, Ontario, USA où l'on a construit en 1961, pour 42 000 dollars, une station de 1 140 m³/j employant charbon actif et diatomite, filtration à la diatomite (surface filtrante 18,6 m²) et chloration (4).

TABLEAU 2
CALCUL DU PRIX DE REVIENT DE L'EAU PRODUITE EN POINTE DANS UNE STATION FILTRANTE À LA DIATOMITE

Capacité nominale		10 000 m³/j	100 000 m³/j
Amplitude de la pointe	(m³/j)	2 500	25 000
Premier établissement pour la capacité de pointe	(SKr)	200 000	$1 \cdot 10^6$
Volume d'eau de pointe produit	(m³)	35 000	350 000
Coûts du capital	(S.Kr/an)	30 000	150 000
Coûts d'exploitation	(S.Kr/an)	10 000	70 000
Coût total annuel	(S.Kr/an)	40 000	220 000
Coût total de l'eau de pointe	(S.Kr/m³)	1,2	0,6

Résumé

La pointe de charge est normalement égale au débit maximal des installations. Les variations journalières doivent être compensées par des réservoirs à l'intérieur du réseau de distribution. Les pointes dues à la sécheresse durent habituellement de quelques jours à une semaine et plus; elles déterminent souvent le moment où la capacité des installations doit être augmentée. La hauteur de ces pointes est normalement 20 à 30%.

D'une façon surprenante, la consommation en eau a stagné en Suède ces cinq dernières années. Il en résulte que la capacité des stations existantes couvrira les pointes de charge pendant un temps plus long que prévu. En certains cas, des stations nouvelles ont été construite avec une sur-capacité considérable.

Il y a différentes façons de résoudre les pointes de charge:

- restrictions à l'utilisation de l'eau;
- emploi de réservoirs naturels ou de captages d'eau de bonne qualité en réserve;
- construction des ouvrages avec une sur-capacité;
- installation d'une station de pointe spéciale donnant une eau de qualité un peu inférieure. Cette eau est alors mélangée à l'eau normalement traitée. Ce type de station doit être optimal pour une courte durée de fonctionnement, c'est-à-dire avoir un faible coût d'investissement mais un coût d'exploitation élevé.

Besoins en eau pour l'incendie

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1 Introduction

Toutes les idées suggérées par le Comité Scientifique et Technique pour être incorporées dans le rapport n'ont pu être incluses dans l'espace réduit fixé aux Sujets Spéciaux. D'autre part, ce thème est abordé pour la première fois par l'A.I.D.E. Nous avons donc décidé de centrer cette première communication sur les prises publiques pour l'extinction des incendies utilisées par les services d'incendie. En ce qui concerne la conception du réseau de distribution d'eau, cette partie est considérablement à part du problème de l'alimentation des installations privées contre les incendies. Nous avons également eu tendance à étudier le sujet du point de vue des zones urbaines, car il s'agit du champ spécifique d'action pratique des deux rapporteurs.

Notre but a été de réaliser une synthèse des diverses informations existantes, en essayant d'établir des comparaisons entre les renseignements disponibles pour en déduie des lignes générales et à partir de celles-ci et de notre propre expérience, en tirer des conclusions qui puissent servir d'orientation aux services d'eau.

Nous allons examiner la question sous différents aspects pour en tirer les conclusions successives. Le thème sera subdivisé en: utilisation de l'eau; hydrants et leurs caractéristiques; quantité d'eau nécessaire; situation des prises publiques pour pouvoir l'obtenir et aspects économiques et légaux.

2 Utilisation de l'eau

L'utilisation de l'eau contre les incendies prend des aspects très variés en ce qui concerne les formes et les quantités. Nous avons délimité le champ en établissant trois groupes:

1°. Grands ensembles avec installations privées, manoeuvrées souvent par un personnel spécialement préparé pour la lutte contre les incendies. Ils disposent en général de sources d'eau autonomes ou de stockages en réservoirs. Leurs besoins en eau du réseau public ont été l'objet d'une solution convenant à chaque cas spécifique.

2°. Un grand nombre d'installations privées dans des édifices dont les besoins sont moyens, qui dépendent souvent exclusivement de l'eau du réseau public. Elles ont comme point commun de prétendre effectuer seulement une action initiale avec du personnel non préparé en général. Leurs besoins en eau sont limités, et si le feu prend un certain développement, ceci déclenche l'intervention du service d'incendie.

3°. Les interventions du service d'incendie à partir des bouches d'incendie qui correspondent pour une bonne part à des feux dérivant du groupe précédent qui n'ont pas pu être contrôlés au départ.

Si nous nous limitons aux deux derniers groupes, nous avons analysé la façon d'utiliser l'eau. Nous citerons par ordre quelques lignes générales:

Les installations internes privées contre l'incendie ont pour mission d'essayer d'éteindre le feu au départ sans que l'intervention du service d'incendie soit nécessaire.

Lorsque ce Service a été appelé pour intervenir, il le fait presque toujours en utilisant des prises publiques et en lançant l'eau de l'extérieur. A partir de ce moment-là, l'utilité de l'installation interne est minimale ou nulle.

Le service d'incendie commence par intervenir avec l'eau stockée dans les citernes de ses motopompes en la mettant sous pression. Les engins actuels travaillent à deux niveaux de pression: de l'ordre de 7 et 30 bars. Le premier, avec des lances normales pour obtenir un effet direct de l'eau, en utilisant leur portée et parfois l'effet de choc, et le second pour obtenir du brouillard ou de l'eau pulvérisée, qui cause moins de dégâts et consomme moins d'eau, bien que sa portée soit limitée.

Une fois que l'intervention est commencée, on prépare simultanément l'alimentation des motopompes-citernes, une fois localisées les bouches d'incendie du secteur. Si elles sont rapprochées, les motopompes-citernes sont alimentées directement au moyen de tuyaux d'une longueur maximale de 150 mètres; si la distance est plus grande, des citernes nourrices transportent l'eau jusqu'à la motopompe.

La lutte ainsi organisée, on utilise le nombre d'équipes d'intervention nécessaires. Leur importance numérique diminue à mesure que le feu est maîtrisé.

Une fois terminée la phase active de l'incendie, on retire successivement les motopompes et on maintient par prudence la possibilité d'arrosage direct au moyen de lances à partir des prises publiques, quand la pression du réseau est supérieure à 3 kgf/cm^2.

On peut déduire de la séquence antérieure, en ce qui concerne le service d'eau, quelques conclusions:

2 (a). Les installations privées contre l'incendie dans des édifices à besoins moyens ont une action limitée et ne demandent qu'un débit réduit. Pratiquement, leur emploi n'est pas simultané avec celui des bouches d'incendie publiques.

2 (b). L'alimentation directe des lances à partir des prises publiques n'est pas pratique, ce qui fait que la pression ne constitue pas un facteur essentiel.

2 (c). En ce qui concerne le rendement, le service d'incendies n'a pas besoin que les prises soient très rapprochées. Des distances de 150 mètres au maximum du point d'utilisation sont tolérables.

TABLEAU I RESUME DE DONNEES GENERALES

Type d'information	Pays dont provient l'information						
	HOLLANDE	ETATS UNIS	BELGIQUE	FRANCE	ANGLETERRE	SUISSE	ALLEMAGNE
Principal origine de l'information	Comité de Distribution de l'Institut de Contrôle et de recherche des Services d'Eau (KIWA)- Juin 1974	Directives de la "National Board of fire Underwriters"	Conférence de l'Inspecteur Général du Service d'Incendies. Juin 1968	Circulaire nº 465, du mois de décembre des Ministères de l'Intérieur, Urbanisme et Architecture.	"Manual of British Water Engineering Practice".- Ed. 1969	Prescriptions de prevention du feu-Edition provisoire 1973. Service de Prevention d'Incendies.	
Quantité d'eau nécessarie	Aire de Risque A : 360 m^3/h Aire de Risque B : 270 m^3/h Aire de Risque C : 180 m^3/h Aire de Risque D : 90 m^3/h Aire de Risque E : 90 m^3/h Aire de Risque F : 60 m^3/h	Maximum par incendie Ville de 200 000 hab. : 2 725 m^3/h " " 95 000 hab : 2 040 m^3/h " " 55 000 hab : 1 590 m^3/h " " 33 000 hab : 1 135 m^3/h " " 10 000 hab. : 680 m^3/h " " 1 000 hab : 227 m^3/h	Minimum 50 m^3/h	Minimum 60 m^3/h Dans de risques importants plusieurs fois 60 m^3/h		Grand risque: 216 m^3/h Risque moyen: 108 m^3/h Petit risque: 54 m^3/h	Especial: Plus de 192 m^3/h Maximum: 192 m^3/h Minimum: 24 m^3/h
Caractéristiques des hydrants	Débit: 75 m^3/h Perte de charge: 1 kgf/cm^2 Diamètre d'admission: Il n'est pas indiqué.	AWWA C 502 Débit: 2 x 60 m^3/h Perte de charge: 0,14 kgf/cm^2 Diamètre d'admission: 100 mm	NBN 610 Débit: 75 m^3/h (En sortant par une prise de 70 mm) Perte de charge max.: 1 kgf/cm^2 Diamètre d'admission: 100 mm Norme belge contre incendies Débit: 48 m^3/h	NF S 61-211 Débit: 60 m^3/h Perte de charge max.: 0,1 kgf/cm^2 Diamètre d'admission: 100 mm NF S 61-213 Débit: 120 m^3/h Perte de charge max: 0,1 kgf/cm^2 Diamètre d'admission: 150 mm	B.S. 750:1964 Débit: 123 m^3/h Perte de charge max.: 1,75 kgf/cm^2 Diamètre d'admission: 76 mm	Débit: 70 à 90 m^3/h	
Distance entre hydrants	Elle n'est pas indiquée	100 à 150 m	100 m dans de zones urbaines. 200 m dans de zones rurales.	200 à 300 m	90 m dans de zones urbaines. 140 m dans de zones résidentielles.	80 à 120 m	
Diamètres des tuyaux plus petits	100 et 150 mm	150 et 200 mm	100 et 150 mm	Egal ou supérieur au diamètre d'admission de l'hydrant.			
Durée		4 à 10 heures	2 heures	2 heures		1,5 à 2 heures	

3 Résumé de renseignements pratiques généraux

Les normes ou pratiques en usage dans différents pays en ce qui concerne la capacité, les caractéristiques et la situation des prises publiques pour l'extinction des incendies présentent avec évidence un manque de définition concrète, excepte en ce qui concerne la normalisation des bouches d'incendie. Il ne faut pas s'étonner de ce manque de précision, car nous nous trouvons en présence de besoins imposés par les circonstances, indéfinis et variables, qui doivent être couverts par un réseau de canalisations dont la structure géométrique obéit à des situations urbaines jamais répétitives.

Bien que nous nous soyons limités à quelques pays seulement, il a été difficile de préparer un résumé systématique et vraiment comparatif. Bien que cela nous ait obligé à une simplification excessive, nous avons préparé le TABLEAU I. Il s'agit d'un essai comparatif pour la conception, après une élaboration raisonnée et en profitant d'une expérience vécue, de critères plus généraux et concrets en fonction de besoins essentiellement variables et divers.

4 Bouches d'incendie. Caractéristiques d'essai et de fonctionnement

On utilisera indistinctement et suivant les besoins de chaque agglomération, les bouches "enterrées" sous regards, qui ne dépassent pas du sol (NF S 61-211 et B.S. 750 : 1964), et les poteaux ou colonne qui dépassent du sol (AWWA C 502, NBN 610 et NF S 61-213). Les premiers n'offrent pas d'obstacles aux piétons, sont bon marchés et souffrent peu de pannes; les seconds sont plus visibles, sont chers, tombent plus facilement en panne et sont exposés aux chocs extérieurs. La possibilité toujours plus fréquente que, même sur les trottoirs, un véhicule en stationnement puisse rendre une bouche inutilisable, a fait penser à l'étude d'un modèle plus simple, robuste et bon marché, qui, en dépassant du sol, évite cette possibilité. L'opinion des rapporteurs est que dans les secteurs urbains, les bouches "enterrées" devraient être les plus courantes les poteaux d'incendie étant réservés aux cas spéciaux, comme les zones couvertes de végétation, les cas d'inutilisation possibles par stationnement, etc.

Analysons le comportement hydraulique d'une bouche installée. Chaque Norme établit pour la bouche d'incendie proprement dite l'essai hydraulique auquel elle doit répondre. Ainsi dans la figure 1, les points 1, 2 et 3 correspondent aux Normes indiquées; il s'agit de pertes de charge très basses pour des débits de 60 m^3/h ou 2×60 m^3/h. La Norme NBN 610 paraît admettre une perte élevée, mais elle concerne un poteau d'incendie de deux prises de 70 mm et une de 100 mm, essai au cours duquel une seule des bouches de petit diamètre est maintenue ouverte.

Le comportement réel est différent de celui de l'essai, ceci étant dû aux conditions inévitables d'installation et de fonctionnement. Chaque bouche d'incendie exige un T sur le réseau, une vanne à coin et une section courte de tuyau avec les coudes nécessaires pour placer la bouche dans la position correcte (fig. 2). La perte de charge dans la bouche proprement dite est très petite par rapport à celle de la section d'accouplement et de ses pièces. De plus, la bouche alimente au moyen de tuyaux flexibles, plus ou moins longs, les motopompes-citernes du Service d'Incendie. Dans l'ensemble, entre le tuyau qui est à la pression du réseau et la motopompe du Service d'Incendie, la résistance hydraulique est beaucoup plus grande. En réalité, on obtient des caractéristiques de fonctionnement qui donnent une perte de charge qui correspond au moins à la droite qui passe par le point A de la figure 1 (60 m^3/h et 1 kgf/cm^2 de perte de charge). Le débit réellement donné par une bouche dépendra des conditions d'installation, de la façon réelle de la raccorder à la motopompe-citerne, de la pression disponible dans le réseau et de la structure du réseau de distribution en question.

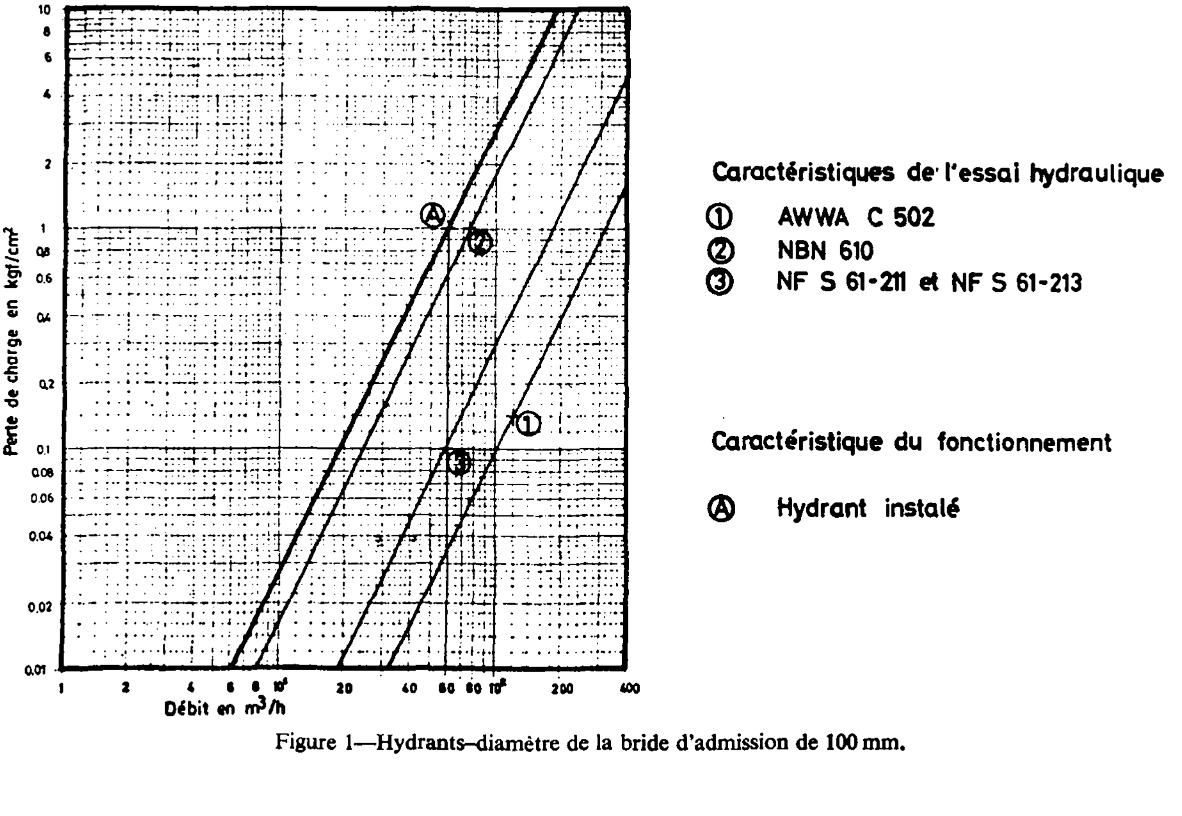

Figure 1—Hydrants–diamètre de la bride d'admission de 100 mm.

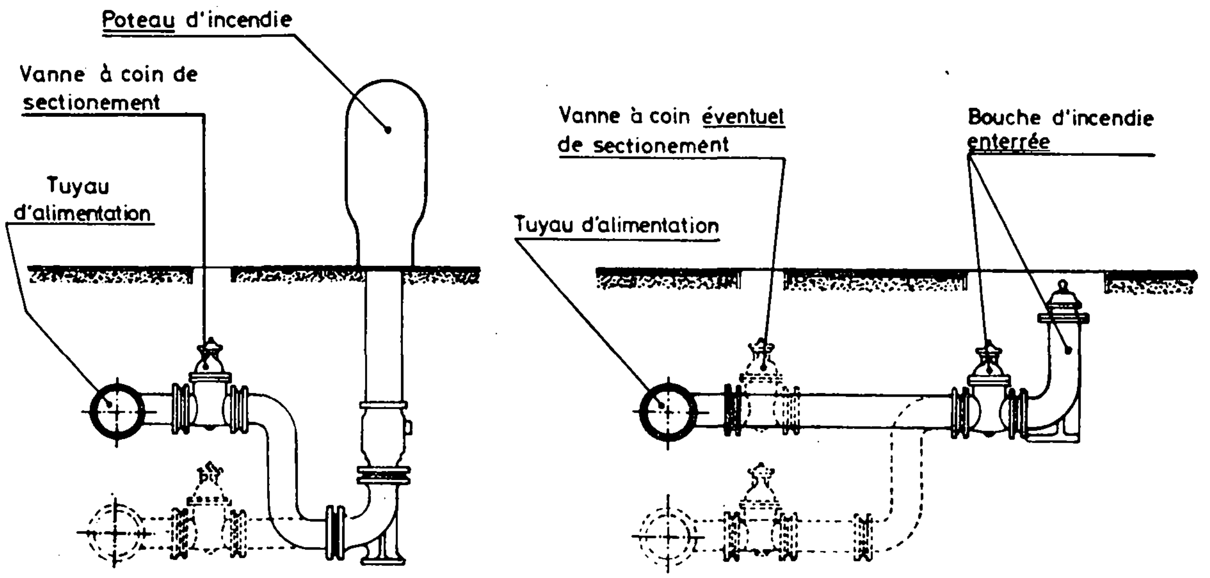

Figure 2—Schéma d'installation des hydrants.

5 Quantité d'eau nécessaire

Dans le TABLEAU I, on observe dans les pays européens une coincidence dans les besoins minimaux qui nous conduit à fixer ces besoins à 60 m³/h par incendie. Ce débit a été estimé récemment par le groupe de travail de KIWA (Hollande) pour le Risque F (minimum). De plus, pour 40 000 interventions en France, il a suffi d'un débit de 50 m³/h pour 90% des interventions. Cette justification du minimum nous paraît suffisante.

75% des interventions durent moins de deux heures, ce qui permet de dire qu'une quantité d'eau de 120 m³ au total est suffisante pour un pourcentage très élevé des incendies. Si la consommation spécifique est basse (100 l/hab. jour) et, avec une capacité limitée des réservoirs (25% de la consommation journalière), en se basant seulement sur un tiers de cette capacité, une agglomération de 15 000 habitants a une réserve suffisante avec ses seules disponibilités d'alimentation en eau potable. C'est seulement pour des agglomérations plus petites qu'il sera nécessaire de prévoir des réserves additionnelles de 120 m³.

Le débit de 60 m³/h coincide d'une part avec le débit habituel de fonctionnement des motopompes de capacité la plus courante des Services d'Incendie et d'autre part avec les débits nominaux qu'on peut atteindre des bouches d'incendie les plus fréquentes, en entendant par cela celles de 100 mm de diamètre à la bride d'admission. Ainsi donc, un incendie qui n'a pas des demandes supérieures au débit minimum peut être éteint avec l'eau fournie par une seule bouche normale.

Nous voyons dans le TABLEAU I quelles sont les distances recommandées entre bouches d'incendie. En prenant 100 m, on se conforme à ce que l'on considère comme normal dans de milieux urbains car le renseignement recueilli en France a un accent rural prononcé et se rapporte à des conditions minimales de protection. Cette distance appliquée régulièrement et de façon idéale, supposerait une disponibilité de 60 m³/h pour chaque 100 m × 100 m = 1 ha de superficie. Il est logique d'admettre, pour des risques maximaux, des distances inférieures pour les bouches d'incendie normales ou des bouches plus grandes (d'un diamètre d'admission de 150 mm) avec la même distance. Ceci peut nous amener à prévoir avec de tels risques 60 m³/h pour chaque 0,5 ha. Ainsi le débit maximum de 360 m³/h que prévoit le groupe de travail KIWA suppose un incendie qui affecte 3 ha de superficie. Par contre, avec le débit maximum de 2 725 m³/h prévu aux Etats-Unis, chaque bouche d'incendie se voit assigner une moyenne de superficie à protéger de 0,37 ha, c'est-à-dire une

TABLEAU II

Risque	Débit par incendie	Moyenne de superficie par hydrant de 60 m³/h	Distance moyenne entre les hydrants
Aire de Risque A	360 m³/h	0,5 ha	70 m
Aire de Risque B	270 m³/h	0,7 ha	80 m
Aire de Risque C	180 m³/h	1 ha	100 m
Aire de Risque D	120 m³/h	1,50 ha	125 m
Aire de Risque E	90 m³/h	2 ha	140 m
Aire de Risque F	60 m³/h	4 ha	200 m

En ce qui concerne le débit maximum, le TABLEAU I offre à première vue de grandes différences (360 ou 2 725 m³/h) qui sont plus apparentes que réelles. En même temps que des besoins de débit, il faut traiter de la superficie qui correspond à chacune des bouches d'incendie qui contribuent à fournir ce débit. C'est ainsi qu'apparaissent les distances entre bouches.

superficie totale affectée de 17 ha. Il n'y a pas une grande différence dans la distribution des bouches d'incendie, mais, par contre, on suppose que la superficie affectée est presque six fois supérieure.

Dans une distribution de bouches d'incendie présentant une certaine régularité, une superficie affectée plus grande ne suppose pas autre chose qu'un nombre plus grand de bouches en fonctionnement simultané. Si la demande en eau par hectare a été réglée moyennant une distance minimum correcte entre bouches, dans le cas du réseau maillé d'une grande ville passent au second plan les préoccupations de l'organisme fournisseur d'eau en ce qui concerne le nombre de bouches en fonctionnement simultané, car en même temps que la superficie augmente, devient disponible un nombre plus grand de canalisations importantes permettant d'assurer la fourniture du débit total demandé.

En résumé, et presque sans les modifier, nous proposons comme débits nécessaires d'eau ceux qui sont indiquées dans la classification du groupe de travail KIWA, complétés par l'indication de la superficie qui doit correspondre approximativement à chaque bouche de 60 m^3/h (TABLEAU II).

Les caractéristiques des aires peuvent être définies comme suit:
Aire de Risque A (risque très élevé): Zones portuaires anciennes à forte densité d'édification. Quartiers commerciaux ou d'affaires caractérisés par des rues étroites et des édifices élevés avec des pâtes de maisons fermés et sans murs coupe-feux. Ensembles industriels avec stockage de produits à pouvoir calorifique élevé, où n'ont pas été prises les mesures nécessaires au point de vue structure et protection contre le feu.
Aire de Risque B (risques élevé): Zones avec magasins fermés ou à l'air libre, où les mesures nécessaires au point de vue structure et protection contre le feu ont été prises. Ensembles industriels anciens situés à côté de quartiers à forte densité d'édification. Quartiers commerciaux et d'affaires caractérisés par des rues étroites et des édifices élevés avec des murs coupe-feux. Zones comportant des risques considérables d'incendie dans des villes anciennes importantes au point de vue historique.
Aire de Risque C (risque considérable): Zones modernes entourées de magasins fermés ou à l'air libre, où les mesures correctes de protection contre le feu ont été prises. Edifices et complexes industriels modernes où ont été prévues les mesures adéquates de sécurité au moment du projet. Petits quartiers industriels. Grands quartiers commerciaux constitués dans leur ensemble par des édifices élevés dont la structure est incombustible. Grands blocs d'appartements oú ont été prévues les mesures adéquates de sécurité contre les incendies.
Aire de Risque D (risque modéré): Edifices dont la structure est incombustible, situés dans des pâtés de maisons fermés avec des rues larges. Edifices résidentiels peu élevés. Commerce de détail et petites industries.
Aire de Risque E (risque bas): Constructions isolées d'un maximum de quatre étages. Zones résidentielles modernes dans des pâtés de maisons ouverts ou fermés avec une occupation de 50% du terrain.
Aire de Risque F (risque minimum): Edifices dans des zones rurales où les risques d'incendies sont peu élevés. Fermes dispersées et zones de loisirs. Quartiers urbains constitués par des édifices unifamiliaux, isolés et peu élevés.

6 Réseau de distribution et diamètres minimaux

Deux exemples d'application du TABLEAU II faciliteront sa compréhension et permettront surtout de tirer des conclusions importantes en ce qui concerne la structure du réseau et les diamètres minimaux. Nous avons choisi deux cas réels de structures urbaines. Pour le premier, nous avons supposé un réseau idéal et pour le second un réseau réel existant déjà et ayant évolué au cours de nombreuses années.

La première application pratique est constituée par une section de réseau qui est très caractéristique de la ville de Barcelone. Elle comporte des édifices d'habitation, de bureaux et de commerces de détail, de bonne construction, ayant plus de 50 ans d'âge, de six à dix étages de haut, les rues ayant environ 20 mètres de large et ne comprenant pratiquement pas de fabriques ou de magasins. Elle constitue nettement une Aire de Risque D (risque modéré) à laquelle correspond une moyenne de 1,5 ha pour chaque bouche d'incendie de 60 m^3/h. La superficie totale étant de 28 ha, il faudrait prévoir 19 bouches d'un diamètre d'admission de 100 mm, distribués d'une façon assez régulière et aux endroits les plus logiques. Les carrefours sont sans aucun doute les endroits les plus favorables pour la manoeuvre des

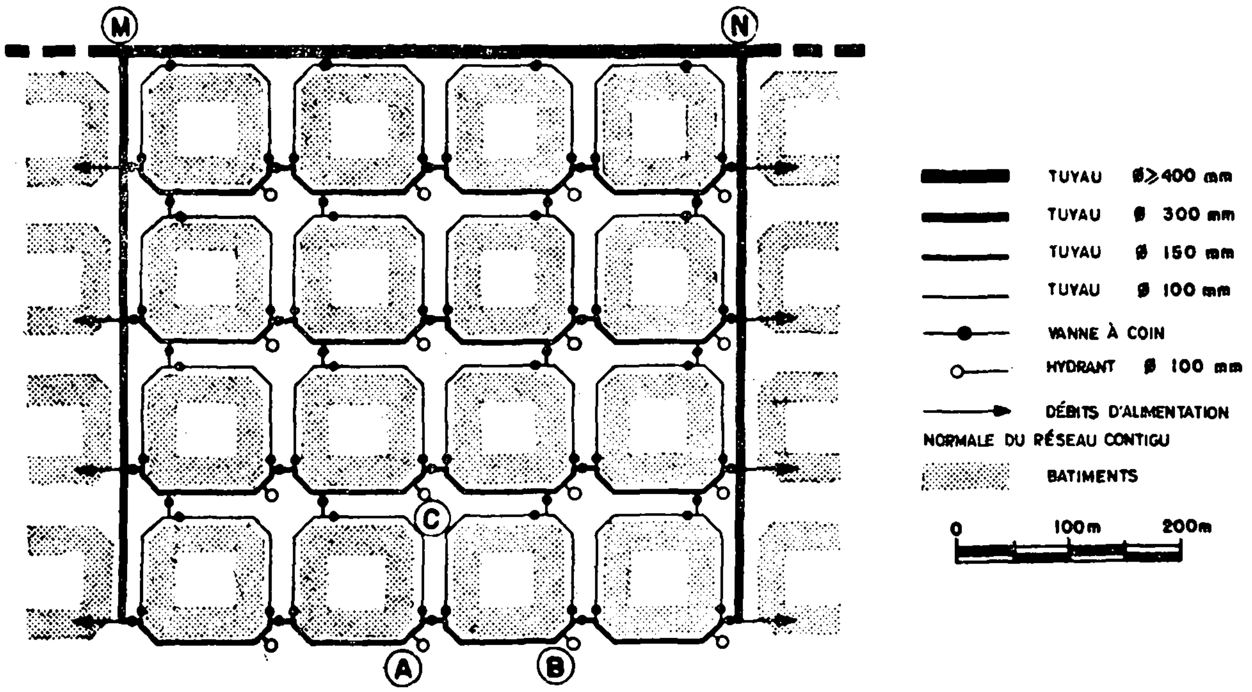

Figure 3.

véhicules et pour l'obtention d'une bonne portée, et coincident de plus avec les noeuds réels du réseau maillé.

Si on place des bouches à tous les carrefours, leur total sera de 16, ce qui correspond réellement à 1,75 ha par bouche c'est-à-dire, une superficie par bouche plus grande que celle qui conviendrait. Cette distribution est celle que reflète la figure 3. La distance entre bouches est en réalité de 133 m, alors que celle que recommande le TABLEAU II est de 125 m.

Il a fallu définir une structure logique du réseau parmi les nombreuses structures que l'on pouvait imaginer. Une conduite primaire de grand diamètre s'étend dans la partie haute de la zone et d'elle partent, tous les 530 m dans le cas présent, des conduites secondaires de 300 mm. Entre deux de celles-ci s'étend le réseau local tertiaire, dérivant tous les 133 m de ces conduites secondaires. C'est un exemple de structure très symétrique qui se rencontre dans la réalité à Barcelone. Ce réseau idéal a été l'objet de sondages au point de vue calcul en utilisant le modèle mathématique digital correspondant et en prenant comme hypothèse l'addition de la consommation pointe d'eau à la prise d'eau sur une deux ou trois bouches d'incendie, en appliquant à ces bouches la caractéristique A de la figure 1. On a pu ainsi vérifier que si le réseau tertiaire est réalisé dans son ensemble avec des tuyaux de 100 mm de diamètre, le débit en cas d'incendie est à peine suffisant, bien que cela fasse descendre la pression du réseau à des valeurs de l'ordre de 1 kgf/cm^2; toute anomalie donnerait lieu à un manque d'eau constituant une situation critique. Il est apparu comme évident que suivrè la règle de 125 mm comme diamètre minimum, c'est-à-dire, utiliser ce diamètre pour tout le réseau tertiaire, serait excessif. Pour cette raison, on a pris comme calcul définitif l'hypothèse de disposer de tuyaux de 125 mm entre les deux conduites secondaires et de réaliser tout le reste en tuyaux de 100 mm en plaçant toujours les bouches d'incendie sur les tuyaux de 125 mm. Cette structure est celle que reflète la figure 3.

Les résultats du calcul définitif sont résumés dans la TABLEAU III. Nous sommes partis d'une pression dans la conduite primaire de 4,5 et 3 kgf/cm^2, car dans le plus grand nombre des cas pratiques les pressions seront situées entre ces deux limites, qu'on suppose constantes dans les points M et N de la figure 3. On a considéré que la zone était située dans son ensemble à la même cote altimétrique.

Il faut observer qu'il s'agit d'un réseau maillé complet, avec un diamètre minimum de 100 mm pour l'alimentation normale et de 125 mm pour les conduites sur lesquelles sont branchés les bouches d'incendie de diamètre d'admission de 100 mm. Ces conduites de 125 mm connectent les conduites secondaires, ce qui fait que les bouches sont toujours alimentées par les deux côtés. Dans ces conditions, les 120 m^3/h nécessaires pour un incendie sont largement obtenus avec deux bouches (174 m^3/h). La distance entre conduites secondaires est importante; nous pouvons affirmer que la distance de 530 m pourrait être portée jusqu'à 800 m sans que se produisent des changements qui pourraient faire que les débits ou les pressions soient insuffisants. Finalement, nous constatons que le fait d'avoir trois bouches fonctionnant simultanément a peu d'importance par rapport au cas où l'on en utilise une seule. Ceci nous confirme que le réseau est suffisamment dimensionné, bien maillé et que la distance entre bouches est correcte.

La seconde application pratique a été réalisée sur une section de réseau réel, pas complètement développée, qui alimente un quartier ancien, de population dense avec des rues étroites dans une zone près du port. Il s'agit d'une Aire de Risque A (risque très élevé) à laquelle, selon le TABLEAU II correspond une moyenne de 0,5 ha pour chaque bouche de 60 m^3/h. La superficie totale étant de 14,7 ha, il faut prévoir 29 bouches d'un diamètre d'admission de 100 mm. En plaçant la plupart d'entre elles aux carrefours et le reste en dehors, pour respecter approximativement la distance de 70 mètres (TABLEAU II) on a réparti 26 bouches la superficie par bouche étant de 0,56 ha et la disposition celle qui est reflétée par la figure 4.

On a pris la configuration réelle du réseau, les bouches étant toujours placés sur des conduites de 150 mm de diamètre, le reste étant de 100 mm ou moins. Dans ce réseau réel, il existe des mailles non fermées et dans certains cas la conduite de 150 mm se termine à l'endroit où est situé la bouche, comme dans le cas des bouches A, B et C. Les conduites secondaires sont situées à 650 mètres approximativement les unes des autres. Comme plus haut, nous avons utilisé le modèle mathématique et nous avons additionné le débit de pointe d'alimentation normal et la prise d'eau des bouches d'incendie, les résultats sont résumés dans le TABLEAU IV les pressions aux points M, N et O de la figure 4

TABLEAU III—(voir fig. 3)

Pression en M et N	4,5 kgf/cm^2		3 kgf/cm^2	
Hydrants en service	Débit en m^3/h	Pression minimale dans le réseau en kgf/cm^2	Débit en m^3/h	Pression minimale dans le réseau en kgf/cm^2
A	119	3,6	94	2,3
A B TOTAL	110 111 221	3,1	87 87 174	1,9
A B C TOTAL	105 105 110 320	2,8	82 83 85 250	1,8

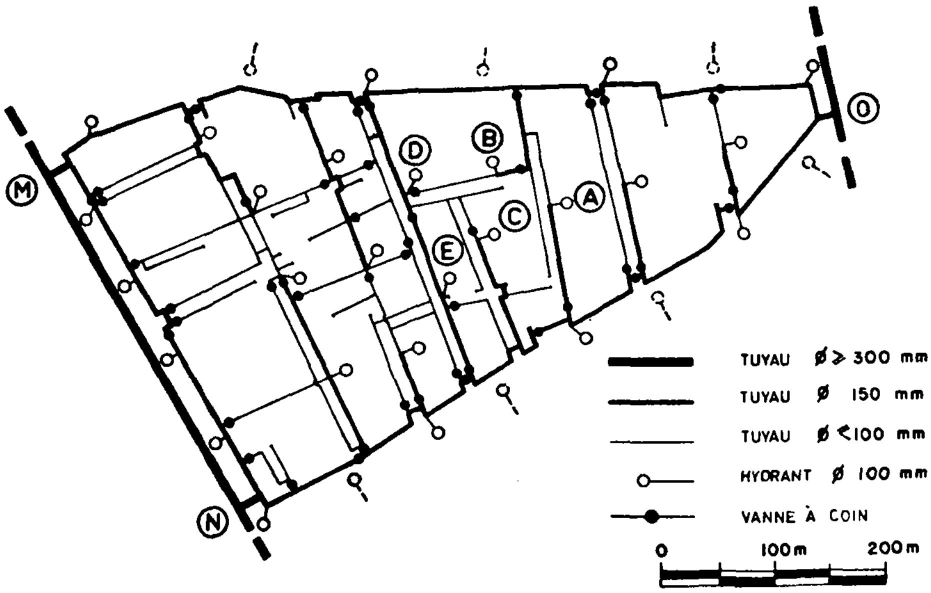

Figure 4.

TABLEAU IV—(voir fig. 4)

Pression en O	4,5 kgf/cm^2		3 kgf/cm^2	
Hidrants en service	Débit en m^3/h	Pression minimale dans le réseau en kgf/cm^2	Débit en m^3/h	Pression minimale dans le réseau en kgf/cm^2
A	120	3,6	97	2,4
A B C TOTAL	107 106 109 322	2,9	86 85 88 259	1,9
A B C D E TOTAL	94 92 94 94 97 471	2,2	75 75 74 74 77 374	1,5

étant supposées constantes. La différence de niveau entre ces trois points est de 3 mètres.

Nous devons attirer l'attention sur le fait que le calcul a démontré qu'avec des conduites de 125 mm, il n'aurait pas été possible d'obtenir les 360 m^3/h avec six bouches d'incendie. Une situation plus décentrée des bouches pourrait être plus défavorable, et c'est pourquoi il est recommandé de fermer les mailles, avec uniquement un peu plus de canalisation de 150 mm. La distance entre conduites secondaires peut à peine dépasser les 650 mètres car la pression minimale atteint déjà des limites non désirables, avec la possibilité de cas de dépressions dans quelques logements élevés. L'influence entre bouches est sensible, car le débit varie de 97 m^3/h quand l'une d'elles est en service, à 74 m^3/h quand on a besoin de cinq bouches pour obtenir le total de 360 m^3/h nécessaire par incendie.

Nous avons ainsi expliqué l'emploi du TABLEAU II et l'on pourrait faire remarquer qu'une vérification semblable dans chaque cas serait la méthode la plus complète. Nous estimons que cela n'est nécessaire que dans des cas spéciaux, car les observations réalisées plus l'interprétation logique des résultats, permettent à notre avis d'arriver aux conclusions d'orientations suivantes:

6 (a). Les débits nécessaires pour la lutte contre les incendies sont ceux qui sont spécifiés dans le TABLEAU II qui est complété par la description de chacune des Aires de Risque.

6 (b). Dans le cas d'un réseau maillé, les conduites secondaires, dont le diamètre est de 300 mm ou plus, ne doivent pas être situées à plus de 650 m les unes des autres pour le Risque A, avec la possibilité d'augmenter progressivement cette distance jusqu'à 800 m pour le Risque D et jusqu'à 1 000 m pour le Risque F.

6 (c). Les bouches d'incendie dont le diamètre d'admission est de 100 mm devront être placés sur des canalisations de 150 mm de diamètre au minimum dans les Risques A, B et C; on peut réduire ce diamètre minimum à 125 mm pour les Risques D, E et F.

6 (d). Les canalisations connectant les conduites secondaires et sur lesquelles sont branchés directement les bouches d'incendie ne doivent comporter aucune partie dont le diamètre soit inférieur au diamètre minimum, ce qui veut dire qu'elles doivent fournir une alimentation correcte par les deux côtés. Le reste des canalisations tertiaires peuvent être d'un diamètre inférieur, quoiqu'il soit très convenable qu'elles ferment des mailles.

6 (e). Les bouches dont le diamètre d'admission est de 150 mm équivalent à deux bouches de 100 mm branchés au même endroit. Il faut cependent limiter ce genre d'installation aux cas spéciaux. Si l'on augmente proportionnellement la superficie assignée, les mêmes conclusions sont valables; si tel n'est pas le cas, il faut tenir compte des effets que peut provoquer une plus grande proximité.

6 (f). Les sections les plus eloignées du réseau ont les plus grandes difficultés à cause de leur structure souvent ramifiée, structure qui arrive aussi fréquemment pour les Risques E et F des réseaux ramifiés. Pour tous les types de risques, un réseau ramifié devra faire l'objet d'un calcul approximatif pour chaque cas particulier, en partant des débits et des superficies du TABLEAU II.

6 (g). En général les réseaux primaire et secondaire seront amplement suffisants pour les débits proposés dans le TABLEAU II. S'il existait un facteur anormal dans leur structure ou dans les risques à prévoir, il faudrait effectuer une vérification adéquate.

7 Observations complémentaires

Parmi les renseignements recueillis, il existe des éléments non quantifiables. En conséquence, nous proposons les conclusions complémentaires suivantes:

7 (a). Dans la pratique général, on accepte des perturbations dans l'alimentation normale en eau pendant un incendie. Dans les incendies normaux (TABLEAU II) les pressions minimales dans le réseau ne devraient pas être inférieures à 1,5 bar pour éviter les retours d'eau au réseau de distribution. Dans des cas exceptionnels, cette pression pourra être inférieure (dans certains questionnaires on admet 0,5 bar) car une telle situation n'est pas plus défavorable ni plus fréquente que la vidange du réseau en cas d'avarie et de réparation.

7 (b). L'interdiction pour les motopompes des Services d'Incendie d'aspirer directement sur les bouches d'incendie devrait être prise comme norme. Cette aspiration pourra avoir lieu dans des cas exceptionnels, étant indispensable d'aspirer à l'aide de tuyaux souples à parois minces qui s'aplatissent lorsque la pression à l'intérieur est inférieure à la pression atmosphérique.

7 (c). Il faut que les Associations Nationales de Services d'Eau interviennent dans la rédaction des règlementations légales relatives à la protection contre les incendies, en prenant soin de tenir compte des critères logiques au point de vue hydraulique pour la définition de la distribution des hydrants et de leur nombre, ainsi que pour l'utilisation de l'eau.

8 Quelques considérations économiques et légales

Les aspects techniques de ces problèmes ont pu faire l'objet d'un traitement complet et on a pu arriver à certaines conclusions. En ce qui concerne l'aspect économique, il faut rechercher la simplicité et l'efficacité, car la valeur absolue n'est proportionnellement pas élevée. L'aspect légal tout au moins en Europe, est très peu défini; il demanderait une vision spécialisée à travers un rapport dédié spécialement à cet aspect. Nous nous limitons donc, en ce qui concerne ces deux points, à quelques considérations générales.

8.1 Aspects économiques

Les débits par hectare prévus dans le TABLEAU II sont très supérieurs à ceux nécessaires pour l'alimentation normale. Par conséquent, projeter les réseaux en prévoyant une protection correcte contre l'incendie, supposera un renchérissement par rapport à un réseau strictement calculé pour l'alimentation normale.

Le problème est complexe car si d'une part en cas d'incendie, il faut prévoir des débits très supérieurs, on peut, d'autre part, admettre dans ce cas des pertes de charge supérieures. Néanmoins, projeter un réseau qui suppose une protection correcte contre les incendies oblige à l'emploi de diamètres minimaux supérieurs à ceux qui résulteraient d'un calcul uniquement basé sur l'alimentation normale. Ce renchérissement doit être financé par l'État ou les Municipalités (Services d'Incendie) ou être répercutée en proportion sur le prix de l'eau. La première solution est celle que préconise la plus grande partie des réponses au questionnaire, qui considèrent que ‚tant l'investissement supplémentaire que l'installation des bouches d'incendie et leur entretien doivent être à la charge du Service d'Incendies. Dans un cas, il a été prévu une indemnisation directe de la part de l'Etat ou de la Municipalité qui couvre un certain pourcentage du budget du Service d'Eau. Dans quelques cas seulement, l'augmentation se répercuté sur le prix de vente de l'eau.

Sur un plan théorique et schématique, nous avons essayé de calculer approximativement l'ordre de grandeur de l'augmentation du coût des réseaux et l'influence de l'installation des bouches d'incendie. Nous avons pris comme base les renseignements sur les réseaux de sept villes espagnoles, réseaux pratiquement dimensionnés seulement en fonction de l'alimentation normale et nous avons préparé le TABLEAU V.

Pour l'estimation des coûts unitaires, nous avons pris comme point de départ les prix-catalogue d'un type déterminé de tuyau, en prévoyant 20% pour la pose, avec des tranchées de volume variable, tout ceci pour des diamètres de 80, 300, 700 at 1 000 mm pris comme valeurs représentatives moyennes de chacun des intervalles de diamètres.

Nous estimons que l'étude des réseaux qui correspondent aux débits à fournir en cas d'incendie supposerait passer de 80 à 125 mm de diamètre, ce qui suppose passer de 448 pesetas/m à 640 pesetas/m avec une augmentation de 43% sur le coût du premier intervalle. Si on considère que ceci représente un 20% du total, l'augmentation sur le total du réseau de distribution serait de 9%.

Nous avons calculé ci-après, en nous basant sur un montant total de 40 000 pesetas par bouche d'incendie, ce que représente la protection contra l'incendie suivant le risque de chaque aire. Dans la TABLEAU VI nous donnons le détail du calcul et la répercussion sur le coût d'un réseau de distribution estimé à 700 000 pesetas par hectare. Nous pouvons prendre 6% comme valeur moyenne des résultats.

Si l'on ajoute à cette valeur le 9% calculé antérieurement, nous arrivons à la conclusion que la protection contre les incendies suppose une augmentation totale de 15% sur le coût du réseau de distribution. Ce coût peut représenter 20 à 65% du total des investissements d'un Service d'Eau, ceci dépendant des caractéristiques propres à chaque cas. On peut calculer ainsi que l'augmentation provoquée par le calcul des réseaux, en

TABLEAU V

Intervalle de diamètres en mm	< 150		150-500		500-1000		> 1000		
Ville	km	%	km	%	km	%	km	%	TOTAL
MADRID	1398	42	1636	50	197	6	56	2	3 287
BARCELONA	1 491	67	589	26	98	4	56	3	2 234
SEVILLA	640	66	250	26	52	6	22	2	964
VALENCIA	507	69	178	24	36	5	17	2	738
PALMA DE MALLORCA	322	72	98	22	27	6	-	-	447
CORDOBA	315	80	50	13	27	7	1	-	393
BURGOS	63	46	71	52	2	2	-	-	136
TOTAL	4 736	58	2 872	35	439	5	152	2	8 199
Coût prévu par intervalle: Pesetas/m	448		1 587		5 568		11 215		
Montant par intervalle: Millions de pesetas	2 121		4 558		2 444		1 705		10 828
% que suppose l'intervalle dans le total du montant	20		42		22		16		100

TABLEAU VI

Aire de risque	Nombre d'hydrants par hectare	Coût des hydrants Pesetas/ha	% du coût du réseau
A	2,00	80 000	11
B	1,43	57 200	8
C	1,00	40 000	6
D	0,67	26 800	4
E	0,50	20 000	3
F	0,25	10 000	1

tenant compte de la protection contre les incendies, doit être comprise entre 3 et 10% du total des investissements cités cidessus.

Si l'on considère que les investissements représentent 35% du prix de l'eau, l'augmentation de ce prix, s'il fallait prévoir une protection contre les incendies, serait de l'ordre de 1 à 3,5%. Même si l'on tient compte du fait que les frais d'entretien augmentent aussi, ce pourcentage orientatif correspond aux réponses reçues dans le questionnaire que nous avons envoyé qui estiment que l'augmentation totale est de 0,6 à 5%.

8.2 Aspects légaux

On ne peut pas oublier la tendance bien définie qu'a la jurisprudence en ce moment à exiger des responsabilités civiles dans les cas de dommages aux tiers qui sont conséquence d'un mauvais fonctionnament ou du mauvais état de biens ou services publics. A l'intérieur de cette tendance, une normalisation tendant à fixer les règles de disponibilité et d'emploi de l'eau en cas d'incendie pourrait d'une certaine façon faire que les Services d'Eau assument des responsabilités importantes dans tous les cas où le débit fourni par les prises d'incendie serait inférieur à celui qui est spécifié dans cette normalisation, à la rédaction duquel le Service d'Eau n'aurait pas pris part et qu'il n'aurait pas approuvée.

On peut conclure des réponses faites au questionnaire, en premier lieu, que jusqu'à présent il y a eu peu de conflits qui aient eu pour origine ce problème et en second lieu, que tous les Services d'Eau sachant que la fourniture d'eau pour les incendies dépend de nombreuses circonstances, très souvent indépendantes de sa gestion, estiment qu'il ne faut pas accepter une responsabilité quelconque dans les cas où pour faire face à un incendie la fourniture d'eau viendrait à manquer ou serait insuffisante. Dans de nombreux cas, on essaie d'établir un certain critère général au moment de l'établissement des contrats concernant les installation privées contre les incendies en y incluant une clause qui stipule, et ceci de façon très ample et assez vague, que le Service d'Eau est dégagé de toute sorte de responsabilité. D'une part, nous ignorons, par manque d'expérience, la valeur juridique que pourrait avoir ce type de clauses dans les cas de décision judiciaire et, d'autre part, il faut noter que les prises publiques pour l'extinction des incendies ne sont sujettes à aucune spécification contractuelle.

Il est évident qu'un réseau de distribution d'eau peut offrir une aide importante pour la protection contre le feu. Pour cette raison, il est logique que les Services d'Eau remplissent éventuellement cette fonction sociale tant qu'elle ne représente pas pour eux une série de responsabilités, qui dans certains cas pourraient revêtir un caractère très grave. Dans ce dernier cas, on pourrait arriver à la solution absurde de ne pas fournir un service qui dans tous les cas représenterait une aide très importante.

Si l'on cherche un équilibre entre les deux extrêmes, nous pensons qu'il est intéressant de préparer, avec la collaboration des Services d'Eau concernés, une normalisation qui facilite la matérialisation d'une protection adéquate contre les incendies. Mais il ne faudrait pas que cette bonne volonté puisse être à l'origine d'une prise de responsabilité dans les cas d'une fourniture déficiente, sauf bien entendu si ceux-ci avaient pour cause une négligence évidente et élémentaire.

Nous pensons qu'il est de l'intérêt général que l'A.I.D.E. approfondisse ce thème de façon à obtenir une base juridique adéquate se conformant à ce qui a été exposé ci-dessus. Pour cela, nous proposons au Comité Scientifique et Technique la création d'un groupe mixte technico-juridique à l'intérieur du Comité international permanent Distribution de l'eau pour qu'il approfondisse cette importante question.

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Water requirements for Firefighting

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1 Introduction

Not all the topics proposed by the Scientific and Technical Committee for inclusion in the report could be included within the limited space allotted to Special Subjects. In addition, this subject is being dealt with by the I.W.S.A. for the first time. We therefore decided that this first paper should concentrate on public hydrants used by firefighting services for extinguishing fires. As far as the design of distribution systems is concerned this subject is quite independent of the problem of supplying private firefighting systems. We have also tended to approach this subject with urban areas in mind, as this is the particular interest of the two authors.

The object has been to gather together the various items of information existing and try to establish comparisons between the available facts in order to deduce some general lines of approach to the subject, and from these and our own experience, draw conclusions which will be of use to water suppliers.

Different aspects of the subject will be studied, from which conclusions will be drawn. The subject will be subdivided into the following: use of water, hydrants and their characteristics, quantities of water required, the positioning of public hydrants to enable the water to be obtained, and economic and legal aspects.

2 Use of water

Water for firefighting is used in various ways in terms of application and quantity, and this particular aspect has been subdivided into three categories.

1. Large residential blocks with their own installations, often operated by personnel specially trained in firefighting. They generally have their own sources of water or storage tanks. Provision for their water requirements from the public distribution system has had to be worked out in each specific case.

2. A large number of private installations in buildings for which the water requirements are moderate and which often depend entirely on the public water supply system. They have in common the fact that they are only used in the initial stages of firefighting generally by untrained personnel. Their water requirements are limited, and if the fire gains a hold, the public fire services are required.

3. The use by the firefighting services of water supplied from hydrant points. Into this group fall many fires from the previous group which could not be controlled from the outset.

We have analysed the way in which the water is used in the last two groups. In general terms the following principles apply:

The object of private internal firefighting installations is to put out fires without having to call on the public firefighting services.

When the public fire service is called upon, it almost invariably uses public hydrants and hoses water on to the fire from outside the building. From that moment on the usefulness of the internal installation is negligible or nil.

The public fire service starts by using water stored in the tanks of its engines, putting it under pressure. Present day fire engines operate at two different pressures: about 7 bars and 30 bars. The first is used with normal nozzles to obtain a direct effect with the water, taking advantage of the jetting distance and sometimes the effect of impact. The second is used to obtain a mist or fine spray of water, causing less havoc and consuming less water, although its jetting distance is limited.

Once the operation is started, the supply of water to the fire engines is organised as soon as the hydrants in the sector are located. If they are close, the engines are supplied direct by means of hoses with a maximum length of 150 m. If the distance is greater, service tankers must transport water to the engines.

With the operation organised in this way, the number of engines and firefighting teams used is as circumstances require. The number will decrease as the fire is brought under control.

As soon as the active phase of the fire is ended, the fire engines are withdrawn successively but it is prudent to retain the option of spraying the fire by means of nozzles connected to the public supply, provided that the pressure of the system is above 3 kgf/cm^2.

From the above sequence of events it is possible to deduce the following conclusions with regard to the public water supply.

2(a). Private firefighting installations in buildings having moderate requirements are of limited use and only require a small flow. In practice they are not used at the same time as public fire hydrants.

2(b). Direct supply of nozzles from public hydrants is not practised, and therefore the pressure in the distribution system is not important.

2(c). With regard to output, the fire service does not require the supply points to be very close together. Distances up to 150 m maximum from the point of use are acceptable.

3 Summary of general information

The standards or practices used in the different countries with regard to capacity, characteristics and siting of public hydrants for firefighting reveal a lack of clear definition, with the exception of the standardisation of the hydrant connection. This lack of definition is not surprising as the requirements of hydrant points are imposed by undefined and variable circumstances, which are met by a distribution system whose geometric layout has to conform to urban situations which are never repeated.

Although only a few countries have been considered it has been difficult to prepare a systematic and truly comparative summary. An excessive simplification has been required for Table I but it is an attempt to compare the choice of general criteria, based on reasoning and experience, as a function of the essentially variable and varied requirements.

TABLE I SUMMARY OF GENERAL INFORMATION

Type of information	Countries supplying information HOLLAND	UNITED STATES	BELGIUM	FRANCE	ENGLAND	SWITZERLAND	GERMANY
Main source of information	Water Supply Committee of the Water Industry Research Institute (KIWA) June 1974	Directives of the National Board of Fire Underwriters	Talk by the Inspector General of the Fire Service June 1968	Circular No. 465 dated December of the Ministries of the Interior, Town Planning and Architecture	Manual of British Water Engineering Practice Ed. 1969	Recommendations for Fire Prevention. Provisional Edition 1973. Department of Fire Prevention	
Quantity of water required	Level of Risk A: 360 m³/h Level of Risk B: 270 m³/h Level of Risk C: 180 m³/h Level of Risk D: 120 m³/h Level of Risk E: 90 m³/h Level of Risk F: 60 m³/h	Maximum per fire Town of 200 000 inhabs.: 2725 m³/h „ „ 95 000 inhabs.: 2040 m³/h „ „ 55 000 inhabs.: 1590 m³/h „ „ 33 000 inhabs.: 1135 m³/h „ „ 10 000 inhabs.: 680 m³/h „ „ 1 000 inhabs.: 227 m³/h	Minimum 50 m³/h	Minimum 60 m³/h With high risk, several times 60 m³/h		High risk: 216 m³/h Average risk: 108 m³/h Low risk: 54 m³/h	Special: Greater than 192 m³/h Maximum: Greater than 192 m³/h Minimum: Greater than 24 m³/h
Hydrant characteristics	Flow: 75 m³/h Head loss: 1 kgf/cm² Inlet diameter: not indicated	AWWA C 502 Flow 2 × 60 m³/h Head loss: 0,14 kgf/cm² Inlet diameter: 100 mm	NBN 610 Flow 75 m³/h (from a 70 mm dia point) Loss of head: 1 kgf/cm² (max) Inlet diameter: 100 mm Belgian standard for firefighting Flow 48 m³/h	NF S 61–211 Flow 60 m³/h Loss of head: 0,1 kgf/cm² max. Inlet diameter: 100 mm NF S 61-213 Flow 120 m³/h Loss of head: 0,1 kgf/cm² max. Inlet diameter: 150 mm	B.S. 750: 1964 Flow 123 m³/h Loss of head: 1,75 kgf/cm² max. Inlet diameter: 76 mm	Flow: 70–90 m³/h	
Distance between hydrants	Not shown	100–150 m	100 m in urban areas 200 m in rural areas	200–300 m	90 m in urban areas 140 m in residential areas	80–120 m	
Minimum pipe diameter	100 and 150 mm	150 and 200 mm	100 and 150 mm	Equal to or greater than hydrant inlet diameter			
Time		4–10 hours	2 hours	2 hours		1,5–2 hours	

4 Hydrants
Test and operational characteristics

Two types of hydrant are used, depending on the requirements of each urban area; buried hydrants beneath access covers, which do not project above ground level (NF S 61-211 and B.S. 750: 1964), and post or column hydrants which are above ground level (AWWA C 502, NBN 610 and NF S 61-213). The first type does not cause obstruction to predestrians, is cheap and rarely fails to operate. The second type is more conspicuous, expensive, more easily fails to work and is exposed to external knocks. The increasing likelihood even on pavements, of a parked vehicle rendering a hydrant unusable has caused some thought to be given to a simpler, cheaper and more robust type, above ground, which cannot be put out of action. The authors' opinion is that in urban areas, buried hydrants are best and post hydrants should be reserved for special cases, such as areas covered with vegetation and where there is the risk of a hydrant being rendered unusable by parking.

We go on to analyse the hydraulic performance of an installed hydrant. Each standard establishes the hydraulic tests with which a hydrant must comply. Thus in figure 1 (see page J3) points 1, 2 and 3 correspond to the standards referred to: the head losses are very low for flows of 60 m³/h or 2×60 m³/h. Belgian standard NBN 610 appears to allow a higher head loss, but this relates to a hydrant with two outlets of 70 mm and one of 100 mm, during the test on which only one of the smaller outlets is kept open.

The actual performance of a hydrant in practice differs from that during the test, as an inevitable result of the conditions of installation and operation. Each hydrant installation requires a T connection to the distribution system, a wedge gate valve, and a short section of pipework with bends as appropriate for placing the hydrant outlet in the correct position (see figure 2 on page J4). The loss of head in the hydrant proper is very small compared with that in the connecting pipework. Furthermore the fire engine obtains its water from the hydrant by means of flexible hoses. Overall, between the distribution pipework, which is at mains supply pressure, and the fire engine, the hydraulic resistance is much greater. In fact operating characteristics are obtained which give a head loss corresponding at least to the straight line passing through point A of figure 1 (60 m³/h and 1 kgf/cm² head loss). The flow actually passing a hydrant will depend on the installation conditions, the actual method used to connect the fire engine, the available pressure in the distribution system and the layout of the distribution system.

5 Quantity of water required

In Table I it will be seen that the minimum requirements among European countries are generally consistent and they have been fixed at 60 m³/h, per fire. This rate of flow was recently estimated by the working group of KIWA (Holland) for Risk F (minimum). Furthermore, out of 40 000 incidents in France a flow of 50 m³/h was sufficient for 90% of them. This appears adequately to justify a minimum of 60 m³/h.

75% of incidents last less than two hours, from which it is concluded that a total volume of water of 120 m³ is sufficient for a very high percentage of fires. If the specific water consumption is low (100 l/cap d) and tank capacity limited (25% of daily consumption), on the basis of only a third of this capacity, a town of 15 000 inhabitants will have sufficient reserve in its drinking water distribution system alone to provide water for fire fighting. It will only be for smaller towns that it will be necessary to provide additional reserves of 120 m³.

With regard to the maximum flow, Table I suggests large differences (360 and 2 725 m³/h), which are more apparent than real. As well as the flow requirements, it is necessary to consider at the same time the surface area corresponding to each hydrant contributing to the total flow. This involves the distances between the hydrants.

The 60 m³/h flow corresponds to the normal operating output of fire engines of the capacity generally used by the Fire Services, and also to the nominal flow rates that can be obtained from the most common fire hydrants, i.e. those of 100 mm diameter as measured at the inlet flanges. Thus a fire that does not require a flow greater than the minimum can be put out with water from a single standard hydrant.

Table I shows the recommended distances between fire hydrants. 100 m conforms to what is considered normal for urban environments, as the information gathered in France has a pronounced rural bias and corresponds to conditions of minimum protection. This distance, applied regularly and in ideal conditions, presumes that 60 m³/h are available for each 100 m× 100 m = 1 hectare of area. It is logical to assume that for maximum risks distances between standard hydrants will be smaller or that larger hydrants (inlet 160 mm diameter) with the 100 m spacing will be used. This may result in a provision for such risks of 60 m³/h per 0,5 hectare. Thus the 360 m³/h allowed for by the KIWA working group assumes a fire affecting 3 hectares of area. However, in the maximum flow of 2 725 m³/h allowed for in the United States each hydrant is allotted an average area of 0,37 hectare to protect, that is a total area affected of 17 hectares. There is no large difference in the distribution of the hydrants, but it is assumed that the surface area affected is almost six times greater.

TABLE II

Risk	Flow per fire	Average surface area per hydrant of 60 m³/h flow	Average distance between hydrants
Risk A	360 m³/h	0,5 ha	70 m
Risk B	270 m³/h	0,7 ha	80 m
Risk C	180 m³/h	1,0 ha	100 m
Risk D	120 m³/h	1,5 ha	125 m
Risk E	90 m³/h	2,0 ha	140 m
Risk F	60 m³/h	4,0 ha	200 m

Within a hydrant layout having some regularity, it is assumed simply that a larger number of hydrants will operate simultaneously where a large area is affected. If the water demand per hectare has been adjusted on the basis of a correct minimum distance between hydrants, in the case of a distribution grid for a large town the water supplier's concern over the number of hydrants operating simultaneously will be secondary, because as the area increases the number of large water mains available to supply the required total flow of water will also increase.

To summarise, it is proposed that virtually without modification, the required flows be those indicated by the KIWA working party, supplemented by an indication of the surface area corresponding approximately to each 60 m^3/h hydrant (Table II).

The characteristics of the risk areas are as follows:

Risk A (very high): Old port areas with high building density. Commercial or business districts with narrow streets, high buildings with enclosed blocks of houses and without fire walls. Industrial complexes storing products of high calorific value which have not taken adequate precautions from the point of view of structure and fire protection.

Risk B (high risk): Areas with covered or open stores where the necessary structural and fire preventive measures have been taken. Old industrial areas situated close to high density buildings. Business and commercial districts with narrow streets and high buildings with fire walls. Areas with considerable fire risk in old towns of historical interest.

Risk C (considerable risk): Modern areas surrounded by covered or open stores, where the necessary fire precautions have been taken. Modern buildings and industrial complexes where adequate security measures have been taken into account at design stage. Small industrial districts. Large commercial districts consisting generally of high buildings whose structures are incombustible. Large blocks of flats where adequate fire precautions have been taken.

Risk D (moderate risk): Buildings with incombustible structures situated within closed blocks of houses with wide streets. Low residential buildings. Retail businesses and small industries.

Risk E (low risk): Isolated buildings of not more than four storeys. Modern residential areas in open or closed blocks of houses with 50 % site coverage.

Risk F (minimum risk): Buildings in rural areas where the risk of fire is low. Scattered farms and leisure areas. Urban districts consisting of individual, low rise, single family buildings.

6 Distribution system and minimum diameters

Two examples of the application of Table II will assist in its comprehension and enable important conclusions to be drawn with regard to the layout of the distribution system and minimum pipe diameters. Two specific cases of urban systems have been chosen. For the first an ideal system has been assumed and for the second an existing system which has developed over a number of years.

The first practical application consists of part of a system which is typical of Barcelona. It includes dwelling blocks, offices and retail businesses of good construction, more than 50 years old, from six to ten storeys high, the streets being about 20 metres wide, and including practically no factories or stores. It clearly approximates to Risk Level D (moderate risk) which corresponds to an average area of 1,5 hectare or each 60 m^3/h hydrant. The total area is 28 hectares, for which 19 hydrants of 100 mm diameter inlet should be provided, distributed in a regular pattern and in the most logical positions. Cross roads are without doubt the most favourable positions, for the operation of vehicles and securing a good range, and they are coincident with the actual nodes of the distribution grid.

If hydrants are placed at all the cross roads they will total 16, corresponding to a surface area of 1,75 hectare per hydrant, which is greater than it should be. This arrangement is shown in figure 3 (see page J5). The distance between hydrants is 133 m, while that recommended in Table II is 125 m.

It was necessary to establish a logical layout for the system among the many that could be considered. A large diameter primary main runs along the top of the zone and from it, at 530 m intervals, run secondary mains of 300 mm diameter. Between each pair of the latter run the local tertiary mains, connected to the secondaries at 133 m intervals. This is an example of a very symmetrical layout such as can be found in Barcelona. This ideal system was the subject of tests to veryify design using a digital mathematical model, and taking the total of peak consumption of water and the output from the outlets of one, two or three hydrants, applying characteristic A in figure 1 to the latter. It has thus been possible to show that if the tertiary distribution system is entirely in 100 mm diameter pipework, the flow would barely be sufficient in the case of a fire, even if the system pressure drops to values of the order of 1 kgf/cm^2; any anomaly would cause a shortage of water giving rise to a critical situation.

It was evident that adherence to the rule of 125 mm minimum diameter, i.e. for all the tertiary system, would be excessive. For this reason the final design adopted pipes of 125 mm diameter between the secondary mains, and all the remainder 100 mm pipes, with the hydrants connected to the 125 mm pipes. This is the layout shown in figure 3 (see page J5).

The results of the final design are shown in Table III. The pressure in the primary main was taken as 4,5 and 3 kgf/cm^2, as in most cases in practice the pressures will lie between these two limits, assumed to be constant at points M and N of figure 3. Also, it has been assumed that the whole zone is at the same altitude.

It should be noted that this is a complete distribution grid, with a minimum diameter of 100 mm for normal supply and 125 mm for the mains to which the 100 mm hydrants are connected. These 125 mm mains connect the secondary mains, which means that the hydrants are always supplied from two sides. In these conditions the 120 m^3/h required for a fire is amply supplied by two hydrants (174 m^3/h). The distance between the secondary mains is considerable; it would be possible to increase the distance of 530 m up to 800 m without causing any changes likely to render the flow or pressure insufficient. Finally, we find that the use of three hydrants together has little effect on this system compared with the use of a single hydrant. This confirms that the distribution system is adequately sized, well laid out and that the distance between hydrants is correct.

The second practical application concerns an actual section of the distribution system which has not been completely developed and supplies an old, densely populated quarter with narrow streets close to the port. Its risk level is A (very high risk) which according to Table II corresponds to an average of 0,5 hectare per hydrant of 60 m^3/h flow. The total area is 14,7 hectares and the required total number of hydrants of 100 mm diameter is 29. By placing most of them at crossroads and the remainder around the outside of the area, in order to retain approximately the distance of 70 metres (Table II) 26 hydrants have been used, the area per hydrant being 0,56 hectare. The positions are shown in figure 4 (see page J7).

The actual layout of the distribution system has been used, with the hydrants always connected to the 150 mm mains while the remaining mains are 100 mm diameter or less. In this actual system some open circuits exist and in some cases the 150 mm pipes end at the hydrant connection, as in cases A, B and C. The secondary mains are situated approximately 650 metres from one another. As above, the mathematical model was used and the normal maximum flow of the system was added to the flow from the fire hydrants. The results are summarised in Table IV, with the pressures at points M, N and O of figures 4 being assumed constant. The level difference between these three points is 3 metres.

TABLE III —(see figure 3 on page J5)

Pressure at M and N	4,5 kgf/cm²		3 kgf/cm²	
Hydrants in use	Flow in m³/h	Minimum pressure in system kgf/cm²	Flow in m³/h	Minimum pressure in system kgf/cm²
A	119	3,6	94	2,3
A B TOTAL	110 111 221	3,1	87 87 174	1,9
A B C TOTAL	105 105 110 320	2,8	82 83 85 250	1,8

TABLE IV—(see figure 4 on page J7)

Pressure at 0	4,5 kgf/cm²		3 kgf/cm²	
Hydrants in use	Flow in m³/h	Minimum pressure in the system kgf/cm²	Flow in m³/h	Minimum pressure in the system kgf/cm²
A	120	3,6	97	2,4
A B C TOTAL	107 106 109 322	2,9	86 85 88 259	1,9
A B C D E TOTAL	94 92 94 94 97 471	2,2	75 75 74 74 77 374	1,5

Attention should be drawn to the fact that, according to the calculation, with 125 mm diameter pipework it would not have been possible to obtain 360 m³/h with six hydrants. An arrangement with the hydrants situated further off-centre might be less favourable, and it is for this reason that closure of the supply circuits with a small additional amount of 150 mm pipework is recommended. The distance between secondary mains can barely exceed 650 m as the minimum pressure is already at undesirable levels, with the possibility of below normal pressures in some high buildings. The hydrants are sensitive to the influence of others in the area, for the flow varies from 97 m³/h for one in service to 74 m³/h when five are required to obtain a total of 360 m³/h per fire.

The use of Table II has thus been explained, and it may seem that a similar check would represent the most comprehensive method in each case. However, it is considered that such a check is only necessary in special cases, for the observations made together with the logical interpretation of the results enable the following conclusions to be drawn:

6(a). The flows required for fire fighting are those specified in Table II, supplemented by the description of the levels of risk.

6(b). In the case of a distribution grid, the secondary mains, with a diameter of 300 mm or more, should not be situated at distances greater than 650 m from one another for Risk A, possibly increasing progressively to 800 m for Risk D and up to 1000 m for Risk F.

6(c). Fire hydrants with an inlet diameter of 100 mm should be connected to pipework with a minimum diameter of 150 mm for Risks A, B and C. This may be reduced to 125 mm minimum diameter for Risks D, E and F.

6(d). The pipework connecting the secondary mains to which the fire hydrants are directly connected should not include any section with a diameter less than the minimum, that is they must provide the correct supply from both sides of the connection. The remaining tertiary mains can be at a smaller diameter, although it is useful for them to close off open sections of the distribution grid.

6(e). Hydrants with an inlet of 150 mm are equivalent to two hydrants of 100 mm connected to the same pipework. However, installation of hydrants of this size should be limited to special cases. If the allotted area for the hydrant is increased proportionally, the same comments apply. If this is not the case, the effect of the hydrants being closer together must be taken into account.

6(f). The furthest sections of the distribution system have the greatest difficulties because of their frequently branched layouts, which occurs quite often with Risks E and F. For all types of risks in a widely dispersed system a rough calculation must be made for each particular case, using the flows and areas given in Table II.

6(g). In general, the primary and secondary mains will be ample for the flows proposed in Table II. If there is anything abnormal in their layout or in the risks to be allowed for a check must be carried out.

7 Additional comments

Among the information gathered together, some unquantifiable items have been found. As a result the following additional comments are made.

7(a). In general practice, some disturbance of the normal water supply is accepted during a fire. In normal fires (Table II) the minimum distribution system pressures should not fall below 1,5 bars to avoid backflows into the system. In exceptional circumstances, this pressure can be less (in some questionnaires 0,5 bar is allowed) for such a situation is no less favourable nor more frequent than emptying of the system due to breakdown and repair.

7(b). It must be regarded as normal for fire engines to be forbidden to pump directly from fire hydrants. This may occur in exceptional cases, and flexible hoses, with thin walls which collapse when the internal pressure falls below atmospheric, must be used to connect hydrants to the fire engines.

7(c). National Associations of Water Authorities must take part in drafting legal requirements relating to fire precautions, taking care to adopt rational hydraulic criteria for defining the distribution and number of hydrants and the use of water.

8 Some legal and economic considerations

The technical aspects of these problems have been dealt with thoroughly and it has been possible to come to some conclusions. With regard to the economic aspect, however, the objective should be simplicity and efficiency, for the absolute value is not, proportionally speaking, high. The legal aspect, in Europe at least, is ill defined. It would require a specialised approach, set out in a paper dedicated solely to this aspect. With regard to these two points therefore the following comments are general in nature.

8.1 Economic aspects

The flows per hectare given in Table II are well above those necessary for normal supply. Consequently to design a distribution system allowing for the proper provision for fire fighting assumes an increase in cost in relation to a system strictly based on normal supply.

The problem is complex because if, in designing for fire fighting requirements, much higher flows must be allowed for, higher head losses may also be permitted. However, the design of a system which assumes correct provision for fire fighting requires the use of minimum diameters greater than those which would be adopted if the system were designed only for normal supply. This increase in cost must be financed by the State or the Municipalities (Fire Services) or be recovered as a proportion of the cost of the water. The first solution is that recommended by a majority of replies to the questionnaire, which consider that both the additional investment and the installation and maintenance of the hydrants that should be met by the Fire Services. In one case a direct indemnity was provided by the State or the Municipality covering a certain percentage of the Water Authority's budget. In only a few cases was the increase passed on in the price of water.

In theoretical and diagrammatic form we have attempted to calculate approximately the order of increase of cost of the distribution system and the effect of installing fire hydrants. We used as basis the information obtained on the distribution systems of seven Spanish towns, which were designed in practice purely for normal distribution, and the results are shown in Table V.

For estimating the unit costs, the list prices of a certain type of pipe were used, adding 20% for installation, with trenches of variable volume, for pipes of diameters 80, 300, 700 and 1000 mm which were assumed to be the average representative values for the respective groups of pipe diameters.

It was estimated that the design of the distribution systems allowing for the provision of flows for fire fighting purposes would require pipe diameters to be increased from 80 mm to 125 mm, with the costs increased from 448 pesetas/m to 640 pesetas/m, an increase of 43% on the cost of the first group of pipe sizes. If this group is regarded as being 20% of all sizes, then the increase in cost on the whole distribution system would be about 9%.

Following on this the costs of fire precautions for each level of risk were calculated, using an overall figure of 40 000 pesetas per hydrant. In Table VI the details of the calculations are given together with the effect on the cost of a distribution system estimated at 700 000 pesetas per hectare. An average value of 6% can be obtained from the results.

If the 9% established above is added to this figure the conclusion is that the cost of providing fire fighting facilities represents an increase of 15% on the total cost of the distribution system. The distribution system cost can represent from 20% to 65% of the total investment of a water authority, depending on the circumstances of each case. The increase incurred in system design to include fire prevention can thus be calculated as being between 3% and 10% of the total investment referred to above.

If the investment costs represent 35% of the price of the water, the increase in price, if protection against fire must be made, is in the order of 1–3,5%. Even if it is considered that maintenance costs will also increase, the approximate percentage given above corresponds to the replies received in the questionnaire which estimated a total increase between 0,6 and 5%.

8.2 Legal aspects

One tendency that cannot be overlooked at present is that the law requires acceptance of civil liability in the case of damage to third parties due to the malfunctioning or the bad condition of public services or equipment. One implication of this tendency, a standardisation of the rules on the availability and use of water in case of a fire, is that in some respects the Water Authority would be required to assume considerable responsibility in cases where the flow provided by the hydrants was less than that specified in the standard, which the Water Authority would have taken no part in drafting and would not have approved. It can be concluded from the replies to the questionnaire, firstly, that so far there have been few disputes that have originated in this problem and, secondly, that all Water Authorities, knowing that the supply of water for fire fighting depends on numerous factors, often quite independent of the way in which the

TABLE V

Pipe diameter group mm	< 150		150–500		500–1000		> 1 000		
Town	km	%	km	%	km	%	km	%	TOTAL
MADRID	1 398	42	1 636	50	197	6	56	2	3 287
BARCELONA	1 491	67	589	26	98	4	56	3	2 234
SEVILLE	640	66	250	26	52	6	22	2	964
VALENCIA	507	69	178	24	36	5	17	2	738
PALMA DE MAJORCA	322	72	98	22	27	6	—	—	447
CORDOBA	315	80	50	13	27	7	1	—	393
BURGOS	63	46	71	52	2	2	—	—	136
TOTAL	4 736	58	2 872	35	439	5	152	2	8 199
Cost per group: Pesetas/m	448		1 587		5 568		11 215		
Total cost per group: Million Pesetas	2 121		4 558		2 444		1 705		10 828
Percentage of total represented by group	20		42		22		16		100

TABLE VI

Level of Risk	Number of hydrants per hectare	Cost of hydrants Pesetas/ha	Percentage of cost of system
A	2,00	80 000	11
B	1,43	57 200	8
C	1,00	40 000	6
D	0,67	26 800	4
E	0,50	20 000	3
F	0,25	10 000	1

supply is managed, consider that they cannot accept any responsibility whatsoever in cases where the supply of water fails or is insufficient for fighting a fire. In many cases, attempts are made to establish certain general criteria at the time of drawing up contracts for private fire fighting installations by including a clause stipulating in a suitably broad and vague manner, that the Water Authority is absolved of any sort of responsibility. Through lack of experience, it is not known what value this type of clause has in judicial decisions, and it should be noted that public hydrants for fire fighting are not subject to any contractual specification.

It is clear that a water supply distribution system can provide an important aid in the fighting of fires. For this reason it is logical for Water Authorities to fulfil this social function provided that it does not involve them in responsibilities which may in certain cases be of a very serious nature. In the latter case, the absurd situation could arise where a service would not be provided although in every case, it would be a most important aid.

If a balance between these two extremes is to be sought, we consider that, with the collaboration of the Water Authorities concerned, some form of standardisation should be drawn up which would facilitate the provision of adequate protection against fires. But the goodwill of the Water Authorities should not result in their being laid open to responsibility for deficiencies in supply, except where the cause is clearly negligence.

It is considered that it would be in the general interest for the I.W.S.A. to take this matter further and to draw up an adequate legal basis in accordance with the proposals above. To this end the creation of a joint legal-technical group within the International Standing Committee on Water Distribution, to undertake further investigation of this important question, is proposed to the Scientific and Technical Committee.

Design, Construction and Operation of Service Reservoirs

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1 Introduction

The present report is based on Arbeitsblatt W 311 (Technical Sheet) of the DVGW Regelwerk, a document which the Technical Committee "Water Reservoirs" has updated in accordance with the most recent state of the art. Since its coming into force it forms part of the Technical Regulations of the Federal Republic of Germany. Also included are various questions submitted in advance by some of the I.W.S.A. member countries.

The report deals with all existing types of reservoirs, i.e.

—buried basins (water surface approximately at ground level),
—elevated tanks (water surface and base far above ground level),
—water silos, also called standpipe towers (water surface far above and bottom slightly below ground level), placing particular emphasis upon the following selected subjects;
—planning principles,
—preservation of water quality,
—operational requirements.

2 Design of Service Reservoirs

2.1 Functions

Water reservoirs are installations of water supply. Their storage capacity is necessary

—to compensate for the difference between inflow and outflow;
—to bridge over temporary breakdowns;
—to keep water in stock for fire fighting.

Their water level determines the pressure in the associated supply network.

2.2 Types

The various types of reservoirs can be classified according to

—their location with respect to the ground level (buried basins, water towers, or standpipe silos);
—their mode of operation (continuous flow reservoirs, balancing reservoirs);
—their location with respect to the supply network (overhead or underground reservoirs), and
—their functions (e.g. receiving reservoirs, raw water reservoirs, pure water reservoirs, mixing reservoirs).

2.3 Requirements

Requirements can be subdivided into three different fields:

—operation
—construction
—design.

Requirements imposed by operational experience and needs have priority over any others.

2.3.1 Operational Requirements

a. *Preservation of Water Quality*

The water stored
—must be protected from pollution, and
—must not undergo any changes in quality, e.g. by heating (or cooling), micro-organisms or stagnation.

The installations themselves must look neat and clean.

Contamination can be prevented by appropriate design of the entrance and of the ventilation system, which must exclude any direct contact between the outside world and the water surface. The vent openings should, as a minimum, be so designed as to prevent penetration by birds, insects, dust, foliage, etc. and have an adjustable cross-section. This is achieved in most cases by narrow-meshed, corrosion-proof grids or filters.

Deterioration in quality due to heating (or cooling) is to be prevented:

—in the case of buried basins: by an adequate (0,7 to 1,0 m) soil cover, and
—in the case of water towers and silos: by adequate insulation of the external surfaces, paying due attention to the heat capacity of the stored water varying with the local operational and climatic conditions.

The preservation of water quality under *hygienic, chemical and sensorial* aspects requires

—the use of harmless building materials for the construction and interior lining of the water chambers;
—adequate water and air replacement, the latter combined as required with filtration and with temperature and humidity control.

At the current state of the art, an adequate *replacement of the water* in the water chambers is best achieved by producing a displacement flow. A parallel flow can be obtained using meander- or spiral-shaped diffusers or an elongated, rectangular layout; a spiral flow by means of tangential inflow and central discharge in circular designs; and a sink flow by means of a suitably designed inlet and reservoir layout.

Reference can be made here to the model experiments by Stefaniak of Munich, Langer of Berlin, Marotz of Stuttgart and Mosonyi of Karlsruhe. One feature common to all these experiments was the steady-state condition (inflow = outflow) which, however, is virtually never encountered in waterworks operations.

Moreover the staining methods and drift indicator measurements applied were incapable of determining the influences resulting from temperature or density differences. Since problems of scaling up exist, which are due to divergent opinions as to the applicability of the model laws according to Reynolds, Froude and

Strovhal, the experiments recently conducted under operational conditions by Schubert and Maier using chemical indicators in different sized water reservoirs of the Lake Constance Water Supply System deserve to be given special attention. These experiments have shown that, even in so-called "dead spaces", mixing is much more intensive than previously assumed. The present requirement of a displacement flow can thus be reduced to one of a mixing flow. This greatly reduces the expenditure involved in designing the inlet and outlet, which may now be of similar form to that suggested in 1967 by Reitinger, Vienna.

Largely unexplored so far is the influence exerted on the water quality by the wall/water and air/water interfaces as well as by the condensation water. More recent investigations by Thofern, Botzenhart and Speh of Bonn have revealed heavy bacterial accumulation in these areas. However, this must not necessarily lead to an increased bacterial count in the water discharged, due to the large water volume and adequate water exchange provided.

Replacement of the air in the water chambers is a matter of importance both for the aforementioned hygienic reasons and those of taste. Because of the costs of alternative solutions, natural ventilation via adequately dimensioned openings protected by screens or filters will suffice in most cases.

Exhaust air chimneys in the area of the operators' building and supply air ducts in the area of the water chambers for pre-cooling of the air have proved successful.

In special cases and for especially large reservoirs, conditioning of the supply air may become necessary. This involves major expenditure.

Deterioration of the water quality from the chemical point of view occurs e.g. when the water is fed into the water chambers above the water level. When designing the inlet one must consider, therefore, whether aeration of the water is desirable or whether it should be avoided, e.g. in order to prevent any calcareous deposits.

b. *Fulfilment of Operational Tasks*

The operators must be able

—to carry out maintenance work economically, safely and, if possible, without interruption of operations;
—to work under humane conditions; and
—to obtain the necessary information on the prevailing operating conditions quickly and in readily overseeable form.

Layout planning and equipment shall be designed to meet these requirements. Service reservoirs should basically consist of the operators' building and the water chambers, with suitable building parts taking the place of the operators' building in the case of water towers or silos.

The layout plan for the *operators' building* is to be drawn up so as to comply with the water supply functions of the service reservoir.

The size of the building or room depends on the space required for the hydraulic (mechanical) and electrical equipment. As a minimum the facilities to be provided should comprise:

—An entrance (with porch and snowbreak in regions with prolonged winter conditions).
—An anteroom on the ground floor, giving access to and offering a view of the reservoir (with a separate dry room where electrical and telecommunications equipment is extensive, as well as storage space for cleaning equipment, disinfectants, cleaning agents, emergency ladders, work clothing, etc.).
—A stairway in a separate staircase or in the anteroom to provide easy access to the basement floor.
—A pipework basement on the basement floor to accommodate the hydraulic (and, where necessary, also the mechanical) equipment.

In special cases other rooms are to be provided for
—high-tension equipment
—low-tension equipment
—a battery
—machines such as emergency power unit, pumps
—facilities for intermediate chlorination (disinfection)
—personnel rest room.

Possibilities for extension are to be provided.

As a rule the reservoir space as calculated will be divided into two *water chambers* of identical size, shape and type so that an adequate water supply will be guaranteed both during cleaning and maintenance work.

One water chamber may be deemed sufficient if
—there is another water reservoir in the area supplied, or
—special operational measures (e.g. direct inflow from the water supply line via a pressure-reducing valve) ensure an adequate water supply, even for fire-fighting purposes, during reservoir cleaning and maintenance operations.

For low-capacity water towers ($\leqslant$ 300 m^3) only one chamber is provided as a rule. However, the basic rules mentioned above should be observed in this case too.

The *access* to the water chambers should be so designed as to facilitate supervision as well as cleaning or maintenance work. For example, stairs made of concrete, prefabricated concrete parts or corrosion-resistant steel (ship's stairways) have proved their suitability. Ladders should be used only in small reservoirs of low water depth.

Access may also be provided from the pipework basement through water-tight pressure doors. The transportation of cleaning and maintenance tools and equipment should above all be kept in mind when selecting the access facilities.

Cleaning the walls and supports of the reservoir becomes more difficult as water depth increases; this should be taken into consideration when specifying the *water depth* in the water chambers. Wall heights of 3—4 m can still be handled without special equipment. In large buried basins, water towers and silos, cost considerations make it necessary to provide for far greater water depths (6, 10 or 15 m).

For all service reservoirs a paved *access road* with parking and turning area near the building must be provided.

Access at ground level to the operators' building is to be provided by a solid, heat-insulated door of adequate size.

Where drive-through gates are necessary, their vertical clearance should be at least 3,20 m.

All service reservoirs should be equipped with *mechanical handling equipment* permitting the movement of even the heaviest structural part from the vehicle to its place.

These may consist of

$$\left.\begin{array}{l}\text{—hooks}\\ \text{—rails}\\ \text{—beams with trolleys}\end{array}\right\}\text{to suspend blocks}$$

—crane runaways (hand- or motor-operated) for large structures with heavy loads.

In continuous ceilings, assembly openings permitting the passage of the largest structural parts required should be provided at suitable points.

Admission of a certain amount of *natural light* is required, thus enabling operational and maintenance work to be performed in the operators' building in the absence of electric lighting or in the event of a power failure.

The need for protection from damage or foreign matter penetration makes careful selection of materials necessary. Glass blocks are particularly suitable. In moist rooms such as the pipework basement, intensive daylight admission involves the risk of unpleasant spore or algae formation. All light fittings are to be equipped with ventilating devices corresponding to the material and construction employed.

As a matter of principle, water chambers must never be illuminated by daylight, which would soon lead to the formation of algae on walls and ceilings as well as in the water itself. Artificial light must therefore be used for visual inspection. Should glazed openings be necessary in the operators' building for inspection of the water chambers, the admission of daylight is to be restricted as far as possible. Solid doors with a pedestal behind them in the area of the water chambers, from which visual inspection of the chambers is possible, constitute a better solution.

In designing the *drainage system* all possible cases of need must be taken into consideration. These include

—overflow
—emptying
—cleaning
—dripping and condensed water
—drainage operations
—rain and snow melting
—effluents.

Overflow is the determining factor in the quantitative assessment of the dimensions, which are to be based on the maximum possible inflow into the reservoir.

All drainage installations (except for domestic sewage) are led through a common controllable shaft located either inside or outside the operators' building. Domestic sewage is to be treated separately. Drainage should be designed to match the existing outfall. The water must be evacuated safely, using, e.g., a retaining basin, seepage or an outlet of suitable hydrodynamic design.

Suitably installed traps, grilles, siphons, downflow baffles and the like must be installed to prevent animals, persons and odours from penetrating into the water reservoir through the drainage installations.

In the *hydraulic field*, the minimum requirement of *ducts and lines* comprises

—a supply line with inlet,
—a delivery line,
—an overflow line,
—a drainage line.

The supply and delivery lines must be so arranged in the water chambers that the required replacement of the water is achieved.

The reservoir inlet must be designed to take account of the chemical composition of the water (saturation index and oxygen content).

All ducts and lines must be equipped with the required *valves and fittings*. These include:

—shut-off devices (slide gate or flap valves),
—intake fittings (with free inflow),
—aeration and ventilation fittings in the discharge,
—safeguards against pipe rupture.

The *measuring and control equipment* is to be installed either in the supply line or in the delivery line, or in both; the manufacturer's installation instructions are to be observed in each case.

Clearly visible (enamelled or painted on) water level gauges mounted on supports or walls are recommended to enable the water level to be read by the operating personnel.

Protection against freezing of the hydraulic equipment is usually not necessary in buried basins. On the other hand, all pipe runs of water towers or silos with little or only intermittent water flow are to be protected from freezing (insulation or electric heating, if required).

All ducts, valves, special devices and all passages through water chamber walls must be accommodated in an easily accessible fashion in the operators' building.

All ducts, fittings and valves are to be carefully protected from corrosion.

Water sampling and tapping facilities are to be installed in the individual pipelines for internal use and cleaning purposes.

Local and regional fire-fighting regulations shall be duly adhered to.

Operating errors are avoided by suitable marking (labels) of ducts and valves.

High-tension installations for rated voltages up to 1 000 V, where required, shall conform to the specifications for equipment used in "Moist and wet rooms". This does not apply to those parts which can be installed in dry, heatable rooms for switching and control equipment.

Electrical installations must conform to these specifications, compliance with which is to be ensured by expert control.

Motors and drive units of the selected Protection Class and with rated voltages from 42 up to 1 000 V max. must meet the local requirements of the installation. The applicable regulations for earthing and potential equalisation are to be observed.

The replacement of extremely long power supply lines by stationary or portable power generating units has proved successful in the case of electrical equipment with moderate power consumption, which can remain otherwise unchanged. Special safety measures regarding waste gases and oil are required in this case.

The *low-tension installations* for measurement and (manual or automatic) control may comprise the following plant components or operating conditions:

—water level in the water chambers,
—supply and delivery quantities,
—water level or flow rate-dependent control of inflow valves,
—flow rate-dependent control of other shut-off devices (e.g. for safeguarding against pipe rupture) or of metering systems (e.g. of disinfectants).

The necessary measurements are made, displayed and converted into commands in the building. Interventions and recordings may also be carried out in situ, or alternatively from remote operating stations.

These sensitive instruments and systems are to be installed in dry auxiliary rooms. Where this is not possible, instruments of special design must be employed.

To make monitoring and control possible even in the case of power failure the installation of a battery, or an emergency power unit in the case of larger plants; is necessary. Special rooms are to be provided for this purpose.

As regards the *external installations*, the requirements to be fulfilled include not only their integration in the landscape but also, and particularly, the operational demands to be imposed with respect to maintenance and safety.

—Embankments should be flat enough to be walked on and easily kept in proper condition,
—Water reservoirs should be fenced in to protect them against unauthorised entry, contamination and accidents,
—Clumps of shrubs or trees as well as too solid fencing should be avoided so that the plant will remain readily overseeable.

c. *Protection against unauthorised entry*

To protect the structures against unauthorised entry certain principles should be observed in their planning.

It should also be considered whether additional safety can be achieved by electrical, optical, acoustic or mechanical protection and warning installations.

The former include

—a high degree of reliability of the lighting and venting systems,
—the robust design of doors and door frames,
—the selection of strong and dependable locks,
—the avoidance of direct access to open water surfaces,
—an easy access and passage for vehicles, and
—the enclosure and appropriate design of the external installations.

The latter mainly comprise warning systems which will activate optical or acoustic signals when contacts, light barriers and the like are actuated.

Whenever there is a need for such systems, expert advice should be obtained.

2.3.2 Constructional Requirements

The constructional requirements can be classified into three different fields:

—stability,
—tightness of the water chambers,
—design.

a. *Stability*

Water reservoirs should be solidly founded to safely prevent cracks resulting from settlement. Particular attention is to be given to the following points:

—The foundation soil should be capable of bearing the load, as homogeneous as possible and only slightly susceptible to settlement.
—The admissible bearing pressure should be carefully specified.
—If necessary, ground improvements or constructional measures should be taken.
—In cases of doubt foundation experts should be consulted.
—Structures reaching down to the water table should be fully protected against the uplift pressure.
—To avoid saturation of the foundation soil, bed and exterior walls are to be equipped with drainage facilities.
—The static calculations should cover all possible load conditions in the worst possible combinations.

b. *Watertightness of the Water Chambers*

Leaky reservoirs endanger the stability of the structure as well as the water quality and lead to economic losses. Absolutely watertight chambers are, therefore, a must. This is achieved by

—secure foundation,
—suitably arranged joints to prevent shrinkage cracks,
—watertight, crack-free concrete.

c. *Design*

The design of the water chambers must meet the requirements of

—operation,
—stability and watertightness,
—economy of operation.

The design of the *operators' building* depends on its size and thus on the layout plan.

The following objectives should be striven for:

—logical organisation and lay-out of the building in conformity with the operational sequences;
—technically adequate equipment and landscaping;
—low-maintenance construction with good thermal insulation and damp-proofing of exterior walls and ceilings.

Structural separation of the building from the water chambers is expedient.

To avoid temperature changes of the stored water and to protect the structure against high additional loads resulting from temperature variations, thermal insulation is necessary in most cases.

Particular care is to be devoted to the joints.

Construction joints cannot be avoided; they are vital even in elongated structural parts.

Expansion joints permitting permanent mobility should be avoided if possible. If they cannot be avoided for specific reasons, particular attention must be paid to

—the use of a flexible joint seal,
—a construction permitting movement, and
—the use of a flexible joint-sealing compound which has been approved for use with drinking water.

The *bed* of the excavation is to be immediately and thoroughly drained, with the drainage lines running to manholes.

For practical reasons it is expedient to erect *walls* of uniform thickness from bottom to top. Minimum wall thickness is 250 mm. Only in special cases (large diameter, elevated level, etc.) will prestressing methods be economically justified. An economic comparison with a conventional construction method should always be made, with due attention being paid also to the drawbacks of prestressed concrete in the event of subsequent alteration or extension.

The *ceilings* of the water chambers should be designed to ensure the required air circulation.

As in most cases an *extension of the reservoir capacity* will become necessary in the course of time, the design must allow for this possibility. This means that the required ground area must be available or at least earmarked from the outset and that in its first stage of construction the structure must be designed in such a way that its later extension can be carried out smoothly and without substantial interference with operations.

The simplest method of extension consists of the addition of another water chamber. This requires that the operators' building be designed with this possibility in mind in the planning of the first stage of construction.

In exceptional cases it will also be possible to expand an existing water chamber. This presupposes suitable soil conditions and appropriate preparation, down to the smallest details, of the connections as early as the first stage of construction.

2.3.3 Aesthetic Requirements

Water reservoirs, particularly water towers and silos, are usually located at topographically highly exposed sites. Particular attention should therefore be paid to a satisfactory architectural design. The aim should be

—to give the structure a clear and simple form in keeping with its purpose.
—to adjust the structures, shapes and materials to the demands imposed by the landscape,
—to convey a message about the importance and value of water as an ingredient of life.

These creative efforts should apply not only to the structure as such but also to its immediate environment. Planning should therefore include terrain features, providing a site-oriented vegetation along good landscape-gardening lines, and designing an approach road and enclosure which are both suitable for their purpose and in harmony with the given surroundings.

2.4 Dimensioning

When dimensioning the capacity of water reservoirs the following principles are to be observed:

—The minimum capacity should be equal to the fluctuating volume of a peak consumption day plus an adequate safety margin.
—Future demand developments should be taken into consideration for a period of at least 10 years.
—Too long a retention time during low consumption periods should be avoided.
—The fire-fighting water capacity should conform to local regulations.

The reservoir capacity of small and medium-sized installations should be equal to the maximum 24-hour demand of the area supplied; this order of magnitude is also desirable for large installations. In the case of water supply in groups the overall capacity of all reservoirs should conform to the above demand. It is pointed out that a generously dimensioned capacity will offer not only optimal safety but also the possibility of considerably postponing the need for developing new water resources, especially in the case of supply systems covering a large area and having a supply/simultaneity factor < 1.

2.5 Arrangement

Elevated tanks should be located as close as possible to the area supplied, at a favourable position in relation to the water catchment area, and readily accessible for construction and operational purposes. Buried basins must not lie in flood regions.

The possible basic forms include the following:

—rectangular form
—circular form
—meander form (as a special variety of the rectangular form)
—spiral form (as a special variety of the circular form)
—special forms developed as a result of model experiments or because of special terrain conditions (e.g. tunnel reservoirs).

In multiple-chamber designs the various water chambers are arranged as follows:

—rectangular forms	adjacent to each other
—circular forms	in spectacle form or concentrically

In the case of water towers the concentric arrangement is the only one possible for static reasons.

3 Construction of Service Reservoirs

3.1 Building Materials

Water reservoirs are built to-day exclusively of reinforced concrete. Construction should comply with the pertinent national standards (in West Germany: DIN 1045), with the chief requirement being that the concrete must be impervious to water.

For the further equipping of the water chambers and operators' buildings such materials should be employed as will meet the specific requirements imposed, particularly those resulting from the high humidity level in the spaces concerned, and need only little maintenance. Frost-resistant tiles, clinkers, cast stone slabs have proved particularly satisfactory as wall and floor coverings in operators' buildings. In auxiliary rooms (porch, staircase) of the operators' buildings special plasters based on cement or plastics are also frequently used.

The other materials to be used should also be moisture-resistant and need little maintenance. Materials to be recommended include:

—light-alloys for doors and windows,
—fittings of galvanised steel or plastics,
—stainless steel for ladders and railings.

3.2 Static Calculation

Reinforced concrete parts of water reservoirs shall be so designed as to ensure that the concrete will remain crack-free.

Surfaces wetted by the water should have a concrete cover of 50 mm; wall thickness should at least be 250 mm. The forces resulting from the variable water level, soil pressure, surcharge, shrinkage, creep and temperature should be introduced into the design calculations in their most unfavourable combinations in each case.

In small or medium-sized water reservoirs the use of conventional reinforced concrete (mild steel) is economical. In the case of reservoirs of large dimensions, particularly those with high water levels, a comparison with the prestressed concrete building technique should be made.

3.3 Prefabricated Parts Construction

The prefabricated parts construction technique has so far only been applied in water reservoir construction. Evidently the prefabrication of wall and ceiling parts as well as of supports is not in general associated with appreciable economic gains. The reasons for this are:

—the small number of identical elements,
—the difficulties involved in the standardisation of water reservoirs (including their operators' buildings) because of the broad variety of requirements and conditions, and
—the difficulty of mastering the sealing problems under the vagaries of construction site conditions.

Rationalisation efforts have therefore been concentrated mainly on the construction work itself where remarkable results have been obtained in e.g. the manufacture of efficient formwork, reinforcement with steel matting, manufacture and handling of concrete. Also to be mentioned here is the application of the slip form construction method in the erection of water towers.

3.4 Lining

Properly manufactured, non-porous, smooth and water-tight concrete needs no after-treatment of any kind. Where these preconditions are not fulfilled it will be advisable to line the water chambers. Cement plaster applied in three layers, compacted and smoothed with a steel trowel has proved particularly successful for decades. If, for aesthetic reasons, light-coloured surfaces are necessary, special plasters or paints based on cement are most suitable.

Chlorinated rubber or epoxy resin paints and coatings, however, have frequently fallen short of expectations, since drying out and thus the required adhesion to the base surface usually cannot be achieved, poor workmanship has often affected the result, and repairs are difficult and expensive.

The decision as to what surface treatment will be applied is determined by

—the chemical properties of the water, and
—the suitability of the materials to be used from a physiological point of view.

Tile surfacing, although expensive, has yielded good results provided the tiles are laid carefully and without any voids. In practical operations, however, the tile and joint material used must be taken into consideration when selecting cleaning agents.

In water towers and silos a multiple-layer coating of glass fibre reinforced polyester has also proved satisfactory.

3.5 Exterior Insulation

Insulation is necessary

—to protect soil-covered surfaces against detrimental effects (of soil acids and aggressive waters),
—to protect ceilings and any surfaces exposed to groundwater against penetration of foreign water.

Exterior walls of impervious concrete will be adequately protected by two coats of insulating paint.

On ceilings of water chambers the following seal construction has proved satisfactory:

—priming coat,
—2 layers of no. 500 bitumen roofing felt, bonded with inorganic filler,
—50 mm of reinforced protective concrete,
—2 coats of protective paint,
—100 mm thick drainage layer of gravel.

Surfaces exposed to groundwater are to be sealed against hydraulic water pressure. In West Germany the most expedient way to achieve this is to adhere to pertinent AIB regulations.

3.6 Thermal Insulation

As regards the general and thermal insulation of free-standing structures (operators' building, water tower, water silos) the pertinent working and application instructions are to be observed.

The measures prescribed for the thermal insulation of walls and ceilings must also be applied in the case of structural details and single components.

Doors and windows (insulating glass), frames, lintels (cold infiltration points) must be suitably equipped or designed.

3.7 Protective Coatings

All parts of steel or cast iron must be protected against rust. A bright metallic or clean surface, free of mill scale, grease or moisture is to be produced prior to painting. The hot galvanising of steel parts will improve their resistance to corrosion.

Bitumen-based protective paints, also available in (somewhat dull) colours, have proved particularly effective. They offer good anti-corrosion protection and can be readily repaired. Chlorinated rubber paints and single- or multiple-component paints may also be used. The results obtained will largely depend on the proper pre-treatment and application, which must strictly conform to the manufacturer's instructions.

An adequate thickness and non-porosity of the coating, both of which are to be proved by means of suitable measuring instruments, are of vital importance for a durable protection against corrosion.

3.8 Leakage Testing

Leakage tests are carried out separately for each water chamber, with immediately adjacent chambers having previously been emptied.

Over a period of at least 48 hours there must be no measurable decrease in the water level and no formation of droplets on the exterior walls. For at least a week prior to testing the water chamber must be kept continuously filled so that the concrete can become saturated with water. During the testing period the reservoir must remain locked and sealed. The results of the leakage test are to be entered in a test record.

Because of the long period of contact, thorough disinfection can be achieved during the leakage test by adding 2 mg/l of chlorine to the water.

3.9 Commissioning

The commissioning of the reservoir presupposes that

—plant performance has been verified by an acceptance test establishing the proper construction, including also leakage and functional tests,
—the water chambers and their diffusers, pipelines, etc. exposed to the water flow have been cleaned and disinfected,
—water samples have been bacteriologically tested with negative results, following which the water was declared fit for consumption,
—personnel have been trained to operate the plant and provided with instructions and directions for operation.

All these activities are to be entered in a record.

4 Operation of Service Reservoirs

4.1 Guidelines

Water reservoirs shall be so operated that no deterioration of water quality will occur and that the water delivered will meet all requirements.

The same hygienic requirements apply to the storage of water as to that of any other commodity vital to life. Utmost cleanliness is therefore essential.

Only skilled personnel of sound health are to be employed in the operation of water reservoirs.

For access to water reservoirs by external personnel a special authorisation by the operating company shall be required.

4.2 Mode of Operation

The mode of operation shall be governed by the principles of both safety *and* economy. The storage capacity available is to be utilised to a high degree.

4.3 Maintenance

The maintenance of service reservoirs and all their components must ensure that they are able to fulfil their functions at any time.

The maintenance work required should be laid down in operating instructions in which due differentiation will be made between daily, weekly and less frequent duties.

Any defects are to be eliminated immediately or measures taken for their elimination.

All maintenance work is to be recorded in a station logbook.

Utmost cleanliness (protective clothing) shall be observed whenever maintenance work must be performed inside the water chambers.

4.4 Cleaning

For hygienic and sanitary reasons the water chambers must be cleaned at least once or twice a year. Any deposits on walls or bottoms are to be removed.

The best cleaning effects are achieved by spraying under high pressure (10–15 bar). Such pressure, unless it is available at the reservoir inlet, can be generated by either stationary or portable pumps. Unless a chemical additive with disinfecting properties is added to the cleaning water, disinfection by means of a chlorine solution (2–3 mg Cl_2/l) must follow.

When rinsing or disinfecting water is introduced into outfall drains or sewers, the admissible concentration levels are to be observed in each instance, e.g.:

—pH 6,0–9,0

—residual chlorine content 0,2 mg/l in the outfall drain
2,0 mg/l in the sewers

4.5 Accident Prevention

A principle to be observed right from the planning stage is that both the overall structure and its various details should be so conceived as to exclude as far as possible any risk of accidents in servicing and maintenance work. This applies in particular to the design of stairways, ladders and platforms, the arrangement of windows and lighting fixtures as well as to electrical installations and lifting and handling equipment of every type. The observance of pertinent regulations is supervised in West Germany by the employers' liability insurance companies and, under a law recently enacted, by safety inspectors.

These regulations also cover all measures required to prevent accidents to personnel during their operational activities.

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Sujet spécial 8

Conception, construction et exploitation des réservoirs de distribution

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1 Introduction

Le présent rapport est basé sur la Feuille technique W 311 du règlement de l'Association allemande des spécialistes du gaz et des eaux (DVGW), document que le Comité technique "Réservoirs pour l'eau" que j'ai l'honneur de présider a mis à jour pour tenir compte de l'état de l'art le plus actuel. Depuis son entrée en vigueur, il est inclus dans les Règlements techniques de la République Fédérale d'Allemagne. En outre, j'aborderai des questions soulevées par certains pays membres de l'A.I.D.E.

Conformément au sujet fixé, j'élargirai mon rapport pour inclure tous les types de réservoirs:

—bassins enterrés (surface de l'eau à peu près au niveau du sol),
—réservoirs surélevés (surface de l'eau et radier largement au-dessus du niveau du sol,
—cheminées d'équilibre (surface de l'eau largement au-dessus et radier légèrement au-dessous du niveau du sol).

Compte-tenu du court délai qui m'est imparti et pour ne pas faire double emploi avec le rapport présenté par le Prof. Eero Kajosaari, d'Helsinki, en 1974 au Congrès de Brighton, je mettrai l'accent sur quelques sujets:

—principes de planification,
—conservation de la qualité de l'eau,
—exigences d'exploitation.

2 Conception des réservoirs de distribution

2.1 Fonctions

Les réservoirs font partie de la distribution d'eau. Leur capacité d'accumulation est nécessaire,

—pour compenser les différences entre les débits adduits et les consommations,
—pour passer les arrêts temporaires,
—pour les besoins d'incendie.

Leur niveau détermine la pression dans le réseau associé.

2.2 Types

Les divers types de réservoirs peuvent être classés selon:

—leur situation par rapport au sol (bassin enterré, château d'eau ou cheminée d'équilibre),
—leur mode d'exploitation (réservoir à débit continu ou réservoir d'heures creuses),
—leur emplacement par rapport au réseau de distribution (réservoir surélevé ou enterré), et
—leurs fonctions (réservoir de réception, réservoir à eau brute, réservoir à eau purifiée, réservoirs de mélange).

2.3 Conditions particulières

Les conditions particulières peuvent être réparties entre plusieurs domaines:

—exploitation,
—construction,
—conception.

Les conditions imposées par l'expérience et les besoins de l'exploitations ont priorité sur toutes les autres.

2.3.1 Conditions d'exploitation

2.3.11 Conservation de la qualité de l'eau

L'eau dans le réservoir

—doit être protégée contre la pollution et
—ne doit pas subir de changement de qualité, notamment par réchauffement (ou refroidissement), macroorganisme ou stagnation.

Les installations elles-mêmes doivent avoir un aspect net et propre.

Les *contaminations* peuvent être prévenues par un dessin approprié de l'entrée et du système de ventilation, qui doivent exclure toute relation directe entre le monde extérieur et la surface de l'eau. Les évents doivent être conçus de façon à empêcher le pénétration d'oiseaux, insectes, poussière, feuilles, etc. et avoir une section variable. On l'obtient généralement par des grillages à mailles fines, inoxydables, ou par des filtres.

La *détérioration de qualité* due au réchauffement (ou au refroidissement doit être empêchée:

—pour les bassins enterrés, par une couverture de terre convenable (0,7 à 1,0 m) et
—pour les châteaux d'eau et cheminées d'équilibre, par une isolation convenable de l'extérieur, en tenant compte de ce que la capacité thermique de l'eau emmagasinée varie suivant les conditions locales d'exploitation et les conditions climatiques.

La conservation de la qualité *hygiénique, chimique et organoleptique* de l'eau exige:

—l'emploi de matériaux inoffensifs pour la construction et le revêtement intérieur des cuves;
—un remplacement adéquat de l'eau et de l'air, ce dernier subissant au besoin une filtration et un contrôle de température et d'humidité.

Selon l'état de l'art actuel, on obtient le *remplacement* convenable *de l'eau* dans les cuves par un écoulement en masse. On peut obtenir un écoulement parallèle en utilisant des diffuseurs (méandre ou spirale) ou un plan allongé, rectangulaire, un écoulement spirale par une entrée tangentielle et une sortie centrale avec un plan circulaire, ou un écoulement en entonnoir à l'aide d'une entrée et un plan convenables du réservoir.

On peut citer ici les expériences sur modèle faites par Stefaniak, de Munich, Langer, de Berlin, Marotz, de Stuttgart et Mosonyi, de Karlsruhe. Une caractéristique commune à toutes ces expériences est le régime permanent (entrées = sorties) que l'on ne rencontre cependent virtuellement jamais en exploitation réelle.

De plus, les méthodes de coloration ou d'indicateurs de dérive employées étaient incapables de déterminer les influences résultant des différences de température ou de densité. Comme il y a des problèmes d'échelle, dus aux opinions divergentes sur l'applicabilité des lois de similitude de Reynolds, Froude et Strovhal, les expériences faites en grandeur réelle par Schubert et Maier en utilisant des indicateurs chimiques dans des réservoirs de différentes tailles de l'adduction du lac de Constance méritent une attention particulière. Ces expériences ont montré que, même dans ce que l'on appelle les espaces morts, le mélange est beaucoup plus intense qu'on ne l'admettait jusqu'ici. L'exigence actuelle d'un écoulement en masse peut donc être limitée à un écoulement turbulent. Ceci réduit beaucoup les dépenses impliquées par la réalisation des entrées et des sorties, qui peuvent maintenant avoir une forme similaire à celle suggérée en 1967 par Reitinger, de Vienne.

On n'a encore que très peu exploré l'influence exercée sur la qualité de l'eau par les interfaces paroi/eau et air/eau, ainsi que par l'eau de condensation. Des recherches récentes par Thofern, Botzenhart et Speh, de Bonn, ont révélé une accumulation de bactéries en ces endroits. Mais cela n'amène pas nécessairement une augmentation des teneurs en bactéries des eaux sortantes, en raison du grand volume de l'eau et des échanges d'eau convenables qui sont réalisés.

Le renouvellement de l'air dans les cuves est une question importante à la fois pour les raisons d'hygiène ci-dessus mentionnées et pour le goût. Etant donné le coût des autres solutions, la ventilation naturelle par des ouvertures convenablement dimensionnées protégées par des grillages ou des filtres suffira généralement.

On s'est bien trouvé de disposer les cheminées d'évacuation de l'air dans la zone des bâtiments d'exploitation et les conduites d'arrivée d'air dans celle des cuves car le pré-refroidissement de l'air s'est montré favorable. Dans les cas spéciaux et pour les réservoirs spécialement importants, le conditionnement de l'air introduit peut devenir nécessaire. Ceci implique des dépenses importantes.

Une *détérioration de la qualité chimique de l'eau* peut se produire, par ex. quand l'eau pénètre dans les cuves par surverse. Lorsque l'on dessine les entrées, on doit donc considérer si l'aération de l'eau est désirable ou s'il faut l'éviter, par ex. pour prévenir les dépôts calcaires.

2.3.12 Prise en compte des besoins d'exploitation

Les exploitants doivent pouvoir:
—réaliser les travaux d'entretien économiquement, en sécurité et si possible sans interrompre l'exploitation,
—travailler dans des conditions humaines et
—obtenir les informations nécessaires, rapidement et sous une forme facilement contrôlable, sur les condition de l'exploitation.

Le plan des ouvrages et les équipements doivent être conçus pour satisfaire ces exigences. Les réservoirs de distribution doivent comprendre en principe un bâtiment d'exploitation et des cuves, les aménagements structurels convenables prenant la place du bâtiment d'exploitation dans le cas des châteaux d'eau et cheminées d'équilibre.

Le plan du *bâtiment d'exploitation* doit permettre de réaliser les fonctions distribution d'eau du réservoir.

La taille du bâtiment ou de la salle dépend de la place nécessaire pour l'équipement hydraulique, (mécanique) et électrique. Il faut prévoir au minimum:

—une entrée (avec un porche et une barrière à neige là où l'hiver est prolongé);
—une antichambre au rez-de-chaussée, donnant accès au réservoir et permettant de le voir (avec une chambre sèche séparée si les équipements électriques et les télécommunications sont importants), ainsi qu'un magasin pour l'équipement de nettoyage, les désinfectants, les produits de nettoyage, les échelles de secours, les vêtements de travail, etc.
—un escalier séparé ou dans l'antichambre pour accéder facilement au sous-sol;
—un sous-sol pour les conduites, où se trouvent les équipements hydrauliques (et s'il y a lieu mécaniques).

En certains cas, il faut prévoir d'autres salles pour:
—l'équipement haute-tension,
—l'équipement basse-tension,
—des accumulateurs,
—des machines telles qu'un groupe de secours, des pompes pour la chloration intermédiaire (désinfection),
—la salle de repos du personnel.

Il faut prévoir des possibilités d'extension.

En général, le volume calculé du réservoir sera divisé en deux *cuves* de taille, forme et type identiques de façon à garantir l'alimentation en eau même pendant les travaux de nettoyage et d'entretien.

Une seule cuve peut être suffisante si:

—il y a un autre réservoir dans la région alimentée,
—des mesures d'exploitation spéciales (par ex. passage direct de la conduite d'adduction par l'intermédiaire d'une vanne réductrice de pression) permettent une alimentation convenable, même pour les besoins d'incendie, pendant les opérations de nettoyage et d'entretien.

Pour les châteaux d'eau de faible capacité ($\leqslant$ 300 m^3) il n'y a généralement qu'une seule cuve. Mais les principes ci-dessus doivent être également observés en ce cas.

L'accès aux cuves doit être conçu de façon à faciliter la surveillance comme les travaux de nettoyage et d'entretien. Par ex. des escaliers en béton ou en béton préfabriqué ou en acier inox (escaliers de navires) se sont montrés convenir. Il ne faut utiliser des échelles que pour les petits réservoirs de faible hauteur d'eau.

Il faut aussi prévoir des accès par des portes étanches à partir de la chambre des tuyauteries en sous-sol. Il faut tenir compte de la manipulation des appareillages de nettoyage et d'entretien lorsque l'on conçoit les accès.

Nettoyer les parois et les piliers du réservoir devient de plus en plus difficile quand la profondeur de l'eau augmente. Il faut y penser quand on fixe *la profondeur de l'eau* dans les cuves. Des parois de 3 à 4 m de haut peuvent encore être traitées sans équipement spécial. Dans les grands réservoirs souterrains, les châteaux d'eau et les cheminées d'équilibre, des considérations de prix de revient imposent des hauteurs bien plus grandes (6, 10 ou 15 m).

Pour tous les réservoirs de distribution, il faut aménager une *route d'accès* revêtue avec possibilités de parking et de demi-tour.

L'accès, au rez-de-chaussés, au bâtiment d'exploitation doit être pourvu d'une porte solide, à isolement thermique, de taille convenable.

Lorsqu'un portail est nécessaire, sa hauteur minimale doit être de 3,2 m.

Tous les réservoirs devraient être équipés de *moyens de levage* permettant la mise en place des pièces les plus lourdes.

Ces moyens peuvent consister en:

—crochets
—rails
—poutres avec chemin de roulement

pour suspendre les charges

—ponts-roulants (à main ou à moteur) pour les bâtiments importants à équipement lourd.

Des ouvertures permettant le passage des pièces les plus grandes doivent être prévues aux endroits convenables dans les planchers continus.

Il faut prévoir un certain *éclairage naturel* permettant d'exécuter dans le bâtiment d'exploitation les manoeuvres courantes et les travaux d'entretien même en l'absence de lumière électrique ou en cas de panne de courant.

La protection désirable contre les dommages et la pénétration de corps étrangers rend nécessaire une sélection soigneuse des matériaux. Les blocs de verre conviennent particulièrement. Dans les pièces humides comme le sous-sol des tuyauteries, un éclairage naturel trop intense entraîne le risque de développement de champignons ou d'algues déplaisants. Tous les dispositifs d'éclairage doivent être équipés des moyens de ventilation convenant au matériau et au mode de construction.

En principe, les cuves à eau ne doivent pas être illuminées par la lumière du jour, qui amènerait rapidement la croissance d'algues sur les parois et la couverture comme dans l'eau elle-même. Il faut donc utiliser la lumière artificielle pour l'inspection visuelle. S'il doit exister des ouvertures vitrées pour permettre de surveiller les cuves depuis le bâtiment d'exploitation, l'admission de lumière du jour doit être aussi restreinte que possible. Des portes massives avec, derrière elles du côté des cuves à eau, un piédestal pour permettre l'inspection constituent la meilleure solution.

Le dimensionnement des *évacuations à l'égout* doit tenir compte de tous les cas possibles:

—débordements,
—vidange,
—nettoyage,
—fuites et eau de condensation,
—drainage,
—pluie et fonte des neiges,
—eaux sanitaires.

Le débordement est le facteur déterminant dans ce dimensionnement; il est basé sur le débit maximal pénétrant dans le réservoir.

Toutes les évacuations (sauf les eaux sanitaires) aboutissent à un puits commun visitable situé dans le bâtiment d'exploitation ou en-dehors. Les eaux sanitaires sont traitées séparément.

Les évacuations doivent être adaptées aux égouts ou émissaires existants en utilisant au besoin un bassin de retenue ou d'infiltration ou un orifice de conception hydrodynamique appropriée.

Des trappes, grilles, siphons, cloisons plongeantes et autres dispositifs doivent empêcher les animaux, les personnes et les odeurs de pénétrer dans le réservoir par les évacuations.

Dans le *domaine hydraulique*, le minimum de *conduites* comprend:

—une conduite d'amenée avec orifice d'entrée,
—une conduite de sortie,
—une conduite de trop plein,
—une conduite de vidange.

Toutes les conduites et tuyauteries doivent être équipes des *vannes et accessoires* appropriés, qui comprennent:

—un dispositif de fermeture (robinet-vanne ou vanne papillon),
—des robinets de prélèvement,
—des soupapes d'aération et de ventilation sur l'évacuation,
—des protections contre les ruptures de conduite.

L'equipement de mesure et de commande doit être installé soit sur la conduite d'amenée, soit sur la conduite de sortie, ou sur les deux. Il faut respecter en tous cas les instructions d'installation du fabricant.

Il est recommandé de monter sur les parois ou sur des supports des échelles de niveau (émaillé ou peint) pour permettre au personnel d'exploitation de lire le le niveau de l'eau.

Il n'est généralement pas nécessaire d'installer pour les réservoirs enterrés une protection contre le gel des équipements hydrauliques. Mais inversement toutes les conduites des châteaux d'eau et des cheminées d'équilibre où le passage de l'eau est faible ou intermittent doivent être protégés contre le gel (isolement ou chauffage électrique si nécessaire).

Toutes les conduites, vannes, appareils spéciaux aussi bien que les passages à travers les parois des cuves doivent être disposés d'une façon facilement accessible dans le bâtiment d'exploitation.

Toutes les conduites, vannes et accessoires doivent être soigneusement protégés contre la corrosion.

Des dispositifs de prise d'échantillons et de prise d'eau doivent être installés sur les diverses conduites pour les usages internes et le nettoyage.

Les règlements locaux contre l'incendie doivent être strictement observés.

On évite les erreurs d'exploitation en marquant convenablement (étiquettes) les conduites et les vannes.

Les installations à haute tension jusqu'à 1 000 V là où elles sont nécessaires, doivent être conformes aux règlements pour les équipements utilisés dans les locaux humides. Cela ne s'applique pas aux parties qui peuvent être installées dans les locaux secs, chauffés, pour les dispositifs de commutation et de commande.

Les installations électriques doivent être conformes aux règlements, dont l'observation doit être vérifiée par du personnel spécialisé. Les moteurs doivent être de la classe de protection appropriée, de tension 42 à 1 000 V maximum, et doivent être conforme aux règlements locaux. Les prescriptions relatives à la mise à la terre et à l'égalisation des potentiels doivent être respectées.

Le remplacement des lignes électriques extrêmement longues par des générateurs fixes ou mobiles s'est montré très efficace dans les cas où la puissance électrique nécessaire est modérée, l'équipement restant pour le reste inchangé. Il faut alors prévoir des mesures de sécurité spéciales concernant les gaz d'échappement et les produits pétroliers.

Les installations basse-tension pour les mesures et les commandes (manuelles ou automatiques) peuvent comprendre les composants suivants:

—niveau de l'eau dans les cuves,
—volumes reçus et livrés,
—commande des vannes d'arrivée par les niveaux et les débits,
—commande suivant les débits des autres dispositifs d'arrêt (par ex. en cas de rupture de conduite) ou de mesure (par ex. de la stérilisation).

Les mesures nécessaires sont faites, affichées et converties en commandes dans le bâtiment d'exploitation. Les interventions et enregistrements peuvent être également effectués sur place, ou à partir de stations de télécommande.

Ces instruments sensibles doivent être installés dans des locaux auxiliaires secs. Là où ce n'est pas possible, il faut utiliser des instruments spécialement conçus.

Pour rendre possible le contrôle continu et les commandes même en cas d'arrêt de courant, il faut prévoir une batterie ou pour les installations importantes, un générateur de secours, installés dans des locaux spéciaux.

En ce qui concerne les installations extérieures, les exigences à satisfaire comprennent non seulement leur intégration dans le paysage, mais aussi et surtout les besoins de l'exploitation à imposer en ce qui concerne l'entretien et la sécurité.

—Les berges doivent être assez plates pour qu'on puisse y marcher et les tenir facilement en bon état.
—les réservoirs doivent être clos pour les protéger contre les intrusions non autorisées, la contamination et les accidents.
—les buissons, les arbres ou clôtures trop épais doivent être évités pour que les installations puissent être faciles à surveiller.

2.3.13 Protection contre les intrusions non autorisées

Pour protéger les structures contre toute intrusion, il faut observer certains principes lors de l'établissement de leurs plans. Il faut également considérer si l'on ne peut pas obtenir une sécurité supplémentaire par une protection électrique, optique ou acoustique et des installations d'alerte.

Ces dispositions structurelles comprennent:

—une fiabilité élevée des réseaux d'éclairage et de ventilation,
—des portes et cadres solides,
—des serrures solides et sûres,
—l'absence d'accès directs aux surfaces d'eau libres,
—un passage facile pour les véhicules,
—la clôture et la conception appropriée des installations extérieures.

La sécurité supplémentaire comprend des réseaux d'alerte qui mettent en action des signaux optiques ou acoustiques en cas de franchissement de contacts, de barrières lumineuses ou autres. Chaque fois que l'on utilisera de tels systèmes, il faudra demander l'avis d'experts.

2.3.1 Conditions de construction

Les conditions de contruction peuvent être classées dans trois domaines différents.

—Stabilité,
—imperméabilité des cuves,
—conception.

2.3.21 Stabilité

Les fondations des réservoirs doivent être solides pour empêcher les fissures dues à des affaissements. Il faut être attentif aux points suivants.

—Le sol de fondation doit être capable de supporter la charge, aussi homogène que possible et n'être susceptible que d'un tassement léger.
—La pression admissible sur le sol doit être soigneusement spécifiée.
—Si nécessaire, il faut prendre des mesures pour améliorer le sol ou prévoir une construction spéciale.
—En cas de doute, il faut consulter des experts en fondation.
—Les structures atteignant la nappe phréatique doivent être protégées contre le soulèvement (sous pression).
—Pour éviter de délaver le sol de fondation, il faut équiper les fondations et les parois extérieures de drains.
—Les calculs statistiques doivent couvrir toutes les conditions de charge possibles dans les combinaisons les plus dangereuses.

2.3.22 Imperméabilité des cuves

Les réservoirs qui fuient mettent en danger la stabilité de leurs structures comme la qualité de l'eau et amènent à des pertes économiques. Les cuves doivent donc être absolument imperméables. On y arrive par:

—des fondations sûres,
—des joints convenablement disposés pour empêcher les fissures de retrait,
—un béton imperméable et non fissuré.

2.3.23 Conception

La conception des cuves doit couvrir les besoins:
—d'exploitation,
—de stabilité et d'imperméabilité,
—d'économie d'exploitation.

La conception du *bâtiment d'exploitation* dépend de sa taille et donc de sa disposition.

Il faut viser aux objectifs suivants: organisation logique et disposition du bâtiment en conformité avec les séquences d'opérations; équipement techniquement adéquat et configuration adaptée au paysage; construction facile à entretenir avec bon isolement thermique et protection contre l'humidité des murs extérieurs et du toit. Il est commode de séparer le bâtiment structuralement des cuves.

Pour éviter les changements de température de l'eau contenue dans les cuves et pour protéger la structure des contraintes additionnelles élevées résultant des variations de température, il est généralement nécessaire de prévoir un isolement thermique.

Il faut porter une attention particulière aux joints.

Les *joints de construction* sont inévitables; ils sont même indispensables dans les éléments de structure allongés.

Les joints d'expansion autorisant une mobilité permanente doivent être si possible évités. S'ils ne peuvent l'être pour une raison spécifique, il y a lieu de veiller à

—l'emploi d'un ruban élastique en permanence pour sceller le joint,
—une construction permettant cette mobilité et
—l'emploi d'un composé de scellement des joints élastique en permanence approuvé pour utilisation dans l'eau potable.

Le radier de l'excavation creusée pour la construction doit être immédiatement et soigneusement draîné, les conduites de drainage aboutissant à des puisards d'inspection.

Pour des raisons pratiques, il est expédient de construire des parois d'épaisseur uniforme de la base au sommet.

Ce n'est que dans des cas spéciaux (grand diamètre, grande hauteur) que la préconstrainte sera économiquement justifiée. Il faudra toujours faire une comparaison économique avec une méthode de construction classique, en tenant compte des inconvénients du béton précontraint en cas de modification ou extension ultérieure.

Les *couvertures* des cuves doivent être conçues pour assurer la circulation d'air nécessaire.

Comme, dans la plupart des cas, une *extension de la capacité des réservoirs* deviendra nécessaire avec le temps, les plans doivent prévoir cette possibilité. Cela veut dire que le terrain nécessaire doit être disponible ou au moins réservé dès le début et que, dès la première étape de construction, la structure doit être disposée de façon à permettre sans difficulté et sans gêne notable pour l'exploitation les extensions ultérieures.

La façon la plus simple de réaliser cette extension est d'ajouter une autre cuve. Cela suppose que le bâtiment d'exploitation ait été conçu en tenant compte de cette nécessité dès la première étape de construction.

Dans des cas exceptionnels, il sera possible d'agrandir la cuve existante. Cela présuppose que le terrain s'y prête et que les préparatifs nécessaires, jusqu'aux derniers détails, de la jonction aient été faits dès la première étape de la construction.

2.3.3 Conditions esthétiques

Les réservoirs, et surtout les châteaux d'eau et cheminées d'équilibre, sont généralement situés en des sites élevés. Il faut donc veiller particulièrement à leur architecture. Le but doit être:

—de donner à l'ouvrage une forme claire et simple qui correspond à son affectation,
—d'ajuster les masses structurelles, les formes et les matériaux aux exigences du paysage,
—d'évoquer l'idée de l'importance et de la valeur de l'eau comme ingrédient vital.

Ces efforts de création doivent viser non seulement l'ouvrage lui-même, mais aussi son environnement immédiat. La conception doit donc inclure le relief du terrain, prévoyant une végétation qui s'harmonise avec l'aspect paysager, ainsi que le tracé de la voie d'accès et la nature de la clôture qui doivent tous deux répondre à leur fonction en harmonie avec l'environnement.

2.4 Dimensionnement

Pour dimensionner la capacité du réservoir, il faut observer les principes suivants:

—la capacité minimale doit être égale aux fluctuations d'une journée de consommation de pointe, plus une marge de sécurité convenable.
—Il faut tenir compte du développement de la demande pour une période d'au moins dix ans.
—Il faut éviter une durée de séjour trop grande pendant les périodes de faible consommation.
—La réserve d'incendie doit être conforme aux règlements locaux.

La capacité en réserve pour un réseau petit ou moyen doit être égale à la demande journalière maximale de la région desservie; cet ordre de grandeur est également désirable pour les grands réseaux. Lorsqu'il s'agit d'un groupement de communes, la capacité de l'ensemble des réservoirs doit être conforme aux exigences ci-dessus. Il faut souligner qu'une capacité largement dimensionnée offrira non seulement une sécurité optimale, mais aussi la possibilité de reculer considérablement la mise en oeuvre de sources nouvelles surtout pour les réseaux étendus ayant un facteur alimentation/simultanéité inférieur à un.

2.5 Situation et plan

Les réservoirs surélevés doivent être placés aussi près que possible de la région desservie, en une position favorable par rapport au captage, facilement accessible pour la construction et l'exploitation. Les réservoirs enterrés ne doivent pas être dans une région inondable.

Les formes de base possibles sont les suivantes:

—rectangle,
—cercle,
—méandre (variété du rectangle),
—spirale (variété du cercle),
—formes spéciales mises au point d'après des recherches sur modèle ou en raison des conditions topographiques spéciales (par ex. réservoirs tunnels).

Lorsque plusieurs cuves sont nécessaires, ces cuves sont disposées comme suit:

—rectangulaires	voisines les unes des autres
—circulaires	de formes spéciales ou concentriques.

Pour les châteaux d'eau, la disposition concentrique est la seule possible pour des raisons statiques.

3 Construction des réservoirs de distribution

3.1 Matériaux

De nos jours, les réservoirs sont construits exclusivement en béton armé. La construction doit être conforme aux normes nationales actuelles (en Allemagne de l'Ouest: DIN 1045), la première nécessité étant que le béton doit être imperméable à l'eau.

Pour l'équipement complementaire des cuves et des bâtiments d'exploitation, les matériaux employés doivent être conformes aux normes imposées (particulièrement celles résultant du haut niveau d'humidité dans les endroits concernés) et ne nécessiter qu'une surveillance réduite. Les tuiles résistant au gel, les briques vitrifiées, les dalles de pierre moulées se sont montrées particulièrement satisfaisantes en tant que revêtement de murs et de sol dans les bâtiments d'exploitation. Dans les pièces auxiliaires (porche, escaliers) des bâtiments d'exploitation, des plâtres spéciaux dérivés du ciment ou des plastiques sont aussi fréquemment utilisés.

Les autres matériaux utilisés doivent être résistants aux moisissures et ne nécessiter qu'une surveillance réduite. Les matériaux recommandés comprennent:

—des alliages légers pour les portes et fenêtres,
—des assemblages d'acier galvanisé ou de plastique,
—de l'acier inoxydable pour les échelles et les gardefous.

3.2 Calculs statiques

Les parties des châteaux d'eau construites en béton doivent être calculées et étudiées de façon à assurer que la béton ne se fendra pas.

Les surfaces mouillées par l'eau doivent être recouvertes d'une épaisseur de béton de 5 cm; l'épaisseur des parois doit être au moins de 25 cm. Les forces résultant du remplissage (variable), de la pression au sol, des surcharges, de la contraction, du fluage et de la température doivent être introduites dans le calcul des dimensions dans leurs conditions les plus défavorables dans chaque cas.

3.3 Construction préfabriquée

La construction préfabriquée a été rarement employée pour les réservoirs de distribution d'eau. Il est évident que la préfabrication des parois, des couvertures et des supports n'apporte pas en général d'économies importantes. Les raisons en sont:

—le petit nombre d'éléments identiques,
—les difficultés impliquées par la normalisation des réservoirs (y compris les bâtiments d'exploitation) en raison de la grande variété des besoins et des conditions, et
—la difficulté de maîtriser les problèmes de joints étanches dans la bousculade du chantier.

Les efforts de rationalisation ont donc été concentrés principalement sur les travaux de construction auxmêmes, où des résultats remarquables ont été obtenus, par ex. dans la fabrication d'excellents éléments de coffrage, le renforcement du ferraillage, la préparation et la manipulation du béton. Il faut également citer l'emploi de coffrages glissants pour la construction de châteaux d'eau.

3.4 Etanchéité

Un béton convenablement préparé, non poreux, lisse et impermeable n'a pas besoin de traitement ultérieur d'aucune sorte. Mais si ces conditions ne sont pas remplies, il sera recommandé d'étancher les parois des cuves. Un mortier de ciment, appliqué en trois couches, compacté et lissé à la truelle s'est montré efficace depuis des décennies. Si pour des raisons esthétiques on veut des surfaces colorées, on peut employer des ciments spéciaux ou des peintures au ciment.

Mais les peintures et revêtements en caoutchouc chloré ou résine epoxy ont souvent déçu, car ils se dessèchent et l'adhésion à la surface de base ne peut généralement pas être obtenue, une mauvaise application affecte souvent le résultat, et les réparations sont difficiles et coûteuses.

La décision au sujet du traitement de surface à employer sera déterminé par:

—les propriétés chimiques de l'eau, et
—la convenance des matériaux à utiliser d'un point de vue physiologique.

Une couverture de tuiles, quoique onéreuse, a donné de bons résultats, pourvu que les tuiles soient posées soigneusement et sans vide. En pratique cependant, il faut tenir compte du matériau des tuiles et des joints lorsqu'on choisit le produit de nettoyage.

Dans les châteaux d'eau et cheminées d'équilibre, on a utilisé aussi avec succès un revêtement de polyester armé de fibres de verre.

3.5 Isolation extérieure

L'isolation est nécessaire:

—pour protéger les surfaces recouvertes de terre contre les dommages (des sols acides et des eaux agressives).
—pour protéger les couvertures et les surfaces exposées aux eaux souterraines contre la pénétration des eaux étrangères.

Les parois extérieures du béton imperméable seront adéquatement protégées par deux couches de peinture isolante.

Les surfaces exposées aux eaux souterraines doivent être rendues impermeables à la pression de ces eaux. En Allemagne Fédérale, la façon la plus pratique pour cela est de respecter les règlements appropriés de l'AIB.

3.6 Isolation thermique

En ce qui concerne l'isolation générale et thermique des bâtiments isolés (bâtiment d'exploitation, château d'eau, cheminée d'équilibre), il y a lieu d'observer les instructions appropriées de mise en oeuvre et d'application.

Les mesures prescrites pour l'isolement thermique des parois et des couvertures doivent également être respectées pour les détails structuraux et les parties isolées.

Les portes et fenêtres (verre isolant), cadres, linteaux (points d'infiltration froids) doivent être convenablement équipés ou conçus.

3.7 Revêtements protecteurs

Toutes les parties en acier ou en fonte doivent être protégées contre la rouille. Il faut réaliser une surface métallique brillante ou propre, débarrassée des scories de laminoir, avant peinture. La galvanisation à chaud des parties en acier améliorera leur résistance à la corrosion.

Les peintures au bitume, que l'on trouve aussi en couleur (assez ternes) se sont montrées particulièrement efficaces. Elles offrent une bonne protection contre la corrosion les retouches sont faciles. On peut également utiliser les peintures au caoutchouc chloré et les peintures composées simples ou multiples. Les résultats obtenus dépendront beaucoup du traitement préalable de la surface et de l'application, qui doivent être strictement conformes aux instructions du fabricant.

La couche doit être suffisamment épaisse et non poreuse, ce que l'on mesure à l'aide des instruments convenables; c'est vital pour une protection durable contre la corrosion.

3.8 Essai de fuites

L'essai de fuites se fait séparément pour chaque cuve, chaque cuve immédiatement adjacente ayant été préalablement vidée.

En 48 heures au moins, on ne doit pas constater d'abaissement mesurable du niveau de l'eau, ni formation de gouttes sur les parois extérieures. Avant l'essai, la cuve doit avoir été constamment pleine pour que le béton soit saturé en eau. Pendant l'essai, le réservoir doit demeurer clos. Les résultats de l'essai de fuites doivent faire l'objet d'un procès-verbal d'essai.

En raison de la longue période de contact, on peut réaliser une désinfection complète pendant l'essai de fuites en ajoutant à l'eau 2 mg/l de chlore.

3.9 Mise en service

La mise en service du réservoir suppose qu'au préalable:

—La valeur de l'ouvrage a été vérifiée par un procès-verbal de réception établissant que la construction est convenable, y compris les essais de fuite et de marche,
—les cuves et leurs diffuseurs, conduites, etc. exposés à l'eau ont été nettoyés et désinfectés,
—des analyses bactériologiques négatives ont été réalisées, à la suite lesquelles l'eau est déclarée bonne pour la consommation,
—le personnel a été formé à l'exploitation de l'ouvrage et a reçu des instructions et directives d'exploitation.

Toutes ces vérifications font l'objet d'un procès-verbal.

4 Exploitation des réservoirs de distribution

4.1 Principes

Les réservoirs doivent être exploités de façon qu'il ne se produise pas de détérioration de la qualité de l'eau, et que celle-ci soit conforme aux règlements.

Les mêmes exigences hygiéniques s'appliquent à la mise en réserve de l'eau qu'aux autres commodités de la vie. Une propreté parfaite est donc essentielle.

Il ne faut affecter à l'exploitation des réservoirs que du personnel expérimenté en parfaite santé.

Pour accéder au réservoir, toute personne étrangère devra avoir une autorisation spéciale de la société exploitante.

4.2 Mode d'exploitation

Le mode d'exploitation doit être gouverné par les principes de sécurité *et* d'économie. La capacité de réserve doit être exploitée au maximum.

4.3 Entretien

L'entretien des réservoirs de distribution et de toutes leurs parties doit assurer qu'ils soient capables de remplir leur office à tout moment. Le travail d'entretien nécessaire doit être consigné dans des instructions d'exploitation qui distingueront les travaux journaliers, hebdomadaires et moins fréquents.

Tout défaut doit être immédiatement éliminé ou des mesures prises pour son élimination.

Tout travail d'entretien est consigné dans un livre d'entretien.

La plus grande propreté (vêtements protecteurs) doit être observée quand un travail d'entretien est réalisé dans les cuves.

4.4 Nettoyage

Pour des raisons hygiéniques et sanitaires, les cuves doivent être nettoyées au moins une ou deux fois par an. Tous dépôts sur les parois et les radiers doivent être enlevés.

La meilleur nettoyage est réalisé par aspersion sous forte pression (10 à 15 bars). Une telle pression, à moins qu'elle soit disponible à l'entrée du réservoir, peut être créée par des pompes fixes ou mobiles. Si l'on n'a pas ajouté à l'eau de nettoyage un désinfectant chimique, il faut ensuite désinfecter par une solution de chlore à 2–3 mg/l.

Lorsque l'eau de rinçage ou de désinfection est rejetés à l'égout, il faut observer les taux de concentration suivants:

—pH 6,0 à 9,0.
—chlore résiduel 0,2 mg/l dans le drain d'évacuation.
2,0 mg/l dans l'égout.

4.5 Prévention des accidents

Un principe à observer dès l'étude de l'ouvrage est que l'ouvrage dans son ensemble et tous ses détails doivent être conçus de manière à exclure autant que possible tout risque d'accident pendant les travaux d'exploitation et d'entretien.

Cela vaut en particulier pour la conception des escaliers, échelles et plateformes, la disposition des fenêtres et des appareils d'éclairage comme pour l'équipement électrique et les appareils de levage de toute sorte. L'observation des règlements à ce sujet est surveillée en Allemagne Fédérale par les compagnies d'assurance des risques civils de l'exploitant et, en vertu d'une loi récente, par les inspecteurs de sécurité.

Ces règlements couvrent aussi toutes les mesures de prévention des accidents au personnel pendant les travaux d'exploitation.

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International standing committee on pollution and protection of water sources
Subject 1

Quality criteria for surface water to be treated to drinking water

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1 Introduction

When our Committee offered a proposal at the New York Congress on quality criteria for surface waters to be treated to drinking water, it became evident that there was no unanimous agreement on the subject. The Committee decided, therefore, to devote further study to it and a three point programme was drawn up:

(a) To make a survey of the situation in the different countries.

(b) To open a discussion with the different countries, from the standpoint of the water supply industry, with regard to surface water quality.

(c) To try to formulate practical recommendations which are acceptable to the Congress.

To get relevant information from the member countries of I.W.S.A. a questionnaire was compiled. The first part of the questionnaire dealt with drinking water standards and the second part with the situation pertaining to quality control of surface waters. The results of the questionnaire were presented at the Brighton Congress in the form of an interim report. It came out that the situation differs very much in different countries.

The legal situation on drinking water standards is the result of two basically different approaches. The first approach is that the law gives the legal background for action by stipulating that drinking water has to be proper from the health point of view and safe to drink. In the second approach the law indicates directly "Maximum Permissible Levels". There seems to be a tendency in the direction of the second approach. It became clear also that with regard to the required or desired quality from the technical point of view the standards or recommendations are considerably influenced by the situation in the different countries. The survey of the situation of quality control in surface waters revealed that this control exists in almost all countries. However, the legal situation varies considerably from country to country and is generally considered as inadequate to combat successfully the ever increasing pollution. Some countries have standards for surface water quality but these standards and their parameters show a lot of variation; other countries do not have these standards. In general three approaches can be observed:

(a) The decisions on discharge limitations are laid down as consents by Water Authorities, by Commissions or Water Courts.

(b) There are limits fixed for concentrations of certain substances in surface waters which must not be exceeded by the discharge of effluents.

(c) There are surface water quality criteria which are fixed with special conditions in mind.

Regarding these important differences in drinking water quality standards, surface water quality criteria and the quality control of surface water, it is necessary to work out the general background of water quality standardization in order to be able to come to general conclusions.

2 General background of standardisation

It can be said generally that the establishment of quality standards becomes a necessity in order to maintain a balanced relationship between creatures (man, animal, plant), or elements of social structure (for instance agriculture) and their environment at the moment that this balanced relationship is endangered, or in order to restore an already disturbed balance.

Such a disturbance of the balanced relationship between a creature and its environment results in a reduced general well-being of this creature. Put in other words this would mean a decline of its health, if health is defined as the state of physical, psychological and social well-being. Physical well-being is the absence of acute or chronic sickness. Psychological well-being is the state of being in harmony with the surroundings without tension or irritation. Social well-being is, for man, a certain state of harmony which is determined by the social environment which surrounds him in his activities.

A disturbance of the balanced relationship between an element of social structure and its surroundings happens if an environmental factor changes in such a way that it is no longer, or to a lesser extent, suited for its previous social purpose. (For instance a situation where surface water deteriorates in quality in such a way that it is no longer, or to a lesser extent, suited for agricultural irrigation).

From this it can be concluded that there is no need for regulating measures or the establishment of standards as long as there is a balanced and undisturbed relationship. The primitive man in a natural environment certainly does not need detailed and quantified standards.

A permanent disturbance of the equilibrium mainly happens due to the action of man. Such a disturbance does not usually become obvious until a certain limit is exceeded. At that moment a state of pollution has arisen. Action has to be taken, standards have to be given and the pollution has to be reduced.

This establishing of standards for a certain factor is always a very difficult problem. It can, in fact, only be done by government authorities which have to consider all aspects related to the factor. In addition they have to ensure that one particular aspect is not emphasized in such a way that the existing equilibrium of another aspect is disturbed.

This paper will furthermore deal only with the environmental factors of water. The optimum solution would be to ensure that a state of unacceptable pollution does not arise at all, but to determine beforehand the limits which have to be considered in order to avoid a disturbance of the equilibrium. On a scientifically sound and responsible basis these limits can only be determined by quantifying a dose-effect relation for each pollution parameter. In doing so, the difference has to be drawn between substances which are a part of a certain environmental factor and which are necessary for the equilibrium relation between a creature and its environment, and those substances which are foreign to this factor and which can therefore only be considered as harmful additions.

Minimum and maximum standards have to be fixed for the parameters which are necessary elements. Only maximum standards have to be fixed for harmful substances. An example of the latter are the biocides in water. The "ideal" standard for this group of substances is zero. However it has to be kept in mind that the continuous improvement of analysis methods renders the term zero quite unrealistic. The practice should therefore be oriented towards the realization of a zero emission policy into the surface water.

The maximum value for a standard is the threshold of effect level, i.e. the level at which a certain effect can be observed.

In establishing dose-effect relationships and limits, the following has to be kept in mind:

(a) long term effects have to be considered;

(b) antagonisms and/or synergisms between substances can influence their effects;

(c) limits have to consider the total stress on man, physically as well as psychologically;

(d) limits have to be fixed according to the needs of sensitive subgroups such as children, aged people, pregnant women etc.;

(e) the safety factors which have to be included in such limits must be determined in consideration of the knowledge, the effect of a substance and its development pattern, the methods of sampling and analysis.

In this way, specific limits for each substance have to be determined for an environmental factor in consideration of the different aspects which are inherent in this environmental factor (which in our case is surface water). It goes without saying that such limits have to be determined with the cooperation of all the sciences concerned.

The final standard for a parameter is fixed on the basis of various different limits which are determined in consideration of the various uses of the environmental factor concerned. This is a task for government authorities.

In summarizing, the following conclusions can be drawn:

(a) The formulation of standards is a very complex problem which is still at the beginning of its development;

(b) Depending on the circumstances, standards can be nationally, regionally or locally different;

(c) Standards are only valid for a certain period of time.

3 Dealing with the environmental factor, surface water

The formulation of standards for surface water quality has to be carried out in consideration of the various uses and destinations of this surface water. The most important aspects are:

—The water ecology: the presence of animals and plants in and on water in certain numbers and proportions which are typical for a certain type of surface water;

—The use as drinking water for wildlife;

—The use for irrigation in agriculture and for watering cattle;

—The use for recreation;

—The use for the preparation of drinking water.

It is the task of the water industry to cooperate in the last-mentioned use of surface water, to look for "ideal" standards and to formulate limits. These should be based as much as possible on dose-effect relationships. A lot of work still remains to be done in this field because there are still relatively few useful results on the necessary scientific levels.

A multidisciplinary approach is the only promising way of dealing with this problem. However, as long as scientific research lags behind in the production of the necessary information, most of the decisions have to be based on generally accepted data and experience.

Quality requirements for surface water to be treated to drinking water have to be fixed in consideration of quality requirements for drinking water itself. For this purpose the constituents in water can be divided into the two categories mentioned earlier.

Constituents which have to be present in water and constituents which are not part of a natural aquatic environment.

Both categories can only be considered in relation to the "Total Daily Intake". The following can be said about the first category. Physiological health only necessitates pure H_2O (sometimes and arbitrarily together with a small amount of iodine and/or fluoride). However, for health in the sense of general well-being, more stringent requirements have to be considered, e.g. temperature, turbidity, colour, taste and odour, salinity and hardness.

For this category, in principle two limits have to be fixed: a minimum and a maximum concentration.

To this category belong also constituents of water which are not directly related to the use of the water but which are important from the technical point of view. The transport of water, for instance, necessitates certain characteristics regarding oxygen concentration, pH and agressivity.

The second category consist of toxic substances, pathogens and non-toxic constituents of water which do not belong to the first category.

The next question concerns the removal characteristics of water treatment processes with regard to the different constituents and parameters of these categories.

In a number of constituents of the first category, water treatment processes will have hardly any effect or no effect at all. In these cases, the limits for surface water will have to be the same as for drinking water. For the rest of the constituents the limits for surface water will be determined by the limits in the drinking water together with the potential of the applied water treatment processes for removing them.

In the second category, extensive experience exists in the removal and inactivation of pathogens. There will be scarcely any problems in this direction if adequate unit operations are applied. For the non-toxic substances of this category, the same approach holds good as that outlined for the first category.

In the case of toxic substances there are significant differences in the removal capability and efficiency of water treatment processes. With these substances in particular a high degree of safety has to be maintained for the following reasons:

1. Knowledge of their effects is by no means sufficient—in particular for chronic effects.
2. Water cannot be replaced by any other product.
3. The consumer has no choice but to accept the water as it is.
4. There are no large reserves of water which can be taken out of circulation when it becomes obvious that something in the quality is wrong.

From the above it is clear that the evaluation of the capabilities and limitations of water treatment processes, and the safety factors that should be applied, can scarcely or not at all lead to unconditional results in an absolute sense.

The establishment of quality criteria for surface water to be treated to drinking water only makes sense therefore if the necessary restrictions are considered and if the other critical points are left to the knowledge and the decision of the water industry which has to prepare drinking water from a certain surface water. The waterworks are furthermore fully responsible for the quality of the distributed drinking water.

There are of course other uses of surface water which can lead to the formulation of limits for certain constituents in water. Depending on the use, these limits can be more or less stringent than the limits for surface water to be treated to drinking water. Finally, it is the task of the central government authority to formulate definitive standards on the basis of the various limits. However, economical and social aspects may be the reason for the most stringent limits not being realized. If such a situation actually occurs, it should be a temporary character only. The uppermost limit is in any case the "Basic Protection Level"; the dose of a substance which causes an effect which is still about acceptable. However the aim should be to reach continuously a stringently formulated "No Adverse Effect Level".

With these restrictions in mind a closer study will be dedicated next to:

(a) parameters of a toxic character;

(b) parameters of a non-toxic character, the concentration of which is not reduced by treatment processes;

(c) parameters of a non-toxic character, the concentration of which can be reduced by treatment processes.

4 Parameters of a toxic character

Some of these substances are eliminated to a large extent by the existing classical and/or advanced water treatment processes. Others are eliminated to a much lesser extent or hardly at all by commonly used water treatment processes.

Therefore, for surface water to be treated to drinking water, the lowest possible concentrations of these substances have to be aimed for.

Because of safety considerations, the starting point should be that these substances must not be present in surface water in higher concentrations than those considered permissible for drinking water.

This approach by no means ignores the removal capacities of conventional water treatment processes and their improvement by advanced treatment technologies which are still being refined.

On the contrary it is the task of the water industry to continuously improve and optimize the totality of water treatment processes.

However, in view of the fact that the importance of the absence of toxic substances in drinking water can hardly be quantified and that unconditional safety has to be considered, water treatment processes should be considered only as a safety factor. After all, at present there are many places where the concentrations of toxic substances in surface water do exceed the limits for drinking water. But even if at present these limits are not exceeded, or if the situation is improved so as to ensure that they will not be, it must be pointed out that these limits are continuously endangered. In particular in industrialized regions this may be due to incidental pollution or possibly accidents which are difficult to detect. In this case it is exclusively the role of the treatment processes to ensure that drinking water quality remains in accordance with the established standards. This is where the responsiblity of the waterworks lies. It is their duty to provide for a sufficient quantity of water of the necessary quality. Leaving aside economic considerations, this should be the only criterion for judging waterworks companies.

Using this approach, any procedure should be rejected where the permission for a waterworks company to use a certain surface water is, due to the extent of pollution of this surface water, directly combined with the requirement to use certain treatment techniques.

The principle that toxic constituents in surface water should not be present in higher concentrations than acceptable in drinking water means that the lowest possible concentration of these constituents in surface water should be aimed for. This can only be achieved by prohibiting this kind of discharge. However, because the realization of the zero-level—the ideal standard—can be considered next to impossible, one should aim at reaching the "no adverse effect level" with the chosen safety factor. This safety factor can be different and should be determined according to the quality of correlation of the dose-effect relationship. Because, in addition the possible effects of combinations of substances have to be considered, these safety factors should be generally on the high side.

4.1 Inorganic toxic substances

A compilation of the inorganic parameters which belong to this category of toxic substances is given in Table 1, together with the maximum permissible concentrations, which are based on the total toxic load of the population via a standard food-consumption pattern. The values given are based on generally known data as used in different countries or groups of countries such as the European Communities or cooperating water industries in different countries, e.g. the International Consortium on Water Supply Undertakings in the Rhine Catchment Area (I.A.W.R.) and the Union of the Water Supply Associations of Countries of the European Communities (Eureau).

So the given values are not absolute. They are one possible approach and will regularly necessitate correction because the total toxic load via the standard food pattern is not equal in all cases, and the food pattern itself can vary greatly from country to country. Also, as yet the various dose-effect relationships (short term and long term effects) are not always scientifically well investigated.

The real goal, that the concentration should be as low as possible—in principle non-detectable—should never be forgotten.

TABLE No. 1

MAXIMUM PERMISSIBLE CONCENTRATION OF INORGANIC SUBSTANCES WITH A TOXIC CHARACTER

Parameter	Unit	M.P.C. μg/l	Remarks
Silver	Ag	10	
Arsenic	As	50	
Boron	B	1 000	
Barium	Ba	100	
Beryllium	Be	0,2	
Cadmium	Cd	5	
Cyanide	Cn	50	
Chromium total	Cr	50	
Copper	Cu	50	
Fluoride	F	700–1 500	depending on water temp.
Mercury	Hg	1	
Nickel	Ni	50	
Nitrate	NO_3	50	
Lead	Pb	50	
Antimony	Sb	10	
Selenium	Se	10	
Zinc	Zn	100	

M.P.C.—Maximum permissible concentration.

4.2 Organic toxic substances

The category of toxic substances includes a number of persistent organic compounds as well as substances from which these compounds can be formed in water. These are mainly organic halogens, organic silica and phosphorus compounds, pesticides, organic tin compounds as well as substances which have a proven or suspected carcinogenic effect.

Opinions on the maximum permissible concentrations of these compounds and substances show much difference. The use of maximum permissible concentrations of some parameters for international standards remains questionable if it concerns substances with different characteristics.

Many of these substances are difficult to remove. **For the time being, the policy should be to avoid the discharge of these substances to surface water** or to reduce this kind of pollution within a specified time to the lowest possible level. An exception to this policy should be made only for those substances where it is certain that they are readily biologically degraded to harmless substances.

Further research on short- and long-term dose-effect relationships and the accumulation symptoms will be necessary to get a better knowledge of particular substances and groups of substances. This research will make possible the establishment of reliable Maximum Permissible Concentration values.

The efforts of the water industry in various countries should be concentrated on the realization of the above mentioned policy regarding this category of inorganic and organic substances as an integral and compulsory part of water quality management. This special emphasis is fully justified in view of the suitability of surface water for the preparation of drinking water.

5 Radioactivity

As far as the Maximum Permissible Levels of radioactivity in water are concerned, consideration should be given to the values given in the W.H.O. International Standards for Drinking Water (1971), which are based on the recommendations of the International Commission on Radiological Protection (I.C.R.P.). Special attention must be paid to tritium as this is not reduced by the purification process.

The standards should be used as a basis for water quality management and should be of a mandatory nature.

6 Parameters of a non-toxic nature

The substances dealt with in this category have no direct influence on public health but are of some importance from the organoleptic and aesthetic point of view or from the technical point of view. For instance corrosion and aftergrowth in the pipeline system are both influenced by water quality.

As already mentioned, it would not be realistic to fix international drinking water quality standards for these parameters because the possibilities and the approach to these quality parameters can vary considerably from place to place.

6.1 Substances not influenced by treatment processes

A part of the substances are formed by those which are not influenced by water treatment processes. So the concentration in drinking water is at least the same as in the surface water which is used as the raw water source. **And as no international drinking water standards can be fixed it is also therefore not possible to issue for this category of substances universally applicable numerical quality standards with a mandatory character for surface water to be treated to drinking water.** It is however possible to give advice in the form of guidelines in which are indicated those levels of the particular parameters which should—if possible—not be exceeded and also which levels by preference should not be exceeded. The majority of these values will be based on generally accepted facts and experience. Table 2 is based on this approach.

6.2 Substances Influenced by treatment processes

A certain number of the parameters of a non-toxic character are influenced by treatment processes. The allowable and desirable limits of these parameters in surface water to be treated to drinking water can be found by starting from the limits of the drinking water itself and adding to them the removal capabilities of the purification system.

Different treatment processes are possible, ranging from the well known classical systems such as: riverbank filtration, rapid- and slow sand filtration, coagulation, sedimentation, rapid filtration, up to the very complicated systems in which all the most advanced treatment processes are applied.

TABLE No. 2

RECOMMENDATIONS ON THE CONCENTRATION OF SUBSTANCES WHICH ARE NOT REDUCED BY PURIFICATION PROCESSES

Parameter	Unit	Surface water		Remarks
		Normal upper limit	Desirable upper limit	
Temperature	°C	25		12 could be regarded as optimum
Conductivity	μS/cm at 20°C	1 300	400	
Total hardness	meq/l	7		by preference > 2
Alkalinity	mg/l HCO_3^-			by preference > 120
Calcium	mg/l	100		by preference > 40
Magnesium	mg/l	50	30	
Sodium	mg/l	130	65	
Sulphates	mg/l SO_4^{--}	250	100	
Chloride	mg/l Cl^-	200	100	
Dissolved oxygen	% saturation			by preference > 70

The removal percentages of the latter ones are generally much higher than those of the classical systems. However, because of their complexity the chance of disturbance is much higher. This means that the safety of advanced systems is less than that of the long experienced classical processes and therefore it is questionable whether or not the water industry should be allowed to rely fully on these systems. For those parameters mentioned here it must be repeated that: no generally valid international criteria for surface water to be treated to drinking water can be set up. There will already be differences between drinking water criteria in different countries, depending upon possibilities and circumstances and this is also the case in the purification processes to be applied.

It is the responsibility of waterworks to select treatment processes and to organize technical management in such a way that safe and wholesome drinking water can be delivered in consideration of present possibilities and the structure of society. Waterworks are further responsible for ensuring that these conditions are fulfilled under all circumstances, which implies that possible risks in the treatment process and technical management have to be kept as low as possible. These circumstances, together with the skill and know-how which has to be present in the water industry, must form the first guarantee for a responsibly organized water supply.

But the quality of the surface water to be treated to drinking water must be such that this responsibility can be accepted.

Based on this approach, and considering the efficiency of certain water treatment processes, an indication can be given of the limits of quality that surface water should have for treatment to a normal, suitable drinking water compared to a drinking water with a "by preference" quality.

These indications are given in Table 3. The water treatment processes considered as as follows.

Conventional systems:

Method A (columns 5 and 6):

1. River bank filtration—simple post-treatment, or,
2. Rapid filtration—slow sand filtration.

Method B (columns 7 and 8):
Prechlorination—coagulation—sedimentation—rapid filtration.

Advanced systems:

Method C (columns 9 and 10):
Breakpoint chlorination—coagulation—sedimentation—ozonization—coagulation-supported rapid filtration—activated carbon filtration.

Method D (columns 11 and 12):
Rapid filtration—ozonization—powdered activated carbon—coagulation-supported rapid filtration—slow sand filtration.
All of these treatment processes are completed by post chlorination.

The requirement for a high degree of safety for the preparation of drinking water necessitates the water industry to stipulate with a high degree of urgency and emphasis that water quality management policy plans be drawn up in such a way that drinking water can be prepared from surface water by means of conventional methods.

Advanced treatment systems should—depending on individual approaches—be used for additional safety and necessarily during the period in which the surface water used has not yet attained a quality which makes its use possible for the preparation of drinking water by conventional water treatment systems.

The efforts of the water industry should be concentrated on drawing up and putting into action a water quality management policy plan which should be coordinated countrywide or per river catchment. International river catchments necessitate international water quality management policy plans which are enforced either by an international organization or which are laid down in a bi- or multilateral pact and enforced by the national authorities in each country.

TABLE No. 3

INDICATIVE LIMITS FOR PARAMETERS INFLUENCED BY TREATMENT PROCESSES

Parameter	Unit	Drinking water		Surface water to be treated to drinking water by conventional methods				Surface water to be treated to drinking water by advanced methods			
		normal upper limit	desirable upper limit	method A		method B		method C		method D	
				normal upper limit	desirable upper limit	normal upper limit	desirable upper limit	normal upper limit	desirable upper limit	normal upper limit	desirable upper limit
1	2	3	4	5	6	7	8	9	10	11	12
colour	mg/l Pt	20	5	25	6	40	10	100	50	50	25
odour	threshold-number at 250°C	1+2	odourless	1+9	1+1	1+3	odourless	1+99	1+24	1+49	1+14
oxidation	mg/l $KMnO_4$*	20	5	30	7	40	10	60	30	40	20
Mineral oil	$\mu g/l$	10	1	200	50	300	100	1 000	500	500	250
ammonia	mg/l NH_4	0,5	0,05	3	2	0,5	0,5	4	4	3	2
iron (total)	mg/l Fe	0,3	0,1	2	1	5	2,5	10	5	5	2,5
manganese	mg/l Mn	0,05	0,02	0,5	0,2	1,0	0,5	2	1	1	0,5
phosphorus (total)	mg/l P**				0,05		0,05		0,05		0,05

* Boiling for 10 minutes in acid medium.
** Figures are considered from the point of view of eutrophication of river impoundments.

Sujet 2

Protection des ressources en eau devant l'évolution énergétique

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1 Introduction

Au cours des discussions au sein de la Commission A.I.D.E. pour la protection et la pollution des ressources en eau, on a pu constater que les distributeurs d'eau potable sont préoccupés de la dégradation de la qualité des eaux souterraines ou de surface, due aux nombreuses séquelles de l'utilisation d'énergie.

Afin de voir plus clair dans les problèmes ainsi créés, la Commission a décidé de rédiger un rapport, sur la qualité des ressources en eau, traitant de l'influence de tous les aspects de l'utilisation de l'énergie sous toutes ses formes.

On a toutefois préféré, de ne pas prendre en considération dans le présent rapport, les deux aspects suivants: la pollution thermique et la pollution par les effluents des usines à gaz de charbon.

La pollution thermique, d'une importance majeure au droit des centrales thermiques, a fait l'objet d'un rapport présenté à un congrès précédent de l'A.I.D.E. (K. Haberer, New-York, 1972). L'influence d'effluents des usines à gaz est suffisamment connue de par la littérature.

Le rapport est de ce fait dressé en fonction des trois questions suivantes:

(1) jusqu'à quel point la production d'eau potable est-elle menacée par certains aspects de l'utilisation des différentes sources d'énergie?

(2) quelles sont les données existantes concernant les mécanismes de pollution et l'évolution dans le temps de l'image de pollution;

(3) quelles mesures ont été prises par les distributeurs d'eau, de même que par d'autres entreprises ou organismes publics, pour assurer la protection des eaux souterraines et de surface, destinées à la production d'eau potable, contre les formes de pollution liées à l'utilisation des sources d'énergie?

Une double enquête a été menée à l'aide de questionnaires, à la fois auprès d'un certain nombre de distributeurs d'eau, et auprès d'institutions spécialisées dans un des secteurs de l'énergie.

Du côté des distributeurs d'eau, des informations ont été reçues sur la situation en Afrique du Sud, Autriche, Belgique, Bulgarie, Danemark, Etats-Unis de l'Amérique, Finlande, Hongrie, Japon, Luxembourg, Pays-Bas, Portugal, Suisse.

Le fait qu'un nombre limité seulement des distributeurs d'eau semble disposer de données concrètes sur les problèmes considérés, indique que la plupart des producteurs d'eau potable ne sont pas encore persuadés des risques de pollution qui peuvent menacer la qualité des eaux, par suite de l'utilisation de l'énergie. Si, par sa nature incomplète, ce rapport attirera l'attention des responsables du secteur de la distribution d'eau potable sur cet aspect de leurs problèmes, il aura déjà prouvé son intérêt.

D'autre part, des informations ont été données par la Fondation CONCAWE à La Haye (Conservation of Clean Air and Water in Western Europe; secteur du pétrole), la Direction Protection Sanitaire du Directorat-Général des Affaires Sociales de la Commission de la Communauté Européenne à Luxembourg (EURATOM Secteur nucléaire), la Commission à l'Energie Atomique de France, à Paris et le Centre d'Etudes des Eaux à Medmenham et Stevenage en Grande-Bretagne (Water Research Centre). Les distributeurs d'eau veulent exprimer ici leur reconnaissance à tous ces organismes, qui ont eu l'obligeance de fournir toute information et documentation nécessaire à la rédaction du présent rapport.

Avant d'aborder les problèmes dus aux évolutions dans le secteur de l'énergie, il s'avère utile de donner une description sommaire des différents types de pollution possibles, causés par l'utilisation des différentes sources d'énergie. L'on indiquera pour chaque cas la nature de la ressource d'eau considérée (eaux de surface, eaux souterraines) et le caractère de la pollution (permanente, accidentelle).

A cette fin le schéma suivant sera respecté:

	A Production	B Transport	C Stockage	D Consommation
1. Charbon	1A	1B	1C	1D
2. Prod. pétroliers/gaz naturel	2A	2B	2C	2D
3. Combust. nucléaire	3A	3B	3C	3D

2 Possibilités d'influence sur les ressources en eau

1 Charbon

A Extraction

(a) *Eaux d'exhaure.* (Type: eaux de surface; permanente.)

L'extraction du charbon, aussi bien à ciel ouvert que par galeries profondes, libère des eaux souterraines dont la composition et la quantité sont étroitement dépendantes de la nature des couches géologiques susjacentes.

En République Fédérale Allemande, les venues d'eaux souterraines sont estimées à 1 m^3 par tonne de charbon extraite [1]. La teneur en chlorures peut aller jusqu'à 100 g/l. Le pH est dans ce cas plutôt alcalin. Ou bien l'eau est rendue acide par la présence de pierres sulfureuses (pyrites) dans les couches supérieures ou dans la veine de charbon elle-même. L'eau d'exhaure contient alors en même temps des sulfates, éventuellement des fluorures.

(b) *Eaux de percolation des terrils.* (Type: eaux de surface/eaux souterraines; permanente.)

L'eau de percolation des terrils est dépendante de la composition des résidus d'extraction souvent acide. Elle contient des sulfates, dus e.a. à la présence de pyrite.

(c) *Eaux de lavage du charbon*, (Type: eaux de surface; permanente.)

Les eaux de lavage contenant des particules de charbon augmentent la turbidité des cours d'eau où elles sont rejetées. Ceux-ci prennent alors parfois une couleur noire.

B Transport

(Type: eaux de surface; accidentelle.)

Lors d'accidents au cours du transport de charbon, des particules de charbon peuvent être déversées dans les eaux de surface et en augmenter la turbidité. Le risque de pollution maximale se présente lors du transport de poussiers de charbon par bateau.

C Consommation

(Type: eaux de surface; accidentelle.)

De la suie, produite par une combustion incomplète du charbon, ou des cendres légères peuvent se déposer sur des eaux de surface ou éventuellement sur de l'eau déjà partiellement traitée. Une centrale au charbon de 1 GWe par exemple peut expédier dans l'atmosphère 30 t de cendres par heure si aucun traitement des produits de combustion n'est appliqué [2].

2 Produits pétroliers et gaz naturel

A Extraction - Production

(a) *Opérations de forage*. (Type: eaux souterraines—accidentelle.)

Là où un forage traverse des couches aquifères, la pollution de l'eau souterraine par les hydrocarbures est possible (hydrocarbures lourds pour les exploitations pétrolières, méthane pour le gaz naturel). Au cas où aucune mesure n'est prise pour isoler le puits de la nappe, la pollution prend un caractère permanent [3].

(b) *Traitement du pétrole brut*. (Type: eaux de surface; permanente.)

La désémulsification et le dessalement engendrent des eaux résiduaires salées, riches en composants pétroliers, déversées après traitement dans des eaux de surface, ou dans des couches géologiques appropriées. La teneur en chlorures peut dépasser 100 g/l [1, 3].

(c) *Eaux résiduaires des raffineries*. (Type: eaux de surface; permanente.)

La composition des eaux résiduaires des raffineries est très dépendante des procédés de fabrication et des méthodes d'épuration utilisées. Dans les effluents non épurés on rencontre toujours de l'huile ainsi que des phénols, des composés azotés, des matières en suspension, des acides utilisés et parfois des cyanures et thiocyanates [1, 3, 4, 5].

L'effluent lui-même est composé en partie d'eau de refroidissement et en partie d'eau de traitement. Les sources principales du dernier type d'eau sont le dessalement (eau salée: 10 g/l de chlorures) et le cracking catalytique (condensats acides: 2 g/l et plus de phénols) [1, 6].

Des fuites aux tanks et aux canalisations peuvent influencer les eaux souterraines (voir plus loin).

B Transport

(a) *Transport conventionnel*. (Type: eaux de surface/eaux souterraines; accidentelle.)

Lors d'accidents au cours du transport, de grandes quantités de produits pétroliers peuvent être libérés et venir mettre en danger la qualité des eaux souterraines ou de surface. De nombreux accidents de circulation routière, de chemin de fer ou de navigation au cours desquels de l'essence, des huiles légères ou lourdes ont été répandues sont connus. Les accidents sur terre menacent en premier lieu les captages d'eau souterraine; les accidents de navigation concernent la qualité des eaux de surface. Citons un accident de chemin de fer en Suisse (Kloten 1974) au cours duquel 150 000 l de kerosène furent répandus (coût de la lutte; 1 million de francs suisses). Le rinçage non controlé des navires pétroliers est une source supplémentaire de pollution [7].

(b) *Chargement et déchargement*. (Type: eaux souterraines/eaux surface; accidentelle.)

Les chargements et les déchargements peuvent également être une source importante de pollution des eaux. Le débordement des tanks lors du remplissage, ou la rupture des tuyaux de déchargement sont ici des exemples typiques. La plupart de ces incidents sont attribuables à des défaillances humaines [3].

(c) *Oléoducs*. (Type: eaux souterraines; accidentelle.)

Des fuites ou des ruptures peuvent donner lieu, suivant la gravité de l'incident, à la dispersion dans le sous-sol de quantités plus ou moins considérables d'hydrocarbures. Vingt accidents de ce type ont été signalés en 1973 pour l'Europe Occidentale.

La plupart d'entre-eux était due à des phénomènes de corrosion extérieure. Un quart de ces accidents concernait les stations de pompage, si bien qu'il fut immédiatement porté remède aux défectuosités. Dans aucun cas le captage d'eau potable ne fut affecté [8].

Un cas est rapporté de pollution indirecte d'eau de surface. Il s'agit de la libération d'hydrocarbure en draguant des terres polluées par de l'essence provenant de fuites survenues à un oléoduc provisoire.

C Stockage

(a) *Réservoirs artificiels*. (Type: eaux souterraines; accidentelle.)

Des captages d'eau souterraine durent être provisoirement ou définitivement mis hors de service, par pollution due à des hydrocarbures en provenance de fuites à des tanks.

La plupart des accidents connus concerne des réservoirs enterrés à usage familial. Les réservoirs aériens contiennent, il est vrai, des quantités de produits pétroliers bien plus grandes mais ils sont de manière générale sous contrôle permanent [9].

(b) *Cavités souterraines*. (Type: eaux souterraines; permanente.)

Plusieurs pays envisagent le stockage de gaz naturel dans des cavités souterraines (mines désaffectées de charbon ou de sel) afin de faire face aux pointes de consommation. Bien que l'on n'aie aucune donnée concrète, on doit tenir compte de la possibilité de pollution des eaux souterraines.

D Consommation

(Type: eaux de surface; permanente.)

La combustion de produits pétroliers libère e.a. du dioxyde de soufre. La teneur en soufre des huiles légères ou lourdes s'élève respectivement à 0,3–1,5% en poids et 0,5–3,5% en poids, liée à la teneur en soufre du brut (0,08–5% en poids). Ainsi par exemple, la combustion de 1 t de pétrole à 2% de soufre libère 40 kg de SO_2 [3, 10].

On estime les rejets mondiaux de SO_2 à 100 Mt par an.

Les utilisations d'huile lourde (grosse industrie, centrales électriques, raffineries) sont donc responsables de la plus grande part de la pollution de l'air en SO_2. Une centrale électrique moderne peut libérer quelques centaines de tonnes de SO_2 par jour et par GWe [11].

Comme ces rejets s'effectuent à grande hauteur via des cheminées spéciales, leur influence peut se faire sentir très loin des lieux d'émission. C'est ainsi que l'on constate particulièrement en Scandinavie une augmentation du degré d'acidité et de la teneur en sulfate de l'eau de pluie. Cette évolution affecte également mais dans une mesure moindre l'Europe Centrale, mais pas la Grande Bretagne ni la Suisse par exemple. En raison de cette pollution, qui trouve probablement son origine dans la partie industrialisée de l'Europe Occidentale (région de la Ruhr) le pH des eaux de surface du Sud de la Norvège s'abaisse de 0,05 unité par an [12, 13, 14, 15].

Des phénomènes semblables ont été constatés aux Etats-Unis, où un pH de 3,0 a été mesuré dans l'eau de brouillard et des nuages [16].

La combustion des produits pétroliers rejette aussi dans l'atmosphère des micropolluants qui, via la pluie ou les chutes de neige, se retrouvent dans les eaux de surface. Ainsi sont connus l'émission du vanadium, présent dans les fuels sous forme de complexes de porphyrine, et du plomb, présent dans l'essence sous forme de tétraméthyl ou de tétraéthyl de plomb [7, 15, 17]. En raison de la circulation routière, il semble qu'environ 0,75 Mt de Pb sont émises dans l'hydrosphère chaque année, d'où depuis 30 ans une augmentation d'environ 1% par an de la teneur en plomb des chutes de neige aux pôles [13, 18].

En raison de la combustion incomplète dans les moteurs à essence, des hydrocarbures aliphatiques légers et surtout des aromatiques, se retrouvent également via l'air et les retombées comme micropolluants dans les eaux de surface et les sédiments. On estime qu'en Suisse chaque année 1000 à 10 000 t d'essence arrivent aux cours d'eau. Les moteurs diesel paraissent nettement moins polluants [3, 7, 19, 20, 21].

3 Combustibles nucléaires

A Production

(a) *Extraction et traitement du minerai.* (Type: eaux souterraines/surface; permanente.)

Au cours des manipulations dans les mines et les installations de broyage, l'uranium est concentré de 0,2% en moyenne jusqu'à 70 à 90% (exprimé en U_3O_8). Il en résulte une production de grandes quantités de déchets solides et liquides, qui contiennent principalement des produits de la déstintégration de l'uranium: radium et radon [22, 23]. On estime qu'aux Etats-Unis, jusqu'en 1970, se sont accumulées quelque 500 000 t de déchets solides d'une teneur en radium de 5300 Ci [22]. En Europe cela donnerait environ 300 000 t et 400 Ci [24].

Les eaux de percolation des terrils et les déchets liquides peuvent altérer fortement les eaux de surface et les eaux souterraines, si aucune décontamination n'est prévue.

Ainsi, la population de quelques villes du Colorado a été exposée, via l'eau potable, à une dose de rayonnement de ^{226}Ra qui était près de 2 fois supérieure à la valeur maximale recommandée par la Commission Internationale pour la Protection Radiologique (CIPR), jusqu'à ce que des mesures de décontamination soient prises [22].

Les effluents contiennent de plus des résidus des solvants utilisés au cours des processus d'extraction chimique (acides, bases . . .).

(b) *Raffinage.* (Type: eaux de surface; permanente.)

Les effluents des raffineries contiennent des restes de polluants chimiques (solvants, acides, fluorures, . . .) utilisés lors de la préparation de l'oxyde jaune (U_3O_8, "yellow cake") et des tetrafluorures (UF_4, "sel vert"). Vu que la plupart des éléments radioactifs de la série des uraniens (^{226}Ra e.a.) est en grande partie éliminée dans les broyeurs, la charge en radionucléides est d'importance moindre que la charge chimique [22].

(c) *Enrichissement.* (Type: eaux de surface—permanente.)

Vu la valeur économique de l'uranium enrichi, on trouve très peu de radionucléides dans les eaux résiduaires. Ces dernières contiennent e.a. des fluorures et des nitrates [22, 23].

(d) *Fabrication d'éléments de combustible.* (Type: eaux de surface; permanente.)

Des quantités minimes d'^{235}U provenant de UO_2, perdues au cours de la fabrication d'éléments de combustible, peuvent polluer les eaux de surface.

Des dégagements de PuO_2 ou de ThO_2 peuvent également se présenter au cours de la constitution des barreaux de combustible mixte.

B Stockage

(Type: eaux de surface/souterraine; accidentelle.)

Lors d'un hypothétique accident de criticité, des produits de fission peuvent, par lessivage de l'air et du sol atteindre les eaux de surface ou souterraines. La gravité de la pollution est fonction du degré d'enrichissement et du mode de conditionnement des produits fissiles (poudre, gainé) et des facteurs météorologiques (direction du vent, vitesse, précipitations).

C Consommation

(a) *Eaux résiduaires des centrales nucléaires.* (Type: eaux de surface; permanente.)

En même temps que les eaux de refroidissement, les centrales nucléaires rejettent des eaux résiduaires qui contiennent de très faibles quantités de radionucléides, produits directement ou indirectement par la fission nucléaire.

La nature et la quantité des matières rencontrées ne dépend pas tellement de la nature de la matière fissile, qui est le plus souvent de l'uranium enrichi ^{235}U, mais bien de la nature du modérateur, du refrigérant primaire produisant la vapeur, des adjuvants, du materiau constituant la cuve du réacteur, des méthodes d'épuration et de l'âge de la centrale même [25, 26, 27].

Les principaux types de réacteurs sont [28, 29]:

Abréviation	Combustible	Réfrigérant	Modérateur	
GCR	U Naturel	CO_2	Graphite	(*g*as *c*ooled)
AGR	U Enrichi	CO_2	Graphite	(*a*dvanced *g*as cooled)
LWR:				(*l*ight *w*ater)
BWR	U Enrichi	Eau bouillante		(*b*oiling *w*ater)
PWR	U Enrichi	Eau pressurisée		(*p*ressurised *w*ater)
HWR	U Naturel ou Enrichi	Eau lourde D_2O		(*h*eavy *w*ater)

Deux types de radionucléides sont formés dans le réacteur:

produits de fission et produits d'activation.

Les principaux produits de fission sont ^{3}H (tritium), ^{85}Kr, ^{90}Sr, ^{137}Cs, dont la demi-vie s'étend de 10 à 30 ans et ^{106}Pu et ^{147}Pm avec une demi-vie de 1 à 3 ans.

Parmi les produits d'activation, on rencontre ^{3}H, ^{54}Mn, ^{55}Fe et ^{60}Co.

Dans le procédé LWR, les produits du premier groupe, formés dans les barreaux de combustible sont diffusés dans le circuit primaire, en cas de fuites dans les gaines. Les produits du deuxième groupe sont formés au dehors des gaines, si bien qu'après prélèvement dans le circuit primaire et malgré l'épuration, des traces de produits des deux groupes peuvent se retrouver dans les effluents [24, 25].

Entre 1969 et 1972, les rejets radioactifs (tritium exclu) des centrales nucléaires des pays de la CEE (52% PWR, 25% BWR, 18% GCR et AGR) ont élevé la radioactivité moyenne des eaux de surface de 15 mCi/GWh.

Aux Etats-Unis les procédés BWR et PWR existent en nombre à peu près égal [26, 28, 30]. La teneur en tritium des effluents est relativement élevée surtout pour les centrales des types PWR et HWR.

AGR	400 Ci/an. GWe
PWR	1168 Ci/an. GWe
BWR	162 Ci/an. GWe
HWR	4000 Ci/an. GWe

Dans le procédé PWR l'existence de tritium est due principalement à l'activation des adjuvants anti-corrosion (réaction n, α du Li) et au contrôle chimique de l'activité (réaction n, 2α du ^{10}B [26, 31].

(b) *Pollution atmosphérique.* (Type: eaux de surface; permanente.)

L'eau de pluie peut lessiver une partie des rejets gazeux ou des aérosols de radionucléides en suspension et ainsi polluer les eaux de surface. Cela concerne principalement ^{85}Kr, ^{135}Xe et ^{131}I parmi lesquels seul ^{85}Kr a une longue demi-vie. Une centrale BWR libère de plus du ^{3}H et des traces de ^{60}Co, ^{89}Sr, ^{90}Sr, ^{134}Cs et ^{140}Ba sous forme d'aérosols [24, 25, 30, 32].

(c) *Transport d'éléments de combustible irradiés.* (Type: eaux de surface/souterraine; accidentelle.)

Les barreaux de combustible irradiés contenant principalement des transuraniens (émetteurs α de très longue demi-vie) et des produits de fission (émetteurs β et γ de longue demi-vie) sont transportés en conteneurs blindés par chemin de fer ou par camion vers les centres de retraitement. Un hypothétique accident grave pourrait avoir comme conséquence une pollution importante des eaux de surface ou des eaux souterraines [25, 33].

(d) *Effluents des centres de retraitement.* (Type: eaux de surface; permanente.)

Les eaux résiduaires de ces centres contiennent, à côté des restes de solvants organiques (tributylphosphate, kérosène, trilaurylphosphate) de grandes quantités de tritium et des traces de produits d'activation et de produits de fission, en particulier ^{106}Ru, ^{137}Cs et ^{90}Sr [24, 25, 33].

A la dissolution des noyaux des éléments de combustible dans l'acide nitrique une première partie (30%) du tritium présent se dégage et est évacué sous forme gazeuse avec le ^{85}Kr [31]. Après l'extraction du ^{239}Pu, la solution des produits de fission est concentrée par évaporation entrainant une seconde élimination de tritium (60%).

Les vapeurs condensées sont rejetées également. Les quantités de tritium à évacuer vers les eaux de surface sont donc d'environ 11 000 Ci/an, pour une centrale LWR de 1 GWe [31, 34].

(e) *Fuites aux réservoirs d'entreposage.* (Type: eaux souterraine; accidentelle.)

Les déchets liquides de haute activité sont stockés dans des réservoirs spéciaux en acier. En cas de fuite, les matières radioactives liquides peuvent pénétrer dans le sol d'une façon plus ou moins importante et ce en fonction directe des dispositifs de surveillance mis en place.

On constate alors que ^{137}Cs ^{134}Cs ^{90}Sr et ^{239}Pu migrent peu, tandis que ^{106}Ru pénètre plus rapidement [35].

3 Conséquences pour l'alimentation en eau potable

1 Quels sont les polluants pouvant provoquer des incommodités?

Le degré d'incommodité ressenti par les sociétés de distribution d'eau, face aux sources potentielles de pollution citées, dépend de la nature des polluants et de leur quantité, ainsi que de l'efficacité de leur système d'épuration.

(a) L'apport de matières en suspension dans l'atmosphère ne constitue, à vrai dire, aucun problème, le traitement des eaux de surface comprenant toujours un échelon d'épuration mécanique. Il en est de même pour les polluants qui, de préférence, s'adsorbent aux matières en suspension et sont entrainés par floculation ou par coprécipitation.

(b) Les sels et les radionucléides en solution traversent, sans être modifiés, les étapes d'épuration mécanique et biologique de manière que leur présence peut devenir particulièrement incommode. Leur élimination n'est possible que grâce à des techniques spéciales de dessalement.

(c) Les matières organiques (hydrocarbures, solvants, micropolluants) ne sont éliminées que partiellement lors de l'épuration biologique des eaux de surface. De petites quantités peuvent être éliminées à l'aide de techniques d'adsorption.

Particulièrement vulnérables à cette forme de pollution sont les eaux souterraines, qui généralement, ne font l'objet que d'une épuration sommaire (déferrisation, oxygénation, élimination de manganèse, . . .).

(d) Les hydrocarbures gazeux (gaz naturel) sont éliminés en grande partie, tant des eaux souterraines que des eaux de surface, au moyen du procédé d'aération.

Ce dernier requiert des mesures de protection spéciales.

(e) Le tritium, présent en tant qu'eau tritiée, HTO, ne peut être éliminé.

(f) Les acides ou les bases ne sont que partiellement neutralisés au moyen d'une épuration biologique. Un ajustement du pH est nécessaire.

Il ressort de ces constatations que les substances mentionnées ci-dessous, introduites potentiellement ou effectivement dans les ressources en eau à la suite de l'utilisation des sources d'énergie, sont à considérer comme étant nuisibles à la qualité de l'eau.

Polluants	Origine	Incommodité
sels	mines de charbon, épuration de pétrole brut, production de combustible nucléaire;	goût, corrosion
acides, bases	mines de charbon, terrils, production et retraitement de combustibles nucléaires, gaz de combustion de pétrole;	corrosion, perturbation de l'épuration
matières organiques	extraction et épuration du pétrole brut, raffinage, accidents de transport, fuites aux réservoirs, moteurs à essence, production et retraitement de combustibles nucléaires;	goût, odeur, incommodité mécanique, toxicité
hydrocarbures gazeux	extraction de gaz naturel	danger d'explosion
radionucléides	rayons α: production de combustibles nucléaires; rayons β et γ: centrales nucléaires et unités de retraitement.	radiotoxicité

Ces matières sont rejetées dans les eaux de surface soit de façon accidentelle, soit de façon permanente. Les eaux usées n'étant que rarement évacuées dans le sol, on peut admettre que les eaux souterraines ne sont menacées que par la pollution provoquée par les eaux de pluie (lessivage et percolation) ou par les accidents.

Comme sources polluantes il y a lieu de citer respectivement la combustion et le transport ou stockage de produits pétroliers.

Afin de déterminer lesquels de ces groupes de polluants sont susceptible de présenter le plus grand danger pour l'alimentation en eau dans son ensemble, il y a lieu de disposer de données quantitatives concernant les effluents ou les quantités libérées en cas d'accident.

De semblables données sont relativement bien connues pour ce qui concerne le secteur nucléaire, celui-ci faisant l'objet d'un contrôle très intense [26, 30]. D'autre part pour le secteur pétrolier, il est difficile d'obtenir des chiffres tant au sujet des effluents de raffineries qu'au sujet d'accidents.

Il faut remarquer que les quantités de matières usées évacuées par des unités de production semblables, peuvent différer fortement suivant l'état de vétusté des installations et les méthodes d'épuration appliquées [4, 26].

La possibilité que des matières en provenance du secteur de l'énergie soient trouvées dans le sol ou dans les eaux de surface en un endroit déterminé, peut être dégagée d'une analyse géographique de la région concernée, analyse comprenant, par type de combustible, la localisation d'unités de production, des axes de transport, d'aires de stockage et d'unités de consommation, ainsi que la détermination de caractéristiques hydrologiques, hydrographiques, et hydrogéologiques de la région.

2 Principales sources de polluants incommodes

(a) *Sels.* (Eaux de surface.)

Les mines de charbon et les champs pétroliers produisent des quantités d'eau salée en quantités comparables [1].

Selon nous, les évacuations par les mines de charbon présentent plus d'intérêt pour l'alimentation en eau potable, étant donné que généralement l'activité industrielle concernée se déroule dans des régions fortement peuplées.

(b) *Acides ou bases.* (Eaux de surface, eaux souterraines.)

Les principales sources semblent être les mines de charbon et l'eau de percolation de terrils.

L'influence des eaux de pluie polluées (SO_2) sur le pH se limite aux eaux de faible capacité tampon [14]. Les acides et bases d'une autre provenance sont généralement neutralisés avant évacuation.

(c) *Matières organiques.* (Eaux de surface, eaux souterraines.)

Bien que beaucoup d'hydrocarbures soient libérés lors de l'extraction, de l'épuration et du raffinage, les quantités rejetées sont plutôt minimes, grâce à l'épuration très poussée des eaux usées [1, 4]. Les pollutions accidentelles ainsi que les risques qu'elles présentent sont plus importantes. Elles peuvent se produire au cours du transport, lors du transbordement ou du stockage, aussi bien chez le producteur, que chez un intermédiaire ou chez le consommateur.

Les possibilités de pollution dépendent, dans une large mesure, de l'expérience en matière de manipulation d'hydrocarbures et du contrôle exercé sur les installations. Ce dernier décroît au fur et à mesure que l'on approche du consommateur.

Cela signifie que le nombre de sources potentielles de pollution est fort élevé [5, 36]. Rien que pour la Suisse le nombre d'incidents dus au pétrole est estimé entre 800 à 1500 cas par an.

La gravité de la pollution éventuelle est fonction de la fonction pétrolière intervenant dans l'accident, de la proximité d'eau de surface ou d'eaux souterraines, de la nature du cours d'eau et des mesures prises pour la protection des eaux [37].

Les quantités d'hydrocarbures qui, par les eaux de pluie se mélangent aux eaux de surface, sont, pour ainsi dire, négligeables [19, 20]. La pollution par des solvants organiques utilisés lors de la production et du retraitement de combustibles nucléaires ne présente également qu'un intérêt secondaire [22].

(d) *Radionucléides.*

Sans mesures de décontamination, les effluents des installations de broyage de minerai, contiennent assez bien de ^{226}Ra, ce qui peut rendre le cours d'eau récepteur inapte à la production d'eau potable [22, 23].

Chaque année, les centrales nucléaires du type LWR et les unités de retraitement associées évacuent dans les eaux de surface par GWe, jusqu'à 12 000 Ci de tritium et quelques dizaines de Ci d'autres produits d'activation et de fission [26, 31, 33].

Les entreprises de production de combustible sont plus nombreuses que les unités de consommation; ceci peut être illustré par l'exemple des Etats-Unis aux environs de 1970 [22, 23, 30, 33]:

mines:	unités d'enrichissement:	centrale nucléaires:	centres de retraitement
150	3	15	1

L'importance que représentent pour l'alimentation en eau potable les centrales nucléaires et les unités de retraitement est cependant plus grande que celle des installations de broyage, ces dernières étant situées dans des régions plus retirées.

En résumé, il peut être établi que l'influence du secteur de l'énergie, est exercée principalement par:

—des pollutions accidentelles (apport d'hydrocarbures);

—des mines de charbon (apport d'acides et de sels);

—des centrales nucléaires et des unités de retraitement (apport de radionucléides).

3 Expérience des sociétés de distribution d'eau

Pour la plupart des sociétés de distribution d'eau (90%), ayant bien voulu donner des informations, les hydrocarbures constituent la nuisance principale.

Dans la plupart des cas (80%) la pollution accidentelle lors du transport ou du stockage est évoquée comme source principale de pollution; ceci concerne à la fois les captages d'eaux souterraines et les captages d'eau de surface.

Quelques sociétés attirent l'attention sur l'infiltration d'hydrocarbures dans le sol à proximité de raffineries de pétrole, ou sur le lessivage d'hydrocarbures contenus dans les gaz de combustion.

Les eaux salées ou acides, en provenance de mines de charbon et de terrils, sont estimées par 10% des sociétés comme étant la nuisance la plus importante. 5% seulement estiment que c'est le cas pour les eaux usées de centrales nucléaires.

Il est à noter que l'attention accordée à ces sources de pollution est plus particulière dans les pays où l'énergie nucléaire n'est pas encore fortement développée, que dans ceux possédant déjà une certaine tradition dans ce domaine et où cette forme de pollution est considérée comme étant la moins importante.

Compte tenu de ce qui précède, il est compréhensible que ce fut surtout la pollution par les hydrocarbures et les risques qu'elle engendre qui ont influencé l'exploitation des installations d'épuration.

Les conséquences en sont:

—la mise hors de service temporaire, rarement définitive, d'un captage d'eau;

—un traitement supplémentaire (charbon actif) ou intensifié, ce qui entraine une augmentation des charges financières;

—la construction de réservoirs de sécurité (eaux de surface) et la création de captages de réserve (eaux souterraines);

—l'augmentation du coût d'exploitation suite au contrôle plus intense, à l'accroissement de la main d'oeuvre, etc.

C'est surtout l'utilisation du charbon actif pour l'épuration des eaux qui constitue un élément frappant.

Ainsi, en Suisse, diverses sociétés de distribution d'eau prélevant leurs eaux brutes dans le Lac de Constance, ont installé ou prévu, suite à l'augmentation des risques de pollution par hydrocarbures, apparus lors de la pose d'une conduite de transport le long et à l'intérieur du lac, des filtres à charbon actif.

4 La protection des ressources aquifères

1 Généralités

A Eaux de surface

Dans la plupart des pays, les évacuations dans les eaux de surface sont régies par des lois sur la protection des eaux en général ou des eaux de surface en particulier.

Souvent, ces lois prévoient un système d'autorisation et de prélèvement financier sur base des quantités de matières rejetées; d'autre part l'Etat encourage la construction d'installations d'épuration industrielles par l'octroi de subsides, subordonnés au respect de certaines conditions.

Ces actions sont coordonnées par bassin hydrographique ou par unité administrative, par des institutions spécialisées subordonnées à l'autorité ou fonctionnant de façon autonome. De telles organisations sont développées particulièrement dans les régions industrialisées.

Des accords bilatéraux ou multilatéraux sont conclus afin de limiter la pollution de cours d'eau internationaux.

Le plus souvent, le contrôle de la pollution permanente des eaux est organisé par l'autorité dans le cadre de la législation dont question ci-avant. A défaut, les sociétés de distribution d'eau assurent elles-mêmes le contrôle continu ou périodique afin d'être informées sur l'état des eaux qu'elles prélèvent.

B Les eaux souterraines

En général, la protection des eaux souterraines est moins clairement réglementée que celle des eaux de surface. Un principe qui est généralement d'application, consiste à déterminer des zones de protection autour des captages. A l'intérieur de ces zones, l'établissement de certaines entreprises ainsi que le stockage de produits dangereux, sont interdits, limités ou réglementés [38].

Dans les pays ou la création de ces zones et le contrôle sur ces dernières ne sont pas réglementés par une loi, les sociétés de distribution d'eau prennent, soit collectivement, soit individuellement, des initiatives dans ce sens.

Ce travail est évidemment facilité lorsque les sociétés de distribution d'eau sont elles-mêmes des organismes publics ou semi-publics.

C Les eaux de pluie

Afin de limiter la pollution atmosphérique, conduisant à l'acidification des eaux de surface, la teneur en soufre de fractions de pétrole, fait l'objet de certaines restrictions [39].

2 Produits pétroliers

A Prévention

La plupart des pollutions effectives ou potentielles de captages d'eau par des hydrocarbures résultent d'évacuations accidentelles.

Certains pays possèdent une législation spécifique dont le but consiste à éviter ou à limiter les dégâts ainsi occasionnés. Des dispositions légales ou réglementaires prévoient des prescriptions techniques concernant la construction, l'entretien et le contrôle des réservoirs d'hydrocarbures, de trop-plein, de systèmes de détection de fuites, et des oléoducs [39].

A l'intérieur des zones de protection, des restrictions ou des réglementations strictes sont imposées en matières de dispositifs de détection.

Des mesures à caractère technique, entrainant une réduction des gaspillages au sein des entreprises, sont prises par l'industrie pétrolière. Des techniques d'épuration des eaux usées ont fait l'objet d'amélioration sensibles, de façon que la teneur en hydrocarbures des effluents dans les raffineries récemment construites, s'élève à moins de 5 mg/l [4, 6].

La collecte des huiles usées, rendue obligatoire dans certains pays pour les bateaux et leur réutilisation ont entrainés une amélioration sensible de la teneur en hydrocarbures de certaines eaux de surface (Le Rhin) [9, 40].

B Contrôle, alerte, intervention

La constatation des pollutions accidentelles s'effectue de manière organisée, tant par les sociétés de distribution d'eau, que par les organismes publics. L'organisation des interventions peut être très différente d'un pays à l'autre:

—ou bien, les sociétés de distribution d'eau s'occupent elles-mêmes des interventions et l'Etat n'intervient que dans des circonstances extrêmes;

—ou bien les alertes sont centralisées par un service spécialement équipé à cet effet.

Ce dernier avertit ensuite les sociétés de distribution concernées et organise l'intervention. Le plus souvent, il est fait appel aux pompiers de la région on à la protection civile; il existe rarement des brigades anti-huile.

Il convient ici de souligner les recherches scientifiques et techniques subsidiées par l'Etat ou par le secteur pétrolier (American Petroleum Institute, Fondation CONCAWE), qui ont abouti à une notion assez précise du procédé d'épuration du sol et de l'eau et qui ont permis l'élaboration de méthodes d'élimination adéquates [38, 41, 42].

3 Radionucléides

Grâce à un contrôle très poussé, les pollutions accidentelles par des nucléides en provenance de centrales nucléaires sont peu probables.

Si toutefois elles se manifestent, elles sont limitées en envergure et en durée. L'intervention a lieu suivant un plan de secours préétabli qui prévoit l'assistance d'autorités locales et régionales ainsi que de la protection civile [28, 43].

Les rejets permanents de centrales nucléaires et d'unités de retraitement doivent en tout temps répondre aux lois sur la protection contre les rayonnements, lois qui sont basées sur les recommandations de la Commission Internationale pour la Protection Radiologique (CIPR).

Dans le cadre des lois générales sur la pollution des eaux de surface, les déversements sont soumis à un régime d'autorisation. Lors de la détermination des conditions de déversement, les autorités peuvent procéder de trois manières:

—imposer des restrictions qui seront d'application générale;

—examiner chaque demande séparément;

—imposer des restrictions sans mentionner de limites bien définies.

Ces deux premières méthodes de travail nécessitent une étude radio-écologique approfondie; dans le premier cas pour déterminer, au départ d'une dose de rayonnement maximale admissible, les limites à imposer dans les cas de décharges isolées; dans le second cas pour savoir jusqu'à quel point une décharge déterminée augmente le degré d'exposition de la population aux radiations [44].

A chaque occasion, il y a lieu de déterminer les radionucléides critiques, les voies critiques de cheminement et les groupes critiques de la population.

Une telle étude se rapporte aux aspects démographiques, hydrologiques et climatologiques du site d'implantation, au pouvoir d'autoépuration des cours d'eau récepteurs et des installations d'épuration servant à la production d'eau potable.

Pour illustrer la première méthode de travail, il y a lieu de mentionner qu'en République Fédérale Allemande, on désire limiter la dose génétique de radiation reçue par la population suite à l'exposition via l'eau potable à des radionucléides en provenance des déversements des centrales nucléaires, à 0,3 rem en 30 ans ou à 10 mrem par an; 10% de ceux-ci peuvent être attribués au tritium. Ainsi, il est possible de proposer pour la teneur en tritium du Rhin une limite de 10 000 pCi/l [45, 46]. Principalement sous la pression de l'opinion publique, des mesures de sécurité et de contrôle ont été prises telles que la dose de radiations provenant d'une seule centrale nucléaire et reçue via l'eau potable, est inférieure à 0,5 % de la dose naturelle. Cette dose naturelle varie de 50 à 500 mrem pour les pays de la CEE avec une moyenne de 100 mrem [26, 47, 48].

La dose génétique maximale recommandée par CIPR s'élève à 5 rem par période de 30 ans.

La radio-écologie et la radiologie font l'objet d'études approfondies, effectuées et coordonnées par des institutions nationales spécialisées, telles que des commissions ou des autorités pour l'énergie atomique, des centres d'étude pour l'énergie nucléaire et des instituts pour la protection contre les radiations, et encouragées par des institutions internationales comme l'Organisation de Coopération et de Développement Economique, l'Euratom, l'International Atomic Energy Agency.

5 Évolution

Des données directes au sujet de l'évolution des types de pollution dont question ci-avant, sont rares. Il est souvent fort difficile de comparer des statistiques concernant des pollutions accidentelles. Toutefois, on admet qu'elles suivent de fort près l'évolution du secteur de l'énergie. On constate les mutations suivantes:

—besoins domestiques: gaz de houille→gaz naturel;

—chauffage familial: charbon→pétrole→gaz naturel;

—industrie: charbon→pétrole;

—production d'électricité: charbon→pétrole→énergie nucléaire.

1 Gaz de houille→gaz naturel (besoins domestiques)

Bien qu'il ne soit pas question du gaz de houille dans cette étude, il est important de souligner la conversion, puisqu'elle a entrainé une nette amélioration dans le domaine de la pollution de certaines eaux de surface par les phénols et produits à base de goudron de houille, p.e. en Hongrie.

2 Charbon→pétrole (chauffage familial, industrie, électricité)

A Production de charbon

L'utilisation du mazout en remplacement du charbon pour le chauffage familial est la cause principale de la diminution de la production de charbon dans les régions à forte densité de population. Ainsi, dans le bassin houiller campinois (Belgique) une quantité de 10 Mt était encore produite en 1955, alors qu'en 1975, la production ne s'élevait plus qu'à 6 Mt. Il est prévu que la quote-part du charbon en tant que source d'énergie pour l'alimentation de l'Europe Occidentale passera de 25 à 7% au cours de la période 1970–1985 [49].

Rien déjà que pour l'Allemagne de l'Ouest, cette quote-part sera ramenée de 76 à 4% entre 1950 et l'an 2000 [50].

En chiffres absolus et compte tenu de la consommation d'énergie croissante, cela représente une réduction de moitié de la production charbonnière.

Ce phénomène ne constitue pas une règle générale: aux Etats-Unis, par exemple, on prévoit entre 1970 et 1985 un accroissement de la capacité de production d'environ 25% [51]. Sur le plan mondial l'on prévoit une croissance avec apogée entre les années 2100 et 2200 [52].

Il s'ensuit que la pollution des eaux de surface par les eaux d'exhaure salées ou acides diminuera dans certaines parties du monde (Europe Occidentale). Globalement on ne doit pas s'attendre à une amélioration.

B Produits pétroliers

Le remplacement du charbon par les produits pétroliers, tant dans le secteur de l'industrie que dans celui des besoins domestiques, entraine indubitablement un risque croissant de pollution accidentelle.

Ainsi, par exemple, le tonnage d'hydrocarbures transporté sur le canal Albert, voie navigable la plus fréquentée en Belgique, a été multiplié par 8 entre 1954 et 1973, passant de 0,5 à 4 Mt par an. Les déchargements le long de ce même canal sont passés de 0,1 Mt en 1954 à 1,4 Mt en 1974 [53].

Bien que des chiffres précis ne soient pas connus, il peut être supposé que la fréquence des pollutions par hydrocarbures a augmenté dans une même proportion.

Au fur et à mesure de l'évolution de la conversion, le nombre de réservoirs souterrains à usage privé a également augmenté. Les risques de pollution accidentelle des eaux souterraines par des fuites deviennent d'autant plus grands que la durée d'utilisation des réservoirs est longue.

A partir de 30 années d'utilisation, les réservoirs présentent d'importants signes de corrosion [52].

En même temps que l'accroissement de la consommation d'hydrocarbures à des fins privées ou industrielles, qui atteindra son point culminant vers les années 1990–2000, on peut s'attendre à de nouvelles augmentations des risques.

C Pollution atmosphérique

Bien qu'à l'heure actuelle, ce type de pollution présente une importance moindre, il y a lieu d'attirer l'attention sur l'accroissement du degré d'acidité de l'eau de pluie résultant du dégagement de SO_2 lors de la combustion du mazout; ceci influence surtout les eaux de surface d'une faible capacité tampon.

3 Pétrole → gaz naturel (chauffage familial)

C'est principalement en Europe de l'Ouest qu'un accroissement de la consommation de gaz naturel est enrégistré [49]. Pour le chauffage familial beaucoup de particuliers passent du mazout au gaz naturel, surtout en raison d'impératifs économiques. Cela représente une diminution des risques de pollution accidentelle par des produits pétroliers, à condition que le réservoir à mazout soit vidé lors de la conversion.

4 Pétrole → énergie nucléaire (alimentation en électricité)

Pour le monde entier, on prévoit qu'entre 1970 et 1985, la quote-part de l'énergie nucléaire dans la production d'électricité passera de 3% à 27%. Cette quote-part s'élèverait en 1985 à 807 GWe et en l'an 2000 à 4257 GWe, soit à 63% [29, 49].

Compte tenu de la technologie actuelle en matière de réacteurs nucléaires basés sur la fission, l'on peut s'attendre en ordre principal à ce que la teneur en tritium de certaines eaux de surface augmente considérablement [26].

Pour la RFA, il a été calculé sur base d'une répartition uniforme de la charge de tritium entre tous les cours d'eau, que la teneur dans l'eau de surface augmenterait d'environ 100 pCi/l en 1970 à 10 000 pCi/l en l'an 2000 [31].

Bien que cet élément soit peu radiotoxique et que cette teneur soit de loin inférieure à celle admise par la CIPR pour l'eau potable (3×10^6 pCi/l), cette évolution ne manque pas de provoquer surtout parmi le public, quelque inquiétude. En voici les raisons:

- certaines questions relatives au métabolisme du tritium restent sans réponse;
- le tritium ne peut être éliminé par des techniques d'épuration conventionnelles.

Il est préférable d'établir les unités de retraitement libérant beaucoup de tritium, le long des côtes, où les possibilités de dilution sont plus grandes que dans les cours d'eau.

La fusion nucléaire résoudrait en grande partie ce problème mais l'on admet que des réacteurs de ce type ne seront pas opérationnels avant l'an 2000 [29].

D'une façon générale, il peut être établi que pour la production d'électricité, le passage du pétrole aux combustibles nucléaires influencera dans un sens favorable la pollution des ressources en eau.

6 Conclusions

Comme déjà indiqué dans l'Introduction, la pollution thermique, quoi que très importante au droit de centrales thermiques et surtout au droit de centrales nucléaires, n'a pas été prise en considération dans ces conclusions. Cela ne signifie nullement que son importance serait sousestimée: bien au contraire, le déversement d'énergie sous forme de chaleur est considérée comme une des conséquences les plus importantes de l'utilisation des sources d'énergie pour la qualité des eaux, d'autant plus que, à cause de la tendance à augmenter la puissance installée sur un seul site d'implantation, l'évacuation de chaleur se fait de plus en plus de façon concentrée. On estime que la production d'électricité provoque actuellement la décharge sur les eaux de surface et de mer sur le plan mondial de 10^{19} kcal par an. Cette décharge double tous les 9 ans [54].

1. Des réponses des distributeurs d'eau au questionnaire résulte que les formes possibles de pollution, liées à l'évolution énergétique, peuvent être citées dans l'ordre d'importance décroissante suivante:
   (a) La pollution accidentelle par des produits pétroliers.
   (b) La pollution des eaux de surface par les eaux acides ou salées d'exhaure ou de percolation de terrils.
   (c) La pollution des eaux de surface par des déchets de centrales nucléaires et d'unités de retraitement.

2. La pollution accidentelle par des produits pétroliers semble de loin être la plus répandue et celle qui représente le plus d'inconvénients. Les causes de ce phénomène sont les accidents de transport (par bateau, camion, train, oléoduc), pertes au chargement et déchargement, les fuites de réservoirs (souterrains ou aériens). Il y a lieu de signaler spécialement les risques dus à la corrosion de vieux réservoirs souterrains désaffectés et d'attirer l'attention sur les risques créés par le stockage de gaz naturel ou de produits pétroliers dans des cavités souterraines.

3. Les besoins croissants en énergie entraineront une augmentation globale des risques de pollution des eaux par les produits pétroliers dans les années à venir. Bien que l'évolution de la technologie et la protection de l'environnement diminueront l'accroissement des risques globaux, les risques d'accidents dus à la défaillance humaine, ne peuvent être écartés.

4. Une attention particulière doit être consacrée à l'accroissement de l'acidité de l'eau de pluie suite à la combustion de fractions pétrolières contenant du soufre.

   Dans certaines régions cette forme de pollution atteint des proportions considérables.

5. Il est recommandé de prévoir des directives précises pour les interventions dans les cas de pollution accidentelle par hydrocarbures. Là où les risques sont importants, ces directives doivent, de préférence, être élaborées en collaboration avec les autorités ou corps d'intervention locaux (service des pompiers, protection civile).

6. Des cas de pollution des eaux par l'exploitation minière sont signalés moins souvent. Dans certaines parties du monde, e.a. en Europe Occidentale, on prévoit une diminution de cette forme de pollution dans les années à venir. Cependant, sur le plan mondial on ne s'attend pas à une amélioration.

7. Si l'on ne tient pas compte de la pollution thermique, l'introduction d'énergie nucléaire pour la production d'électricité diminue le degré de pollution des eaux: grâce surtout aux mesures sévères de sécurité, une pollution grave est pratiquement exclue. Les teneurs en radionucléides des eaux de surface, dues aux rejets des centrales nucléaires et des unités de retraitement ne constituent qu'une fraction des teneurs maximales recommandées pour l'eau potable. Toutefois, ici également la pollution deviendra plus importante.

8. Bien que la pollution des eaux par des hydrocarbures soit ressentie comme très nuisible par les distributeurs d'eau, ceux-ci ne disposent que de très peu de données concrètes au sujet de l'évolution de cette forme de pollution. Ainsi, un contrôle régulier de la présence de ces polluents dans les eaux de surface ou souterraines à traiter parrait logique et nécessaire. La même remarque s'applique à la pollution par les radionucléides.

9. Une coopération étroite et une bonne intelligence entre le secteur de l'énergie et les distributeurs d'eau potable sont fortement recommandées, à fin de prévenir les pollutions permanentes, et de diminuer, voir même d'éviter les inconvénients dus à des pollutions accidentelles.

10. Chaque production conventionnelle d'énergie (y compris l'énergie nucléaire) se fait au dépens de la qualité des ressources en eau. Ainsi, toute mesure visant à limiter la consommation d'énergie, conduit forcément à une amélioration de la qualité des eaux. C'est dans cet esprit qu'on peut affirmer que la lutte contre le gaspillage d'énergie représente, à côté d'une importance économique, également un intérêt tout particulier pour la protection des ressources en eau.

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Summary

The report is a synthesis of the data acquired from water supply undertakings in fourteen countries on one hand and specialised institutions in one or another energy branch on the other hand.

It deals with the influence of different aspects of the production, transport, storage and consumption of coal, petroleum products, natural gas and nuclear fuels on the quality of water resources. Thermal pollution and pollution through coal gas plant effluents have not been considered.

Consideration of the information collected has resulted in the following conclusions being drawn:

1. The main types of pollution which occur as a consequence of energy consumption are (in order of decreasing importance):
   (a) Accidental pollution with petroleum products.
   (b) Pollution of surface water with acid or salt-mine water or water leaching from slag heaps.
   (c) Surface water pollution by nuclear power plant wastes and reprocessing units of nuclear fuels.
2. Accidental pollution by petroleum products seems by far the most widespread and causes most difficulties. It is caused by accidents during transport (by water, overland, pipelines), losses during loading and unloading and leaks in reservoirs (overground or underground). Special attention has to be drawn to the risk due to corrosion of underground reservoirs which have been taken out of service. The risks associated with underground storage of natural gas and petroleum should not be underestimated.
3. Ever growing energy needs will increase the global risk of water pollution by petroleum products in the near future. Technological progress and the growing environmental consciousness of the population will slow down the increase of this risk, but on the other hand the risk of an accident through human failure will continue to exist.
4. Special attention should be paid to combustion of sulphur-containing petroleum fractions. In certain regions this type of pollution reaches serious proportions.

5. More precise directives should be worked out for dealing with cases of accidental pollution. When the risk is great action should be organised in cooperation with local authorities or the emergency services (fire-brigades, civil protection).

6. Cases of water pollution due to mining are less frequently mentioned. This kind of pollution will decrease strongly in certain parts of the world, Western Europe a.o. On a world scale no general improvement may be expected.

7. If thermal pollution is not taken into account, the shift to nuclear energy for power supply will result in a decrease of water pollution, especially since strict security measures exclude heavy pollution almost completely. The radionuclide content of surface waters due to nuclear power stations and reprocessing unit effluents is only a fraction of the recommended limits for drinking water. Nevertheless pollution in this field will increase.

8. In spite of the fact that pollution of raw water is considered the greatest hindrance in drinking water production, water supply organisations have relatively concrete data at their disposal about the evolution of this type of pollution. As a consequence, it seems logical and necessary to check the raw water regularly for the presence of these pollutants. The same applies to radionuclide pollution.

9. Close cooperation and good understanding between the energy sector and water supply undertakings has to be recommended strongly in order to prevent permanent pollution and to avoid or to decrease the serious consequences of accidental pollution.

10. Each form of conventional energy production (nuclear energy included) always takes place at the cost of the quality of water resources. Each step in the direction of the limitation of energy consumption has to result automatically in an improvement of water quality. In this spirit, fighting energy dissipation is valuable as water resources quality protection, in addition to its economic significance.

International standing committee on water supplies in developing countries

Subject 1

International collaboration for improving water supply in developing countries

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1 Introduction

Global survey - UN development targets

The development of community water supplies in the industrially developed countries has been accelerated by their expanding economies. New knowledge of water-borne and filth-borne disease has hastened the process. Developing countries of the world, on the other hand, have in general been less fortunate. Their political emancipation carried several inherited burdens and their poor economies have inhibited progress. A runaway growth of population operates as a deadweight. It is therefore not surprising that present levels of health and socio-economic standards are alarmingly low.

Though improved health standards are generally equated to an improved water supply its impact on the betterment of socio-economic conditions is gaining universal recognition. Policy and decision makers are now more than ever aware of the role that water supply plays in productivity and economic growth and this explains their effort towards the development of the water supply sector.

The problems facing every developing country are very much the same. The theme does not change and is played the same way with few variations. Ignorance, poverty and disease continue the vicious circle that hampers development. More specifically, lack of skilled manpower, inadequate funds, poor technology, no community motivation and above all deficient policies and infrastructure are among the main factors affecting development.

A WHO global survey of urban water supply conditions in 75 selected countries mostly in the developing world was conducted in 1962. This survey was followed up again in 1970 and 1975. Results of the latest surveys may be summarised as follows:

—**Urban water supplies**

At the global levels, the percentage of urban population served by house connexions has increased from 50% to 57% in the period 1970–75. The percentage of people served by public standpipes has remained more or less the same at 18%. Thus the total urban population served either by house connexions or by public standpipes rose from 68% in 1970 to 75% in 1975.

Second UN Development Decade Target (1890): 60% of the population to be served by house connexions and the remaining 40% by public standpipes.

—**Rural water supplies**

New estimates of the 1970 population having reasonable access to "safe" water put the figure at 14%. This figure has increased to 20% in 1975.

Second UN Development Decade Target (1980): 25% of the population to have reasonable access to "safe" water.

Taking urban and rural populations as a whole, globally in 1970, 29% of the population of developing countries were served either by house connexions or public standpipes, or had reasonable access to "safe" water and this figure rose to 35% in 1975.

In terms of investment required it is estimated that approximately $14 500 million will be required to meet the global targets for the urban population and $6 500 million for the rural population. The total global investment in community water supply to meet the proposed new targets in the five years (1976–1980) would thus be in the region of $21 000 million which is approximately an annual investment per capita of $1,81, taking the estimated 1980 population of the developing countries as the base.

2 Roles assumed by different international agencies

Besides WHO, other UN Agencies involved in the water supply sector are UNICEF, UNDP, IBRD, UNEP, UN and WFP. Their degree and scope of involvement varies and may be summarised as follows:

2.1 UNICEF has generally been the funding agency for providing supplies and equipment for the implementation of community water supply projects in the rural areas. These projects in most cases have been implemented by WHO. UNICEF's outlay for community water supply has lately averaged $15 million a year and the population benefited in 1974 was over 8 million.

2.2 UNDP's assistance has been mainly oriented towards assisting preinvestment studies for water supply primarily for large urban areas. Invariably these studies have been entrusted to WHO, designated as Executing Agency. Preinvestment studies so far implemented with UNDP/WHO assistance in over two dozen developing countries have resulted in decisions to build new water supply and sewerage schemes worth about half a billion dollars.

2.3 IBRD and Regional Banks like IADB (Inter-American), ADB (Asian and African) and EDF (European) have so far been the principal International funding agencies for the construction of new water supply facilities and the improvement of existing ones. During the period from 1962–1975 both IBRD and IDA loans for Water Supply and Sewerage projects amounted to nearly $1170 million. IBRD works very closely with WHO under a Cooperative Programme for Water Supply and Sanitation and a somewhat similar Programme has also been established with ADB (African).

It would also be worth mentioning the significant contribution to the development of community water supplies made by a number of bilateral agencies like SIDA (Sweden), CIDA (Canada), FAC (France), KFW (Fed. Rep. of Germany), USAID (U.S.A.), DANIDA (Denmark) and OECF (Japan).

2.4 The role of both the UN and UNEP has been relatively limited, UN being mainly concerned with the development of water resources and UNEP with monitoring and surveillance of water quality. As for WFP it assists community water supply programmes in several ways including food aid as partial payment for wages as in Korea and Dominican Republic.

2.5 WHO's involvement in the field of water supply may be summarized as follows:

(i) *Country and Inter-Country projects*—there are well over 200 projects presently in operation. They include primarily strengthening of environmental health services in the country, development of community water supply programmes and training of nationals.

(ii) *Pre-Investment Planning*—so far nearly 60 pre-investment projects for water supply have either been completed or are in operation in around 50 countries. They include generally master plans and preliminary engineering and feasibility studies. These projects are in most cases financed by UNDP and as mentioned earlier they have generated investments worth nearly half a billion dollars so far.

(iii) *Sector studies*—these are studies carried out under a WHO/IBRD Cooperative Programme in which the existing situation of the water supply and sanitation sector is closely studied. So far sector studies have been carried out in over 28 countries and they have proved to be a valuable tool for national planning and sector improvement.

(iv) *Transfer of Information and Methodology*—this activity is carried out through training programmes, formalised courses either in schools of sanitation or universities, fellowships, in-service training, publication of documents, guidelines etc., WHO has also established a network of collaborating centres for community water supply where studies are carried out and information is collected and disseminated.

(v) *Surveillance of drinking water quality*—in collaboration with governments, attention is focused on institutional development for country-wide surveillance of water quality.

3 International collaboration

Five different situations

The scope and degree of involvement of International Agencies in the field of water supply in developing countries varies and depends on conditions and the situation in the country concerned. An important factor to be considered is the ability of the country to collaborate and to develop a programme. Five factors listed below have been identified for the purpose of assessing the country's status and preparedness to secure international collaboration and develop community water supply programmes. The factors shown vertically in a matrix represent country's inputs for programme planning and implementation. These inputs have been related to real life situations in five different hypothetical "countries" shown horizontally in the matrix.

Each situation represents a typical "case-study" and each case has been later discussed on its own merits. They each provide a different picture of how under different existing conditions a country's performance in the field of water supply is assessed and international collaboration accordingly provided. It is evident that the "Yes" and "No" are qualified statements and should be considered with sufficient margin of flexibility. The purpose of the following presentation is to stimulate discussion.

Country "A"

Existing conditions: Government has a policy for the development of water supplies; the country has an organisation responsible for water supplies; it also has manpower available; and a population motivated to develop water supply programmes. However, it lacks financial resources.

The above situation could well be described as existing in some Latin American countries like Columbia.

It is evident that the existing conditions are quite favourable to develop a programme and any International Agency should be willing to collaborate. The lack of funds, though a handicap, could be overcome by stimulating community approach and participation and by the use of simple and effective technology by the locally available manpower.

Foreign financial assistance either international or bilateral should be readily available to match existing resources.

Another aspect to be considered is that adequate operation and maintenance of the schemes is expected to be assured in the country and very likely the water system will be viable and self-supporting. The fact that the country has a government policy and an institutional infrastructure is a great step forward for the development and implementation of programmes.

Country "B"

Existing conditions: The country possesses a government policy for water supplies as well as skilled manpower and a local population well motivated to develop a programme. However, it lacks both an infrastructure and financial resources to undertake a programme.

The above situation, though perhaps not exactly similar, could be found in some countries in South-East

Inputs	CASE STUDIES				
	Country "A"	Country "B"	Country "C"	Country "D"	Country "E"
Government Policy	YES	YES	YES	NO	YES
Infrastructure (Institutional aspects)	YES	NO	NO	NO	YES
Financial Resources (external and internal)	NO	NO	NO	YES	YES
Manpower (skilled)	YES	YES	NO	NO	YES
Community Motivation	YES	YES	YES	NO	YES

Asia like India and Pakistan. If the country wishes to generate foreign funds, particularly from International lending agencies, a prerequisite would be the establishment of an effective organisation, management and financial structure for the water supply sector. Unfortunately very few developing countries have a strong central or regional authority to deal with the water supply sector. In most of the sector studies carried out by WHO/IBRD lack of an institutional structure has been found to be a major constraint for the development of water supply programmes.

Establishment of a sound and effective institutional structure is always one of the main objectives of every WHO assisted preinvestment project and this was the case in Ghana where as a result of the WHO/UNDP project for Accra and Tema the Ghana Water Supply and Sewerage Authority was established. In situations like that of Country "B" once an effective infrastructure has been established it is very likely that, with the manpower available and a motivated community, foreign funds would be forthcoming and help in programme development and implementation as happened in Ghana and in a few other countries.

Country "C"

Existing situation: Though the Government has a water supply policy and the community is keen for the development of their water supplies, the country has to struggle with no financial resources, no infrastructure and above all no skilled manpower.

The above situation is somewhat typical in many of the twenty five countries identified as "least developed" by UNDP.

In such situations training of local personnel and methodology and technology transfer should be the mainstay of any water supply development programme. Training should be geared to the local needs and conditions with objectives clearly defined.

Any international agency interested in collaborating in such countries would always ensure that once the collaboration ceases the nationals are in a position to take over and carry on the programmes on their own. This would include both planning and engineering designs of the systems as well as the study and implementation of organisation, management and financial aspects involved with a view to ensure system viability. In countries in this category like Nepal, Yemen Arab Republic and a few others in Central and West Africa, UNDP financed preinvestment projects for which WHO was the executing agency, contributed to the establishment of an infrastructure, to the training of nationals and subsequent international assistance for financing the investment programme.

In many countries facing the situation and conditions as in Country "C", it is often necessary for the government to subsidise the scheme. The country may also require operational assistance during the initial stages, which would include the study of an adequate tariff or rate structure compatible with the interests of both the consumer and the undertaking. House connexions are often a luxury that only a few can afford, and the bulk of the population would have to rely on standpipes. It is important in such cases that water rates be carefully studied so that even the poor consumer may have access to "safe" water. The health aspects and impacts of a "safe" water supply should in such cases over-ride financial considerations.

Country "D"

Existing conditions: All that the country possesses are financial resources. It lacks skilled manpower, a water supply policy, an infrastructure and a community that is motivated.

This case is typical of many "oil rich" countries.

Here again although initially the country will have to depend heavily on expatriate assistance it should nevertheless make every effort towards attaining self-sufficiency.

Foreign donors and agencies providing assistance either international, regional or bilateral on which the country has to depend have a great responsibility and an important role to play. They should ensure that the engineering designs and schemes that are implemented are suitable to the conditions in the country and tailored to the needs of the community served. As an example it would indeed be most unfortunate if sophisticated equipment and machinery which is expensive and difficult to operate and maintain is thrust on these countries, only because they can afford the price.

Crash courses and in-service training programmes for sub-professionals (plumbers, fitters, mechanics etc.), and for professionals (engineers, managers etc.), should be given every priority from the start of the programme. Training with a view to establishing a strong national expertise should be the main objective. It is through such a nucleus of trained national staff that an infrastructure could be built and decision and policy makers created within the country to establish sound policies and programmes.

Country "E"

Existing conditions: The country has all that is required to develop a sound water supply programme. Namely, a water policy, an infrastructure, funds, manpower and a community that is motivated.

Such countries are by and large very few in the developing world, and could be spotted in Latin America (Brazil).

It is a well known fact that through the years these countries have developed a sound infrastructure and a competent cadre of both professional and sub-professional staff. Initially also they received large financial assistance possibly from Regional or International Banks. However, all these inputs would have been to no avail without a strong government support and a sense of community involvement and participation which was an integral part of every programme and should be an object lesson to many other countries.

It may be mentioned in this connexion that when outside assistance ceased, countries in this category started to generate their own funds and in some cases establishing revolving funds and other ways of securing income necessary for the development of their programme.

4 Prerequisite for successful programme development

The philosophy and approach for programme development and implementation under different situations and conditions was discussed above. There are, however, a few factors that could contribute to the success of programme development some of which are outlined below.

4.1 Government support

A strong government commitment and support is a major prerequisite for the success of any development programme. Clear examples are found in countries like the Dominican Republic and Brazil as mentioned above where by establishing a national institution and policy and implementing a dynamic programme of education, consultation, stimulation and organisation of the communities benefiting, success was assured. Close technical supervision at all levels, fullest exploitation of economies of scale wherever possible and the development of financial and managerial procedures tailored to local needs are among the items deserving full Government support.

4.2 Institutional Building—Infrastructure

Experience from our early ventures in the development of water supply programmes has shown that the most common reason for failure in follow up on investment in developing countries was the absence of a sound and viable organisational structure.

The provision of water supplies requires substantial amounts of capital and the physical plans represent a large portion of the countries' infrastructure assets, both to protect the investment from deterioration and to provide for its effective operation and maintenance. Continuous and sound management is therefore essential.

It is important to recall that, in developing countries, the infrastructure for planning, implementation, management, administration and evaluation of national programmes falls within the purview of more than one ministry. Hence major coordination is required to maximise the efficiency of this important area or function.

4.3 Community participation

It is becoming increasingly evident that one of the main reasons for the slow development of water supply programmes is the lack of community participation and motivation. On the other hand it is also becoming abundantly clear that any visions of a global solution of the community water supply problem will remain a wishful dream without a strong involvement of the community to be served and a determined self-help approach.

The first step in any community water supply programme should be to determine the willingness and interest of the community in the undertaking, secondly the capability of contributing to the costs of construction in labour and cash and thirdly, the capability of the community of managing the system and collecting revenues for operation and maintenance.

Implicit in the above is the great need for community workers, health educators, sanitarians, etc., to motivate people and develop a demand for water supply service in the community. Last but not the least ways and means should also be found to motivate the policy and decision makers. This could be achieved in one way by producing well prepared project feasibility reports in which cost/benefit and financial/economic studies are well documented and easy to understand with a view to assisting the decision making process.

4.4 Resources

4.4.1 Financial resources

Lack of funds is generally a common constraint facing every developing country and the only alternative it has is to secure outside financing.

It often happens, however, that the country does not succeed in trying to obtain foreign funds. One of the reasons is poor project formulation and presentation. Countries should be made aware that a prerequisite for assistance to cover the foreign exchange component of a project is a comprehensive and well prepared preinvestment survey that would include both economic and engineering feasibility studies. Careful presentation and selection of alternatives and a thorough discussion of the economic and financial aspects involved is essential for lending agencies and decision makers. Projects so presented have found acceptance in national development plans and have attracted outside investment from national, international or bilateral lending agencies.

Cooperatives, housing banks, lotteries, revolving funds, to name a few means of support have all been used successfully in securing funds for water supply programmes. Where the economy level is low the government is obliged to subsidise. However, operation and maintenance must be accepted by the community as their responsibility.

4.4.2 Manpower resources

The importance of skilled manpower in the development of any water supply programme is most evident and has been dealt with earlier. Unfortunately the majority of countries in the developing world are faced with the problem of lack of competent personnel both at the professional and sub-professional level. Though the involvement of UN Agencies in improving the quality and numbers of manpower available for water supply work has been consistent yet it is far from meeting the needs of the countries.

Training of nationals both at professional and sub-professional level has always been one of the main objectives of practically every WHO assisted environmental health project. Training for sub-professionals is generally carried out at schools of sanitation. As for professionals, courses in sanitary engineering have been conducted at nearly 25 engineering centres which include the Universities of Nairobi, Lagos, Zaire, Kumassi, Ankara, Rangoon, Bandung, Bangkok, Chile, Bolivia, Peru, Lahore, Tehran, Rabat etc. Funds for training purposes are in many cases provided by UNDP and also by UNICEF for equipment and supplies.

Though methods of training vary in different countries, the type of personnel required is about the same and includes civil and sanitary engineers, health inspectors, water treatment plant operators, mechanics, plumbers, masons, carpenters, administrators, accountants and clerks. To train such a vast quantity of personnel also requires availability of professors, trainees, educational institutes, teaching materials and funds which are woefully inadequate in most developing countries. On the other hand, one has also to guard against merely classroom formal training with no "in-service" experience under supervision.

Considerable progress has been made, particularly in Latin America and in some countries in South-East Asia. However, concerted efforts by the respective governments in collaboration with International Agencies will be required for most countries for many years to come.

4.5 Technology

It can be safely assumed that in practically every situation related to planning and design of water supply projects the technical aspects and the technology involved are known and there is abundance of knowledge and experience in the field. All the same it often happens that systems are poorly planned and designed and the costs inflated to the ultimate detriment of the country concerned.

In many instances alternatives are badly selected with no economic and financial considerations not to speak of engineering aspects and the technology applied. This ultimately reflects in the equipment and materials used being far from conducive to conditions prevailing in the country. In this connexion the WHO International Reference Centre for Community Water Supply at The Hague through its thirsty collaborating institutions in as many countries ensures that the right knowhow and technology is made available to them on request.

The main criteria for the selection of systems should be the ability of the community to support the system both financially and otherwise. In other words, the systems should be planned and designed on the basis of what the population is capable of paying in terms of capital recovery and costs of operation and maintenance, and not left to the whims of the consulting firms and manufacturers of equipment whose interests may not be those of the community served.

5 Conclusion

This paper was intended to illustrate the existing global situation as regards community water supply and efforts made by International Agencies to collaborate with countries for the achievement of their targets for programme development and implementation.

Though great strides have been made in recent years conditions continue to be far from satisfactory and it is very unlikely that the global targets will be met. Some factors and constraints hampering sector development have been described above, and these are primarily: insufficient allocation of finance by governments, lack of trained personnel and inadequate external assistance.

Prospects for the immediate future are unfortunately not very bright. With the ever increasing escalation of prices, world wide inflation and political instability in many developing countries, with the consequent readjustment of priorities for assistance by the rich countries, a heavy onus has fallen on the countries themselves. Are they prepared to face the challenge? It is a question left open to be answered.

Ways, however, will have to be explored of making current investment go further and serve more people. This could perhaps be achieved by encouraging the use of simple technology, self-help projects, appropriate institutional structures, re-orientation of training and optimum utilisation of manpower.

It is also evident that International Agencies cannot isolate themselves from the world-wide financial trends. Rising costs have also affected International collaboration in programme development. However, one would hope this will only be a passing phase and it will not be long before the tempo is restored and stepped up to continue our long and fruitful collaboration with countries as their partners in development.

Subject 2A

Development of water supply in the metropolitan city of Lagos

By A. O. Olodude,

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Introduction

Water of satisfactory quality and in sufficient quantity is a prime requirement of society today. Impure or polluted water in these days of world-wide population explosion and unprecedented industrial growth is rapidly becoming the number one problem of advanced as well as emerging nations. The housewife is no longer satisfied with the quantity of water she can collect from the surface streams or the private shallow wells in her premises, and if she is, she definitely cannot rely on its quality. Not only does the remotest village in emerging countries have its own particular problems of supplying its inhabitants with adequate water, but so also do the great metropolitan cities of the world. The emphasis therefore in recent years has been to search for new raw water sources, treat the water thus obtained to acceptable standards and transport the treated water through pipelines to places where needed in sufficient quantity. In Lagos, the production and distribution of potable water became a public matter early in this century.

Social and economic position of Lagos

Metropolitan Lagos occupies a unique position in the Federal Republic of Nigeria. Part of Lagos recently delineated a special area still serves as the Federal Capital, while Ikeja, within sixteen kilometres of the city serves as the Lagos State Capital. Notwithstanding the fact that a new Federal Capital is now being planned, Lagos will still continue to serve as the commercial nerve centre of the country. Most of the sea-borne cargoes as well as the people coming in or going out of Nigeria pass through Lagos. The convergence of trade routes on Lagos led to its remarkable growth and has brought about a continuing migratory trend into it. In 1973, about 75% of the country's total imports and approximately 50% of the total exports (excluding crude oil) went through Lagos and Apapa ports.

Again, the importance of Lagos for the economy of Nigeria can best be described by the share of the city in relevant economic values and quantities of the country as a whole. Although there are no recent data available for Lagos Metropolitan Area, the figures for the Lagos State are reliable indicators because of the absolute dependence of the immediate hinterland on the performance of the city. The following table (Table 1) gives a comparison of some important economic indicators for Nigeria and Lagos State industries, wholesale and retail trades. All figures indicate a predominance of Lagos State in absolute quantities (numbers of establishments and employees, wages and salaries, sales) as well as the average per unit values (employees per establishment, wages and salaries per employee).

Because of the numerous roles which Lagos plays in the economic and social life of the Federation, it forms a target for job seekers with attendant problems of inadequate services notable among which is water supply.

TABLE 1

Industry and Trade in Nigeria and Lagos State, 1972

Economic Indicator	Nigeria	Lagos State	Lagos / Nigeria (in %)
(a) *Industries*			
No. of Establishments	861	227	26
No. of Employees	61 000	26 000	43
Employees per Establishment	71	115	—
Wages and Salaries (million ₦)	62,9	30,2	48
W & S per Employee (₦)	1 030	1 160	—
Sales (million ₦)	1 607	551	34
Value Added (million ₦)	386	135	35
VA per Employee (₦)	6 330	5 190	—
(b) *Trades*			
No. of Establishments	794	207	26
No. of Employees	53 600	22 200	41
Employees per Establishment	68	107	—
Wages and Salaries (million ₦)	58,4	27,3	47
W & S per Employee (₦)	1 090	1 230	—

Bibliography: Economic Indicators, Vol. 9, No. 8, August, 1973, Federal Office of Statistics, Lagos, Nigeria.

All figures for establishments with 10 Employees and more only.

Existing industrial estates are expected to be almost completely occupied during the next few years and additional areas planned both for future industrial and housing development between 1975 and the year 2000 may practically be equal to the land area now developing or being developed.

Climate and geology

Lagos enjoys a littoral type of climate. All the year round, the temperature hardly falls below 18°C and averages about 27°C with a seasonal variation of about 5°C. Relative humidity drops with the rise in temperature to about 70% in the afternoon during the dry season. The annual mean rainfall of about 1 800 mm occurs primarily in May–July and September–October periods.

The Metropolitan Lagos area consists of outcrops of two main geological formations:

(a) The coastal plain sands which form the low gently sloping dissected uplands reaching in places a height of about 61 metres and

(b) Recent coastal deposits which form the extensive and swampy alluvial plains of the major rivers and creeks along the coast overlying the coastal plain sands.

Historical background

The history of Lagos water supply dates back to October 1910, when construction of Iju Water Works commenced under the leadership of Mr. H. F. Peet, who later became the Director of the Public Works Department. On 1st July, 1915, the Waterworks was publicly opened by

Lord Frederick Luggard, the then Governor General of the Colony of Lagos and Protectorate of Nigeria, who turned on the knob of Idumota Public Fountain and collected water in a silver cup from which he drank. There were over 300 such fountains located all over the city of Lagos at that point in time.

In 1915, when the Waterworks was commissioned, the design capacity was 11 250 m^3 a day, the maximum daily consumption being 2 900 m^3 for a population of approximately 100 000 inhabitants. Water demand rose steadily until 1938 when it reached 13 500 m^3/d for a population of 156 000 people.

The headworks, located at Iju, 32 kilometres to the north of the City of Lagos, consisted of an intake structure on River Iju, a 900 mm cast iron raw water main, two steam-driven low lift pumps, four coal-fired Lancashire boilers, two horizontal flow sedimentation tanks and eight slow sand filter beds. The filtered water was stored in an 18 000 m^3 tank and pumped to a 54 000 m^3 distribution tank at Shaga about one kilometre from the works premises through a 700 mm diameter cast iron rising main by two steam-driven high lift pumps. From Shaga reservoir, a 700 mm diameter cast iron gravity main took off in a north-south direction and continued over a distance of 28 km. From this other feeder mains varying in size from 75 mm to 400 mm diameter branched off to form the distribution networks. Correction of pH is by the addition of hydrated lime, sterilization of filtered water is by the addition of chlorine.

The water supply authority, at its inception, was being managed as a division of the Public Works Department and the immediate control of the waterworks at Iju was under a Head Pumper, later passed on to a Chief Inspector and then to a Superintendent. The distribution centre was located in the city of Lagos. Supply was not metered except for industries and corporations.

With the expansion of the city to the mainland, the demand for water increased and to cope with this a 600 mm diameter cast iron gravity main was laid to Yaba Roundabout and commissioned in 1943.

The population of the city of Lagos and its environs experienced a phenomenal growth between 1945 and the early fifties and during this period districts on the islands were undergoing development while extensive development and resettlement schemes were also being experienced on the mainland districts. The original source of raw water—Iju stream—was found inadequate in meeting the demand for water and a new surface source—Ogun River—about 6 km from the treatment works was developed and commissioned in 1954, bringing the total production capacity to 50 000 m^3/d.

Following the attainment of Independence in October, 1960, urbanization and industrialization created the necessity for the expansion of water supply despite the development of 1954. Further developments were embarked upon between 1954 and 1964 bringing the production capacity to 108 000 m^3/d. The highlights of these developments were:

(a) Replacement of the old coal-fired boilers with two oil-fired horizontal boilers.

(b) New pumphouse at Iju Intake with four electric pumps.

(c) Additional intake and pumphouse on Ogun River.

(d) 900 mm diameter steel raw water main from Ogun to Iju.

(e) (i) sixteen vertical flow sedimentation tanks commissioned in 1960.
(ii) four vertical flow sedimentation tanks commissioned in 1964.

(f) (i) six rapid gravity filters, commissioned in 1960.
(ii) six rapid gravity filters, commissioned in 1964.

(g) Three electric high-lift pumps each with a capacity of 1 350 m^3 per hour.

(h) 1 000 mm diameter steel treated water rising main from Iju to Shaga reservoir.

(i) 1 000 mm diameter steel gravity main from Shaga reservoir to Lagos.

Master plan and feasibility studies—1964

In 1964 a firm of consultants was commissioned by the Federal Ministry of Works and Housing to carry out a feasibility study of the Lagos Water Supply Expansion.

In brief summary, the projected total cost over the ten year period of 1966 to 1975 amounted to ₦36,8 million including ₦2 million interest during construction.

Forecast on Demand

The demand forecast was as follows:

Year	Population	Estimated average demand 1 000 m^3	Maximum demand 1,5 × average demand 1 000 m^3
1964	916 000	167*	250
1970	1,5 m	270	405
1975	2,1 m	388	582
1985	3,5 m	630	945

* In March 1964, Lagos Water Supply supplied 80 550 m^3/d to an estimated population of 916 000. This was an average of 87,75 litres per capita per day as compared with the recommended design figure of 180 l/cap.d.

Recommended construction

The proposed construction to serve the design populations of 1970 and 1975 included:

(i) Impoundment on the Ofiki River, a tributary of River Ogun, to ensure a safe yield at Akute.

(ii) Construction of new raw water intake and an incombustible rubber dam at Akute.

(iii) Pumping and transmission mains.

(iv) Enlargement and modernization of the treatment plant at Iju.

(v) Construction of additional clear water pumping, transmission and storage facilities.

(vi) Provision of additional capacity in the gravity distribution system.

Additions and improvements to the distribution system

1. Increased storage capacity at Shaga from 27 000 m^3/d to 81 000 m^3/d, in the first five year period to 1970. A 56 000 m^3 tank was to be added by 1975, and a third extension of 112 000 m^3 by 1985.
2. Additional elevated storage of 30 000 m^3 was to be provided in six steel tanks on towers of 27 m to 33 m in height.
3. The principal main through a water district grid system was to be served by a main of minimum of 300 mm diameter.

4. Interconnection of the new feeder mains with existing mains at principal intersections was to be made in order to increase the efficiency of the whole distribution system. During the maximum demand conditions, residual head in the system was anticipated in the range 16 m to 18 m.
5. Metered connections for all consumers were considered the best and most equitable method of apportioning charges for water consumed and in reducing water losses.

Although it would appear that the Federal Ministry of Works and Housing had approached U.S.A.I.D. to finance and implement the first part of the Consultant's recommendations, which was the immediate improvement of the system to 225 000 m^3/d, no effective negotiations were concluded. The result was that the next few years that followed witnessed a period of "water crisis".

Crash programme expansion

With the creation of 12 states in Nigeria, the Lagos State Government took over the responsibility for the Lagos Water Supply from the federal government on 1st April, 1968, and quickly focused attention on alleviating the acute water shortage in Metropolitan Lagos. In May, 1969, the Federal Ministry of Works and Housing commissioned another firm of consulting engineers through the U.S.A.I.D. to submit proposals for the implementation of Phase I of the original report. The proposal submitted in November, 1969 provided for an increase of production to 240 000 m^3/d and envisaged construction works to commence in January, 1971. Because of the time factor involved in this proposal, the Lagos State Government opted for a crash programme which allowed a parallel operation of design and construction of the project, thus shortening the implementation period.

Thus in late 1969, proposals were invited for the provision of engineering services for the crash programme for the expansion of water production to 200 000 m^3/d. The scheme chosen, estimated at ₦4,80 million, comprised:

(a) New intake and pumphouse on Ogun River at Akute.
(b) Modernization of existing horizontal flow sedimentation tanks.
(c) Modernization of existing rapid gravity filters and provision of additional filters.
(d) Construction of a sludge thickener.
(e) Additional vertical flow tanks.
(f) Construction of power house and provision of stand-by generators.
(g) Clear water pumping station, pumps and piping.
(h) Electrical equipment.

A study of the losses in the distribution network was also included in the scope of work of the consultants. The contract for the construction of civil works commenced in 1970 and, simultaneously, tenders for the supply and erection of filter equipment, pumps, electrical equipment, generators, pipes and fittings, were invited and reviewed. However, as a result of long delivery periods quoted by the majority of the overseas tenderers, a high level delegation was sent to Europe and successfully negotiated the shortest possible delivery times and better financial terms with the suppliers.

Throughout the construction period, minimum disruption to existing services was ensured and by December, 1972, shortly before the commencement of the 2nd All African Games in Lagos, water production had reached the figure of 150 000 m^3/d. The programme was finally completed in May 1973. This was followed by a systematic tackling of the bottle-necks in the distribution network.

Lagos water supply expansion programme
Phases II and III

With the completion of the crash programme, it became obvious that a more permanent and comprehensive solution was necessary, and accordingly, in 1973, Phases II and III were designed.

Phase II envisages the construction of waterworks at Isasi, about 25 kilometres west of the city on the River Owo with a capacity of 160 000 m^3/d as well as improvements in the production capacity of Iju Waterworks. There will also be construction of additional overhead tanks in the distribution networks, and a low level weir at Akute on the River Ogun. Both improvements will provide 360 000 m^3/d in total by 1978. The above intermediate steps are designed to provide the stop-gaps required for the construction of the main scheme which represents Phase III.

Phase III envisages the construction of Adiyan Waterworks with a capacity of 950 000 m^3/d by 1994 to be achieved in three equal stages namely, 1980, 1985 and 1994. While the consultants were busy with the collecting of preliminary design data for Phase II, the Lagos State Government was commissioned by the Federal Government to provide a scheme capable of supplying 18 000 m^3/d for the Black Arts Festival Village and the Federal Housing scheme at Amuwo Odofin, adjacent to the village site. The project, which commenced in November, 1974, and was completed in December 1975, was constructed at Isasi on the River Owo and represents Phase II Stage I of the expansion programme.

Development concept report

Whilst the activities of Phase II Stage I were being vigorously pursued, the consultants' reports on water demand analysis, for Phases II and III, hydrological investigations (Phase II Expansion), and final development concept (Phases II and III), were submitted during the latter half of 1975. These were followed by the preliminary design for construction of Phase II Stage II.

Water resources

Among the resources of raw water investigated were:

(a) Groundwater
(b) Brackish water from the coastal lagoons, and
(c) Surface water from rivers.

In the selection of water sources for the development of Lagos Water Supply, the criteria used were:

(i) Early availability of water,
(ii) Relatively good knowledge of the technical parameters of the source,
(iii) Guaranteed security of the supply, and
(iv) Cost required for the development of the source.

Out of all the resources investigated, the source best fulfilling these conditions was surface water from rivers if reasonable storage could be made. River Ogun is the only river where adequate storage is feasible by the construction of a dam across its main tributary. Although in the case of the River Owo, geological conditions were found to be unfavourable for the construction of a dam, a weir against the intrusion of brackish water will make it possible to withdraw its total safe yield and it has been selected as a second source. A detailed study of underground resources was commissioned in 1975 to establish effective control of ground water abstraction in Metropolitan Lagos which is now in the hands of private commercial firms and institutions and also with a view to providing detailed and accurate data on the hydrogeology of the area for quick borehole construction take-off when an immediate water supply augumentation is needed.

Population projection and water demand analysis

The existing population projections prepared by different organizations and institutions vary considerably and reflect the poor statistical data available. For the future, three possible trajectories of Metropolitan growth were considered for Lagos as a whole. These present an extrapolation of current trends, and two different assumptions about the extent to which current rates may decline in time. Extrapolation of current trends implies that the population of Lagos Metropolitan Area will double every decade and reach a total of more than six million in 1985 and nineteen million by the end of this century.

This scale of growth may be regarded as very unlikely for two main reasons. Firstly, it would imply a constantly rising rate of immigration mainly from the neighbouring Oyo, Ondo, Ogun and Bendel States, which cannot be sustained over a period of time in view of the considerable difference that exists between the rate of growth of Lagos and the latter. It cannot increase for very much longer at the rate which would be required to sustain an overall immigration rate of 5,3 % per annum for Lagos. The scale of population loss is considerable in terms of the main origin of immigrants to Lagos and for Oyo, Ondo and Ogun States, it represents nearly 10 % of their estimated 1973 population.

Secondly, the evidence of growth of the cities in other parts of the Third World generally shows that the extremely high rates of growth which accompany the early phases of urbanization occur only for a short period of time. Consequently, it is necessary to consider possible levels of reduction in the rate of growth that has been experienced over the last twenty years. This largely involves a falling off in the rate of immigration, but it should be recognized that a reduction in immigration rates will also lead to a reduction of natural increase in the medium term as the proportion of young adults in the population is reduced. In the short term and even in the medium term, the effect of falling rates on the overall growth of population will be small relative to the scale of growth involved. For example, if the rate of immigration is effectively halved and the annual rate of population growth of Lagos during the 1973–1985 period is only 6 % as against the 8 % found in past trends, the present population of the State as a whole will still be doubled by 1985, to a total of about 5 million. In the absence of any other information pointing to a more drastic fall in rates, it is only realistic to plan for at least a doubling of the population over the 1973–1985 period and to reduce the growth rates only after 1985. From the evidence stated above, it seems logical to choose 5 million in 1985 and 9 million in 2000 as the basis for planning considerations (Table 2).

TABLE 2

Projected Population Growth for Lagos Metropolitan Area

	1973	1985	2000
	Thousands		
1973 Population extrapolated at 8% per annum to year 2000	2 400	6 000	19 000
1973 Population extrapolated at 6% per annum to 2000	2 400	4 800	11 600
1973 Population extrapolated at 6% per annum to 1985, and at 4% per annum between 1985 and 2000	2 400	4 800	8 800

Evaluation of future water demands

Assessment of the future water demand is based on the analysis of the past periods, evaluation of future population growth and assumed future average living standards of Lagos inhabitants with respect to both social and economic points of view. Future water demand is divided into three main categories depending on the type of the consumer, whether domestic, industrial or commercial. The figures indicating the water demand for a particular consumer category include all water losses occurring in the system after the main reservoir.

As a result of the rapid expansion of industries, the public water supply system was found inadequate and a considerable amount of water for industrial purposes is drawn from private boreholes. This situation cannot be accepted as normal, and it is assumed that industrial water demand will continue at an expected growth rate of 7 % per annum. As soon as the supply can cope with industrial requirements, the present high percentage of industrial borehole water supplies can be expected to be reduced; commercial and public water demand is estimated at 20 % of domestic demand. The development concept is based on the following water requirements contained in Table 3 below:

TABLE 3

Lagos Metropolitan Area Water Demand Projection

		1973	1985	2000
Projected Population persons		2 400 000	5 000 000	9 000 000
Domestic Water Demand	mgd	40,0	86,0	168,0
	1 000 m^3/d	182,0	390,0	763,0
Public and Commercial Water Demand	mgd	8,0	17,0	34,0
	1 000 m^3/d	36,0	77,0	155,0
Industrial Water Demand	mgd	4,0	15,0	40,0
	1 000 m^3/d	18,0	68,0	182,0
Average Daily Water Demand	mgd	52,0	118,0	242,0
	1 000 m^3/d	236,0	535,0	1 100,0

Water Charges can also be split into two components:

(i) Water supplied to the Lagos City Council Area is billed on a fixed charge of ₦80,00 per annum payable quarterly.

(ii) Metered usage at the rate of 55 Kobo per 1 000 m^3 for other users.

During 1974, the yearly average of water supplied to the City Council totalled 320,29 million cubic metres which would have resulted in a direct revenue of about ₦1,8 million at the rate of 55 Kobo per 1 000 m^3. This amounted to a subsidy of ₦1,00 million by the Ministry to the Lagos City Council. Water supplied to the L.C.C. was estimated at 65 % of the overall annual supply. The balance of the supply should yield a revenue of ₦970 000. However, actual revenue collected was only ₦383 000. From the foregoing, if the entire service area were charged on metered rates, the shortfall in revenue in 1974 was at least ₦1,50 million.

Water sales and profitability

The Water Division operates at a very substantial loss which is increasing each year. This can be clearly seen in the following table (Table 4) which shows the estimated trading positions for 1974/75 and 1975/76 financial years.

TABLE 4

	1974/75 Approved Estimate ₦1 000	1975/76 Approved Estimate ₦1 000
REVENUE	720	920
EXPENDITURE		
Salaries and Wages	593	970
Maintenance and Running Costs	2 000	2 000
Water Connection Services	90	90
	2 683	3 060
DEFICIT	1 963	2 140

The deficit for 1974/75 would be greater than the figure shown for the following reasons:

(a) Actual revenue collected totalled ₦597 000.

(b) Actual cost of payrolls amounted to ₦864 000.

(c) Expenditure did not include for

(i) Gross charges from other divisions of the Ministry or other government agencies for services provided.

(ii) No account was taken of depreciation on fixed assets or interests on loans.

Financial resources

Seventy-five per cent federal contribution to water supply includes the following capital projects:

(a) Water supply expansion Phases II and III,

(b) Mains extension and restructuring,

(c) Water supply metering,

(d) Consultancy fees.

Again, the analysis of the Five Year Development Plan (1970–1975) also reveals that availability of funds has been no constraint on the performance of the Water Division as contained in Table 5 below.

TABLE 5

National development programme 1970–75

	Authorized (₦1 000)	Spent (₦1 000)	% Spent
Nigerian Overall	1 263 532	593 980	47%
Lagos State Ministry of Works and Planning	184 955	49 884	27%
Water and Sewage	31 376	5 675	17%

General problems

As discussed earlier in this text, with the federal government participation in the Lagos Water Supply projects, finance is not a constraint. A major constraint is lack of sufficient high level manpower and the low calibre of the low-level and intermediate operatives with the result that all available engineers are overloaded with responsibilities. However, with the new concept of management, Management by Objectives (M.B.O.), introduced in the country in 1975, increased efficiency in the organization is being realized.

Illegal sales of water, falsified meter readings, illegal tappings and unauthorized diversion of mains supply are rampant although with the recent creation of "complaint centres" within the distribution area these shortcomings are expected to be substantially reduced. Such complaint centres also serve as feedback for information on pipe bursts, malpractices by water supply employees and private plumbers. The feedback is instantly monitored to the operational centres for necessary action. Another major set-back in development, operation and maintenance is inadequacy of materials. For instance, there are only two firms in the country which produce asbestos cement pressure pipes. There are also few foundries. In this respect even when pipes are available, metallic joints and specials may not be available in sufficient numbers. This situation could have been tolerable under normal population growth but not with the present dimension of population explosion.

Existing primary networks are tailored along a north–south axis without booster stations in the system. Until very recently, Metropolitan Lagos had expanded only from the south towards the north hence the later settlements are constantly under low-pressure. The present development concept of water supply expansion has, however, taken care of such problems.

Conclusion

Since the execution of the crash programme in 1973, water supply in Metropolitan Lagos has been given greater priority than in the past. A global and more realistic approach through the long term concept report already accepted by Government has been staged for execution. At 1975 price levels, the sum total of the programme was estimated at ₦380 million exclusive of tertiary distribution networks.

It is obvious that a review of the present water charges is desirable at least to make the organization viable in respect of operational costs. Whether the organization remains an arm of the Ministry or operates as a separate entity in future, it is paramount that it should continue to render effective services to the consumers.

Water supply problems in Jakarta

by Irwin Nazir

Indonesian Water Supply Association

1 Introduction

The state of development of water supply facilities in most cities in Indonesia has been viewed as far from adequate to serve the rapidly growing population and its expanding activities.

Practically in almost all cities in Indonesia, water supply conditions had received little attention and facilities were not properly maintained during a number of years following the outbreak of World War II. This situation had led to deterioration of the then existing facilities, primarily pipelines and accessories of the distribution network, shortening their useful life expectancies and, in many cases, causing substantial decreases in capacity. Illegal service connections, beyond control of the local water authorities caused the conditions to become worse.

More serious problems regarding water supply have been encountered in the large cities where urbanisation has resulted in a high rate of population increase, with which are associated the development of concentrated areas of exceptionally high population densities, and of populated centres in the outskirts, exerting demands for water beyond the design capacity and the reach of the distribution system. People living in such areas must rely upon private water supply systems, or initiate self-help water supply projects, to meet their demand for water or make-up water, where the supply from the public water supply system is inadequate. Many of these private water supply systems are drawn from sources of questionable conditions.

Considerable efforts have been made to attract greater concern of the people regarding their participation in safeguarding potential water resources, considering that private water supply is still indispensable until every household can be served by the municipal water supply system, or before they can be served with adequate quantities.

Although the urgent need for potable water has been recognized, and the Government of Indonesia has embarked upon investment programmes to improve the situation, the limited funds in the budgetary allocation provided for the development of this sector of basic services and the very great demand for water supply improvements in the country have imposed difficulties regarding the selection of priorities and in establishing the extent of the development in each city that has been considered. Aside from the requirements of the very large number of cities, also to be considered in the development programme are the requirements of about 81% of the total population of more than 126 000 000, living in the rural areas. Urban water supply development has reached a total production of only 17 000 l/s, while it has been estimated that only less than 5 000 000 of the rural population will have access to potable water supply by 1976. Financial assistance, in most cases in the form of loans, has been provided by foreign governments and international lending agencies to help speed up the development of water supply in the country. However, considerable funds will still be required to finance further developments in the future.

In the administrative structure of the Government of Indonesia, four Ministries are concerned with the development of water supply. These are: the Ministry o Finance, Ministry of Health, the Ministry of Public Works and Power, and the Ministry of Interior. Water supply development in large and medium size cities is administered by the Ministry of Public Works and Power, while the Ministry of Health is more concerned with developments in rural areas and smaller towns, where initiation of water supply projects is required for the improvement of health conditions, and with quality control. Operation and management of water supply systems is left to the Ministry of Interior, through the local authorities. Although there is no delineation regarding the above mentioned responsibilities, it seems that this concept is being applied in the country. Other Ministries may also initiate their own water supply projects. However, such projects are normally attached to main projects developed by those Ministries.

The Indonesian Water Supply Association (*PER*satuan *P*erusahaan *A*ir *M*inum *S*eluruh *I*ndonesia—*PERPAMSI*)

The Indonesian Water Supply Association was established in April 1972, by the decision of the First Conference of Municipal Water Supply Enterprises, attended by representatives from 54 cities of Indonesia. This association was created in view of the similarity of problems faced by municipal water supply enterprises, and of the need for collaboration to develop autonomous business organizations based on sound principles of economics.

By the decree of the Minister of Interior of June 23, 1975, this association has been given recognition as a "semi-official professional organization", and placed under a board, chaired by the Minister of Interior, which will provide the association with general and technical directives. Other members of this board are: the Minister of Public Works and Power, the Minister of Health, and the Minister of Finance. By that decree, the Indonesian Water Supply Association becomes coordinatively affiliated with the "Inter Indonesian Municipality Organization" (*B*adan *K*erja *S*ama *A*ntar *K*ota *S*eluruh *I*ndonesia—*BKS-AKSI*).

2 The development of water supply for Jakarta

Jakarta City

Jakarta, which was founded in the 16th century and became later the capital city of Indonesia, occupies a land area of 657 km² on the northern coastal region of the Island of Java. This relatively flat coastal plain is dissected by rivers, originating at the mountainous area south of the territory, of which the Ciliwung River is the largest.

Average yearly rainfall varies from 2000 mm near the coast to 4000 mm in the mountainous region, concentrated during the wet season, which lasts generally from November till May. The wettest month is generally January. The dryest month is August, with an average minimum rainfall of ±3,5% of the yearly total. The number of inhabitants of Jakarta, which was 300 000 in

1920, and 800 000 just before World War II, has grown explosively to 4 576 009 in 1971 (as recorded at the time of the last census). The estimated population of 1973 is 4 915 265.

The Jakarta Water Supply Company (Perusahaan Air Minum Jaya)

The Jakarta Water Supply Company was established by the Decree of the Governor of the Special Territory of the Capital Jakarta in 1968, and has since then taken over the management of the water supply system from the Municipal Government.

Water Supply for Jakarta

The Jakarta main water supply dates from 1908, when water was drawn from 15 deep wells located in the southern part of the present city. The total production of these wells was 200 l/s (17 280 m^3/d).

In 1922, an additional supply of 300 l/s (25 920 m^3/d) was provided to the city, which had, at that time, an estimated population of 300 000. The source of this new supply is a spring at the foot of Mount Salak, about 65 km south of Jakarta. No treatment, except for disinfection, is applied to the water, considering its superb quality for human consumption. Unfortunately, the limited rate of supply from this spring necessitated recourse to other sources to meet the requirement of the city.

Further development of the main water supply system took place in 1953 with the initiation of construction of a water purification plant within the city. This plant, known as the "Pejompongan Plant" started its production in 1957 with a capacity of 2000 l/s (172 800 m^3d). Raw water is drawn from a canal (i.e. the "Banjir Kanal") which was originally constructed to divert flood flows from the Ciliwung River. With this plant in operation, the total rate of delivery of the then existing system was 2500 l/s (216 000 m^3/d), supplying 1 950 000 inhabitants of the city. The Pejompongan Plant was expanded with an additional capacity of 3000 l/s (259 200 m^3/d). Construction of the expansion facilities commenced in 1963. In 1970, the plant allowed an additional rate of delivery of 2 000 l/s to the city, while the 3000 l/s design capacity was fully utilized in 1975.

With a total capacity of 5 500 l/s (475 200 m^3/d), on an "average per capita supply" base, the quantity of potable water that could be provided seemed just adequate to meet the average minimum demand for domestic usage. This would have been the situation under satisfactory conditions of the distribution system. However, the present conditions of the system do not allow even distribution of the available water to all parts of the city. Neither does the distribution network cover the whole city, as is indicated in the attached figure. Based on a study carried out in 1972, the estimated population served by the system at the time of study was 48%, of which 55% (or 26% of the total population) was served by direct connection, and the remainder (22% of the total population) through water vendors. Of the area served, there are still parts that are receiving intermittent supply or poor supply.

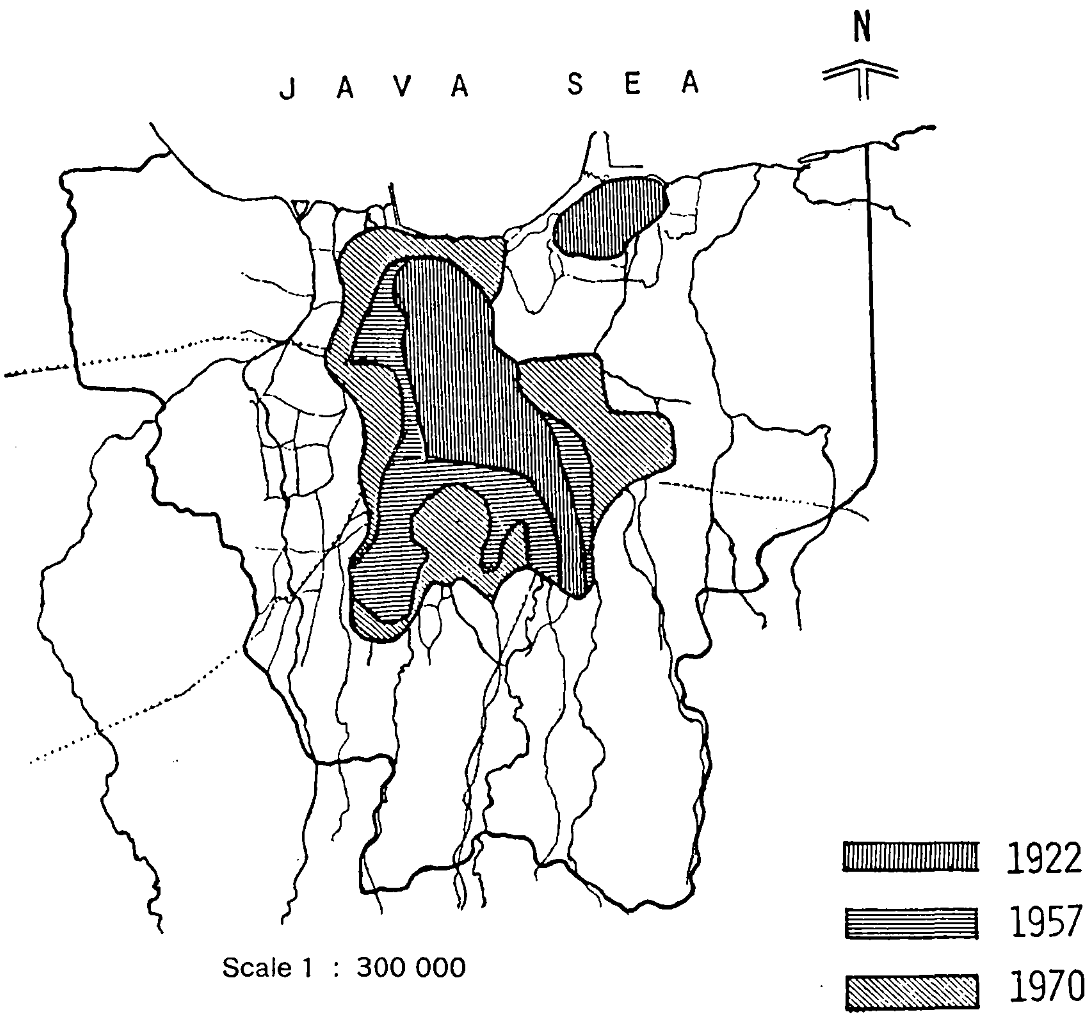

Figure 1 — Jakarta water supply service area

Supplementary water supply facilities have been constructed by the Water Company in parts of the city, outside the area served. During the period from 1968 through 1970, 35 deepwells were constructed. These wells are producing a total rate of flow of 70 l/s (6048 m^3/d). People who do not have access to water provided by the company have to maintain and rely upon private water supply systems. Some factories and housing estates use deep wells, while individual households have their supply from shallow wells. Privately owned deep wells have later been put under the control of the Water Company.

In the Master Plan for the Jakarta Water Supply System, several project stages have been proposed, which include effective utilization of the existing production capacity and the construction of new treatment plants and pipelines to meet the requirements of the city up to the year 2000. It has been estimated that to serve approximately 90% of the projected 8 300 000 people in the year 2000 (directly and indirectly), a production of 17,7 m^3/s (1,53 million m^3/d) will be required. This Master Plan is being used by the Water Company as a guidance in developing the water supply system for Jakarta.

3 Problems associated with the development of water supply for Jakarta

Under the conditions that exist at present, the Jakarta Water Supply Company has to make every possible effort to gradually improve the situation. Because of the interrelationship of the various problems that have to be solved, and realizing that any action taken to solve a certain problem may have a bearing outward the jurisdiction of the company, a careful study has to be conducted before any attempt is made to bring a solution to the problem.

Of the many problems that have been encountered, the most pertinent, and those which are considered as being of international interest, are described in this chapter.

Financial Problems

Major expansions of the water supply system of Jakarta have been financed from loans provided by governments of foreign countries and international lending agencies. Loan agreements are arranged by the central government, through the Ministry of Finance. Under this arrangement, interest and loan principal are paid back by the central government from general revenues.

Even for a large city, such as Jakarta, it is still difficult at present to generate significant revenues from water users which not only cover operation and maintenance costs, but also leave sufficient amounts for system expansions and to pay back loan principal and interest.

The condition of the existing system, primarily the distribution facilities, requires a major portion of the revenues to finance rehabilitation and replacements, and to cover maintenance costs. Considerable amounts have been spent on replacements to minimize water losses and hence increase the quantity of water that could be distributed to the consumers.

Other problems faced by the water company are those associated with the establishing of the pricing scheme. First, the difficulties in ascertaining the budget required to improve the condition of the system and to what extent this budget will have a bearing on the rate of water-use, and how this bearing should be distributed through time. Would it be considered fair to place this budget on the account of the present consumers? To solve these problems, a very elaborate inventory and accounting programme has to be conducted, which will certainly bring forward a variety of new problems. The second problem relating to the pricing of water is more of a socio-economic nature. The significant variation among water users in their income and ability to pay, the kind of water usage, and the benefits received by those supplied with water, are among the factors that have to be considered. To conduct an income survey would hardly be possible, while complete data may not be available at the revenue office. This situation necessitates the setting of an equal unit price of water for household usage, which may impose a heavy burden on the low income families, although these may be more fortunate than families who have to buy their water from vendors at higher prices. Quite a number of consumers of this income category are behind with their payments, facing the risk of being cut off from the supply. On the other hand, to the very rich consumers, who may not even care on whatever level the price of water is set, the established household rate may induce wastage of water, although a two-fold rate has been set for excessive quantities.

All the factors of consideration that have been mentioned are also regarded in establishing other charges that are associated with water usage. Among these charges, the different rates of initial connection fee may be cited as an example. The philosophy behind the charging of initial connection fees stems from the fact that the selling price or the value of a premise would increase remarkably, to the benefit of the owner, when this premise is connected to the city distribution system, and that benefit could best be contributed to the funds required for financing the overall development of the water supply system. This concept of charging initial connection fees has lately become a subject of discussion regarding its justification.

Another source of income of the Water Supply Company is the sale of water from privately owned artesian wells. If the discrimination of charges for usage of water supplied by the city water supply system is based on the kind of usage, the basis of charging well owners for the water drawn from their wells is the location of the well with respect to the service area of the city distribution system. In the present pricing scheme, water from wells located beyond the reach of the city distribution system is charged 25% of the rate of water supplied by the water company, while the rate within the service area of the city distribution system has been set at 50% of the rate of city water. Charging for water from privately owned wells has been in effect only recently, primarily for the purpose of controlling and preventing excessive withdrawal of groundwater, which is of course justifiable. The discrimination of water rates, however, should be viewed from other points.

Managerial and Social Problems

The change of status of the water supply authority in Jakarta has given more autonomy in managing the water supply system. Without disregarding the considerations that relate water supply to social needs of the community, management of water supply has since then been based on the concepts of business organization. Although this connotes something materialistic, it has been considered timely that a water supply undertaking be self sustaining.

The main problem faced by the managing staff of the Jakarta Water Supply Company under its present status as a (local-) government enterprise is to find the most appropriate system of management, under the existing conditions. It seems that the company has inherited the problems that have been faced when the water supply system was administered by the Municipal Water Supply Department, and that the phrase (if there is such): "the property of the government is also the property of the people" seems still to be the principle of a number of consumers, in the sense that one has not to pay for what one owns.

Naturally, the company realizes that part of the problem of management is being caused by deficiencies in the services extended to the consumers, primarily the quality of service, such as: the limited, or intermittent, supply due to the shortage of production, or because of excessive usage of water in areas where the supply conditions are favourable; inadequate pressure in the supply lines because of leakages and illegal service connections, which further creates tendencies for the installation of in-line pumping facilities by consumers and thereby affects the supply to others; and still many other shortcomings.

The deficiencies created by the shortage of production and the improper conditions of the system, the attitude of a number of consumers, the insufficient amount of revenue that could be generated to rehabilitate the system, and the inadequate number of personnel of the company that are required to control the situation, could be considered as elements of what we may call the vicious circle of problems which has rendered the efforts of the company to increase revenue from water-use less effective. Although releases have been issued, through which cooperation of the consumers has been requested to optimize utilization of the existing facilities it seems that other measures need to be developed to remedy the situation.

As the raising of revenue is considered as the most important thing to improve the service to the consumers, the company has given serious thought to problems related to the big amounts of unpaid bills and the collection of bills. Regarding the latter, the Jakarta Water Supply Company has involved private organizations in the collection of bills, on a contractual basis. Certain parts of the service area are allocated to these organizations, and incentives are paid at rates that depend on the amounts of revenue collected by the organizations. At present, this way of collecting bills is still considered as a trial method, and only a part of the area has been included. This method, naturally, reduces the number of personnel under direct employment of the company, which to some extent minimizes the problems associated with management of personnel. This method has so far been considered as the only solution because it is still uncommon for the majority of the people to have bank accounts. With less employees involved in the financial division, more room could be made available to recruit employees of the technical division, who are required for rehabilitation and maintenance to reduce the amount of unaccounted-for water, hence increasing the quantity of water that could be sold.

Another problem which could also be regarded as a social problem and to which a solution has still to be sought, is that of maintaining the quality of raw water in the river. Suspended matter and floating debris in the Banjir Canal water, which were not removed after screening, have frequently been the cause of failure of the desludging equipment of the Pejompongan plant. It seems very difficult to appeal for people's conscientiousness and their participation in not dumping any refuse into streams, although this is partly due to the inadequacy of facilities that could be provided by the municipality.

Considering the distance of flow of the raw water source through urbanized areas, considerable high loads of wastes must have been taken up. At present, a substantial reduction in strength of the pollution load could be attained by dilution. However, this situation should be watched carefully because the problem may become serious in the future if the existing regulations for the disposal of waste matter into public waters could not be enforced effectively, and considerable funds will then be required to restore the conditions of this potential source of raw water supply.

Staffing of the Company and Training of Personnel

Continuous assistance from foreign countries, primarily in technical fields, has been received since the development of the first Pejompongan purification plant in 1953, and it may take a few years before all activities associated with the water supply for Jakarta could be placed entirely in the hands of local staff. Of course, some of the expertise is assigned in accordance with the requirements set by the lending countries and foreign agencies, when providing their loans. It is, however, a fact that the current staff of the Jakarta Water Supply Company is inadequate to handle all the problems which emerge from the conditions of the system, and problems that have come into existence when new technology has been brought into the country. In this respect, the concept of a turn-key project, such as had been the case with the first Pejompongan purification plant, could be critized in the sense that the level of technology that is attached might not be appropriate to the available local expertise at that time. Only through intensive in-service training, have local engineers been able to accomplish proper operation of the plant equipment.

With regard to technical personnel to man the existing plants, the experience from the past has eliminated the difficulties of training of new personnel. The problem is whether the company is able to recruit engineers, considering the limited number of graduates from local technical universities. Difficulties have been encountered in finding a place where staff members of the company could be trained in other fields to enable them to solve other aspects of the problems. A most appropriate place would be a city where the conditions are similar to that of Jakarta.

The Problem of Technology Level

The development of water treatment technology has reached such a state that almost no difficulties will be encountered in treating raw water, of reasonable quality, to make it acceptable for drinking water. This technology has been transferred from the countries where it has been developed, to almost any developing country, Indonesia being one of them, through training programmes, assignment of foreign expertise to developing countries, scientific publications, foreign contractors, etc.

The Experience of Jakarta, and probably also in other developing countries, revealed that the problems regarding the application of advanced technology are mainly associated with (1) the availability of skilled personnel to operate the plant and to maintain proper conditions of the facilities, (2) the availability of equipment, or parts for replacements, which can be purchased easily, and (3) the initial cost of the plant.

The problem of skilled personnel has been pointed out in the preceeding paragraph, and no further discussion on this problem will be required here. With regard to the availability of replacement parts, careful consideration should be given to the selection of type of equipment or instruments, because most of the items are not fabricated in the country and have to be purchased elsewhere. Repair of malfunctioning instruments may not be possible locally and those instruments may have to be sent to other countries for repair, or factory technicians may have to do the repair work in situ. This of course necessitates the allocation of additional funds to cover transportation or travel expenses. Another matter to be considered is that dependability on foreign countries may necessitate the provision of additional investment on spare equipment to avoid loss of continuity of operation of the plant, which again increases the initial cost.

Initial cost is indeed the most important factor to be considered during the planning phase of a water supply project. Based on rough estimates, a so-called modern purification plant is more costly than the conventional-type plant (note that advanced technology is generally related to modern plants), partly because of the imported parts or equipment, and partly other expenses that are generally attached to such projects, such as travel expenses and fees for foreign construction supervisors. Considering that the higher initial cost, on a unit-of-water basis, the higher will be the user-charge, it should be considered necessary to keep the initial cost of a plant as low as possible. Modern plants have been claimed to be of greater efficiency than the conventional-type plants. This is true from the operation and process points of view, land area requirements, and the number of personnel that is needed to operate the plant. The question is whether it would be justified to attain the additional efficiency (relative to that of the less costly plants) at the additional cost. The price of land in some places may be high, but labour in most developing countries is less expensive than in the industrialized countries. Also, less qualified personnel are needed in the case of the conventional-type plant.

Emergency and Complementary Projects

It has been indicated in the preceeding chapter that a Master Plan for the Jakarta Water Supply System has been prepared. If all the projects that have been proposed in the Master Plan could be implemented, Jakarta could then be considered as a city with an adequate supply of water. According to this Master Plan, new water purification plants will be constructed: one plant of 4000 l/s capacity which, in addition to the currently available capacity, will satisfy the demand up to 1980, and one plant of 7400 l/s capacity to satisfy the requirements up to the year 2000. Both plants will be constructed in phases, as will be the construction of the distribution facilities.

During the last two years, certain parts of the city have reached a state of development which requires immediate supply of water. Some of these parts, although included in the projected service area of the proposed purification plants, will not be served earlier than 1990. Considering such a situation, the Water Supply Company has studied the possibilities of initiating emergency projects to supply the needs of the people in those areas, and other areas which may suffer from shortage of water during the years before the proposed plants could be put into operation. This idea of initiation of emergency projects was strengthened by the delay in the implementation of the projects proposed by the Master Plan.

The basic concept of the emergency projects is the development of "independent" water supply systems, consisting of a "moveable" water purification plant and a distribution system, in areas which will be served by the main water supply system of the city in the future. When the areas served by such a plant have received the required supply from the main system, the purification plant will be dismantled and the moveable parts, or units, of the plant will then be moved to other locations. The nature of the projects require careful considerations regarding the selection of areas and the duration of the project to render it economically feasible.

Design plans have been prepared for the installation of a purification plant with a capacity of 400 m^3/h. Additional units will be installed when the experiments with the first 400 m^3/h plant have shown satisfactory performance, and the method which has been applied to solve the temporary problem of water supply has been proven successful.

This type of project could of course be applied as a complementary supply facility to serve remote areas outside the planned service area of the main system. In this case also it is required to study whether this independent system is economically feasible, or whether it will require subsidy from the main system. Considering the existence of a discriminating pricing scheme, the amount of revenue generated from such a project depends, among other things, on the type of occupancy of the proposed areas.

Unfortunately, this concept may not be applicable to certain parts of the city due to unavailability of sources of raw water within economic distance. For such areas, a more feasible solution has to be sought.

4 Conclusions

The explosive growth of the population of Jakarta and the present condition of the distribution system are two major causes of the problems of water supply. New water purification plants have been constructed to increase the rate of production of potable water. However, the poor condition of the distribution system does not allow effective utilization and distribution of the added quantities.

Insufficient amounts of revenue that could be generated, partly due to the high percentage of losses and unaccounted-for water, have produced financial problems, constraining the efforts to improve the conditions of the system, while the limited funds allocated by the central government to the water supply sector and the enormous demand for water supply improvement throughout the country may necessitate giving a lower priority to Jakarta. Therefore, other sources of funds need to be sought to finance the implementation of development plans. Such an unfavourable financial situation makes it difficult for the Jakarta Water Supply Company to manage the water supply system based on economic principles. Although it has been given the authority to do so, there are still many factors that have to be considered, amongst which are the economic conditions of the majority of the people, and the social nature of water supply service.

No serious problems associated with the application of modern technology have been encountered, as far as the requirements for skilled personnel to operate plant facilities are concerned. However, justification from the financial point of view will still be needed, prior to the application of such technology, taking into consideration that, in this aspect and until the near future, Indonesia is not a manufacturing country.

Résumé

La *détérioration* de l'état des ouvrages de distribution d'eau due au manque d'entretien pendant un certain nombre d'années après le début de la Deuxième Guerre mondiale et la croissance explosive de la population de Djakarta peuvent probablement être considérées comme les causes majeures des problèmes que doit surmonter le service des eaux de Djakarta pour assurer à la ville une alimentation en eau adéquate. Bien que la production d'eau potable ait été accrue par la construction de stations de traitement, toute l'eau produite ne peut pas être effectivement utilisée étant donné l'état actuel du réseau de distribution.

Des nombreux problèmes liés au développement de l'alimentation en eau de Djakarta, les problèmes financiers ont donné le plus de souci étant donné l'importance des investissements exigés pour améliorer la situation et pour exploiter convenablement le réseau. Le présent rapport décrit les diverses sources de revenus, les problèmes qui se sont présentés et les facteurs pris en compte en établissant les prix de vente et autres charges liées à l'utilisation de l'eau.

Les deux problèmes principaux relatifs à la détermination des prix de vente de l'eau sont: les bases d'établissement du plan de tarification, et les conditions socio-économiques. Ces dernières, et d'autres déficiences dans le service et le réseau, sont considérées comme la cause des problèmes de gestion du réseau de distribution d'eau; elles affectent en outre le volume des recettes qui pourraient venir de la vente de l'eau. Sont inclus également dans la discussion les problèmes relatifs à la qualité de l'eau brute, qui peuvent devenir plus graves dans l'avenir si l'on ne prend pas de mesures positives.

En ce qui concerne le personnel du service des eaux de Djakarta, on s'est attaché à intéresser des experts étrangers au développement du service des eaux de Djakarta, en liaison notamment avec l'assistance financière apportée par les gouvernements étrangers et les banques de prêts internationales, et avec le niveau de technologie appliqué.

D'autres discussions ont été incluses sur les divers problèmes associés à l'emploi de ce que l'on appelle la technologie moderne de purification de l'eau. Des considérations spéciales ont été faites sur les aspects financiers de cette technologie.

Les importants investissements nécessaires pour développer le réseau de distribution d'eau, le problème d'accroître les revenus provenant de la vente de l'eau et des charges liées à l'utilisation de l'eau, et l'aide limitée qui pouvait être accordée par le gouvernement central ont retardé la réalisation des programmes de développement. La situation a amené à penser qu'il faut modifier les étapes du programme prévu et mettre en route des projets d'urgence qui demandent de plus faibles investissements.

Le rapport conclut que les problèmes auxquels est confronté le service des eaux de Djakarta sont surtout des problèmes financiers, les autres problèmes résultant de cette situation défavorable.

International collaboration for improving water supply in developing countries

by Dr. B. H. Dieterich

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1 Introduction

Global survey - UN development targets

The development of community water supplies in the industrially developed countries has been accelerated by their expanding economies. New knowledge of water-borne and filth-borne disease has hastened the process. Developing countries of the world, on the other hand, have in general been less fortunate. Their political emancipation carried several inherited burdens and their poor economies have inhibited progress. A runaway growth of population operates as a deadweight. It is therefore not surprising that present levels of health and socio-economic standards are alarmingly low.

Though improved health standards are generally equated to an improved water supply its impact on the betterment of socio-economic conditions is gaining universal recognition. Policy and decision makers are now more than ever aware of the role that water supply plays in productivity and economic growth and this explains their effort towards the development of the water supply sector.

The problems facing every developing country are very much the same. The theme does not change and is played the same way with few variations. Ignorance, poverty and disease continue the vicious circle that hampers development. More specifically, lack of skilled manpower, inadequate funds, poor technology, no community motivation and above all deficient policies and infrastructure are among the main factors affecting development.

A WHO global survey of urban water supply conditions in 75 selected countries mostly in the developing world was conducted in 1962. This survey was followed up again in 1970 and 1975. Results of the latest surveys may be summarised as follows:

—**Urban water supplies**

At the global levels, the percentage of urban population served by house connexions has increased from 50% to 57% in the period 1970–75. The percentage of people served by public standpipes has remained more or less the same at 18%. Thus the total urban population served either by house connexions or by public standpipes rose from 68% in 1970 to 75% in 1975.

Second UN Development Decade Target (1890): 60% of the population to be served by house connexions and the remaining 40% by public standpipes.

—**Rural water supplies**

New estimates of the 1970 population having reasonable access to "safe" water put the figure at 14%. This figure has increased to 20% in 1975.

Second UN Development Decade Target (1980): 25% of the population to have reasonable access to "safe" water.

Taking urban and rural populations as a whole, globally in 1970, 29% of the population of developing countries were served either by house connexions or public standpipes, or had reasonable access to "safe" water and this figure rose to 35% in 1975.

In terms of investment required it is estimated that approximately $14 500 million will be required to meet the global targets for the urban population and $6 500 million for the rural population. The total global investment in community water supply to meet the proposed new targets in the five years (1976–1980) would thus be in the region of $21 000 million which is approximately an annual investment per capita of $1,81, taking the estimated 1980 population of the developing countries as the base.

2 Roles assumed by different international agencies

Besides WHO, other UN Agencies involved in the water supply sector are UNICEF, UNDP, IBRD, UNEP, UN and WFP. Their degree and scope of involvement varies and may be summarised as follows:

2.1 UNICEF has generally been the funding agency for providing supplies and equipment for the implementation of community water supply projects in the rural areas. These projects in most cases have been implemented by WHO. UNICEF's outlay for community water supply has lately averaged $15 million a year and the population benefited in 1974 was over 8 million.

2.2 UNDP's assistance has been mainly oriented towards assisting preinvestment studies for water supply primarily for large urban areas. Invariably these studies have been entrusted to WHO, designated as Executing Agency. Preinvestment studies so far implemented with UNDP/WHO assistance in over two dozen developing countries have resulted in decisions to build new water supply and sewerage schemes worth about half a billion dollars.

2.3 IBRD and Regional Banks like IADB (Inter-American), ADB (Asian and African) and EDF (European) have so far been the principal International funding agencies for the construction of new water supply facilities and the improvement of existing ones. During the period from 1962–1975 both IBRD and IDA loans for Water Supply and Sewerage projects amounted to nearly $1170 million. IBRD works very closely with WHO under a Cooperative Programme for Water Supply and Sanitation and a somewhat similar Programme has also been established with ADB (African).

It would also be worth mentioning the significant contribution to the development of community water supplies made by a number of bilateral agencies like SIDA (Sweden), CIDA (Canada), FAC (France), KFW (Fed. Rep. of Germany), USAID (U.S.A.), DANIDA (Denmark) and OECF (Japan).

2.4 The role of both the UN and UNEP has been relatively limited, UN being mainly concerned with the development of water resources and UNEP with monitoring and surveillance of water quality. As for WFP it assists community water supply programmes in several ways including food aid as partial payment for wages as in Korea and Dominican Republic.

2.5 WHO's involvement in the field of water supply may be summarized as follows:

(i) *Country and Inter-Country projects*—there are well over 200 projects presently in operation. They include primarily strengthening of environmental health services in the country, development of community water supply programmes and training of nationals.

(ii) *Pre-Investment Planning*—so far nearly 60 pre-investment projects for water supply have either been completed or are in operation in around 50 countries. They include generally master plans and preliminary engineering and feasibility studies. These projects are in most cases financed by UNDP and as mentioned earlier they have generated investments worth nearly half a billion dollars so far.

(iii) *Sector studies*—these are studies carried out under a WHO/IBRD Cooperative Programme in which the existing situation of the water supply and sanitation sector is closely studied. So far sector studies have been carried out in over 28 countries and they have proved to be a valuable tool for national planning and sector improvement.

(iv) *Transfer of Information and Methodology*—this activity is carried out through training programmes, formalised courses either in schools of sanitation or universities, fellowships, in-service training, publication of documents, guidelines etc., WHO has also established a network of collaborating centres for community water supply where studies are carried out and information is collected and disseminated.

(v) *Surveillance of drinking water quality*—in collaboration with governments, attention is focused on institutional development for country-wide surveillance of water quality.

3 International collaboration

Five different situations

The scope and degree of involvement of International Agencies in the field of water supply in developing countries varies and depends on conditions and the situation in the country concerned. An important factor to be considered is the ability of the country to collaborate and to develop a programme. Five factors listed below have been identified for the purpose of assessing the country's status and preparedness to secure international collaboration and develop community water supply programmes. The factors shown vertically in a matrix represent country's inputs for programme planning and implementation. These inputs have been related to real life situations in five different hypothetical "countries" shown horizontally in the matrix.

Each situation represents a typical "case-study" and each case has been later discussed on its own merits. They each provide a different picture of how under different existing conditions a country's performance in the field of water supply is assessed and international collaboration accordingly provided. It is evident that the "Yes" and "No" are qualified statements and should be considered with sufficient margin of flexibility. The purpose of the following presentation is to stimulate discussion.

Country "A"

Existing conditions: Government has a policy for the development of water supplies; the country has an organisation responsible for water supplies; it also has manpower available; and a population motivated to develop water supply programmes. However, it lacks financial resources.

The above situation could well be described as existing in some Latin American countries like Columbia.

It is evident that the existing conditions are quite favourable to develop a programme and any International Agency should be willing to collaborate. The lack of funds, though a handicap, could be overcome by stimulating community approach and participation and by the use of simple and effective technology by the locally available manpower.

Foreign financial assistance either international or bilateral should be readily available to match existing resources.

Another aspect to be considered is that adequate operation and maintenance of the schemes is expected to be assured in the country and very likely the water system will be viable and self-supporting. The fact that the country has a government policy and an institutional infrastructure is a great step forward for the development and implementation of programmes.

Country "B"

Existing conditions: The country possesses a government policy for water supplies as well as skilled manpower and a local population well motivated to develop a programme. However, it lacks both an infrastructure and financial resources to undertake a programme.

The above situation, though perhaps not exactly similar, could be found in some countries in South-East

Inputs	CASE STUDIES				
	Country "A"	Country "B"	Country "C"	Country "D"	Country "E"
Government Policy	YES	YES	YES	NO	YES
Infrastructure (Institutional aspects)	YES	NO	NO	NO	YES
Financial Resources (external and internal)	NO	NO	NO	YES	YES
Manpower (skilled)	YES	YES	NO	NO	YES
Community Motivation	YES	YES	YES	NO	YES

Asia like India and Pakistan. If the country wishes to generate foreign funds, particularly from International lending agencies, a prerequisite would be the establishment of an effective organisation, management and financial structure for the water supply sector. Unfortunately very few developing countries have a strong central or regional authority to deal with the water supply sector. In most of the sector studies carried out by WHO/IBRD lack of an institutional structure has been found to be a major constraint for the development of water supply programmes.

Establishment of a sound and effective institutional structure is always one of the main objectives of every WHO assisted preinvestment project and this was the case in Ghana where as a result of the WHO/UNDP project for Accra and Tema the Ghana Water Supply and Sewerage Authority was established. In situations like that of Country "B" once an effective infrastructure has been established it is very likely that, with the manpower available and a motivated community, foreign funds would be forthcoming and help in programme development and implementation as happened in Ghana and in a few other countries.

Country "C"

Existing situation: Though the Government has a water supply policy and the community is keen for the development of their water supplies, the country has to struggle with no financial resources, no infrastructure and above all no skilled manpower.

The above situation is somewhat typical in many of the twenty five countries identified as "least developed" by UNDP.

In such situations training of local personnel and methodology and technology transfer should be the mainstay of any water supply development programme. Training should be geared to the local needs and conditions with objectives clearly defined.

Any international agency interested in collaborating in such countries would always ensure that once the collaboration ceases the nationals are in a position to take over and carry on the programmes on their own. This would include both planning and engineering designs of the systems as well as the study and implementation of organisation, management and financial aspects involved with a view to ensure system viability. In countries in this category like Nepal, Yemen Arab Republic and a few others in Central and West Africa, UNDP financed preinvestment projects for which WHO was the executing agency, contributed to the establishment of an infrastructure, to the training of nationals and subsequent international assistance for financing the investment programme.

In many countries facing the situation and conditions as in Country "C", it is often necessary for the government to subsidise the scheme. The country may also require operational assistance during the initial stages, which would include the study of an adequate tariff or rate structure compatible with the interests of both the consumer and the undertaking. House connexions are often a luxury that only a few can afford, and the bulk of the population would have to rely on standpipes. It is important in such cases that water rates be carefully studied so that even the poor consumer may have access to "safe" water. The health aspects and impacts of a "safe" water supply should in such cases over-ride financial considerations.

Country "D"

Existing conditions: All that the country possesses are financial resources. It lacks skilled manpower, a water supply policy, an infrastructure and a community that is motivated.

This case is typical of many "oil rich" countries.

Here again although initially the country will have to depend heavily on expatriate assistance it should nevertheless make every effort towards attaining self-sufficiency.

Foreign donors and agencies providing assistance either international, regional or bilateral on which the country has to depend have a great responsibility and an important role to play. They should ensure that the engineering designs and schemes that are implemented are suitable to the conditions in the country and tailored to the needs of the community served. As an example it would indeed be most unfortunate if sophisticated equipment and machinery which is expensive and difficult to operate and maintain is thrust on these countries, only because they can afford the price.

Crash courses and in-service training programmes for sub-professionals (plumbers, fitters, mechanics etc.), and for professionals (engineers, managers etc.), should be given every priority from the start of the programme. Training with a view to establishing a strong national expertise should be the main objective. It is through such a nucleus of trained national staff that an infrastructure could be built and decision and policy makers created within the country to establish sound policies and programmes.

Country "E"

Existing conditions: The country has all that is required to develop a sound water supply programme. Namely, a water policy, an infrastructure, funds, manpower and a community that is motivated.

Such countries are by and large very few in the developing world, and could be spotted in Latin America (Brazil).

It is a well known fact that through the years these countries have developed a sound infrastructure and a competent cadre of both professional and sub-professional staff. Initially also they received large financial assistance possibly from Regional or International Banks. However, all these inputs would have been to no avail without a strong government support and a sense of community involvement and participation which was an integral part of every programme and should be an object lesson to many other countries.

It may be mentioned in this connexion that when outside assistance ceased, countries in this category started to generate their own funds and in some cases establishing revolving funds and other ways of securing income necessary for the development of their programme.

4 Prerequisite for successful programme development

The philosophy and approach for programme development and implementation under different situations and conditions was discussed above. There are, however, a few factors that could contribute to the success of programme development some of which are outlined below.

4.1 Government support

A strong government commitment and support is a major prerequisite for the success of any development programme. Clear examples are found in countries like the Dominican Republic and Brazil as mentioned above where by establishing a national institution and policy and implementing a dynamic programme of education, consultation, stimulation and organisation of the communities benefiting, success was assured. Close technical supervision at all levels, fullest exploitation of economies of scale wherever possible and the development of financial and managerial procedures tailored to local needs are among the items deserving full Government support.

4.2 Institutional Building — Infrastructure

Experience from our early ventures in the development of water supply programmes has shown that the most common reason for failure in follow up on investment in developing countries was the absence of a sound and viable organisational structure.

The provision of water supplies requires substantial amounts of capital and the physical plans represent a large portion of the countries' infrastructure assets, both to protect the investment from deterioration and to provide for its effective operation and maintenance. Continuous and sound management is therefore essential.

It is important to recall that, in developing countries, the infrastructure for planning, implementation, management, administration and evaluation of national programmes falls within the purview of more than one ministry. Hence major coordination is required to maximise the efficiency of this important area or function.

4.3 Community participation

It is becoming increasingly evident that one of the main reasons for the slow development of water supply programmes is the lack of community participation and motivation. On the other hand it is also becoming abundantly clear that any visions of a global solution of the community water supply problem will remain a wishful dream without a strong involvement of the community to be served and a determined self-help approach.

The first step in any community water supply programme should be to determine the willingness and interest of the community in the undertaking, secondly the capability of contributing to the costs of construction in labour and cash and thirdly, the capability of the community of managing the system and collecting revenues for operation and maintenance.

Implicit in the above is the great need for community workers, health educators, sanitarians, etc., to motivate people and develop a demand for water supply service in the community. Last but not the least ways and means should also be found to motivate the policy and decision makers. This could be achieved in one way by producing well prepared project feasibility reports in which cost/benefit and financial/economic studies are well documented and easy to understand with a view to assisting the decision making process.

4.4 Resources

4.4.1 Financial resources

Lack of funds is generally a common constraint facing every developing country and the only alternative it has is to secure outside financing.

It often happens, however, that the country does not succeed in trying to obtain foreign funds. One of the reasons is poor project formulation and presentation. Countries should be made aware that a prerequisite for assistance to cover the foreign exchange component of a project is a comprehensive and well prepared preinvestment survey that would include both economic and engineering feasibility studies. Careful presentation and selection of alternatives and a thorough discussion of the economic and financial aspects involved is essential for lending agencies and decision makers. Projects so presented have found acceptance in national development plans and have attracted outside investment from national, international or bilateral lending agencies.

Cooperatives, housing banks, lotteries, revolving funds, to name a few means of support have all been used successfully in securing funds for water supply programmes. Where the economy level is low the government is obliged to subsidise. However, operation and maintenance must be accepted by the community as their responsibility.

4.4.2 Manpower resources

The importance of skilled manpower in the development of any water supply programme is most evident and has been dealt with earlier. Unfortunately the majority of countries in the developing world are faced with the problem of lack of competent personnel both at the professional and sub-professional level. Though the involvement of UN Agencies in improving the quality and numbers of manpower available for water supply work has been consistent yet it is far from meeting the needs of the countries.

Training of nationals both at professional and sub-professional level has always been one of the main objectives of practically every WHO assisted environmental health project. Training for sub-professionals is generally carried out at schools of sanitation. As for professionals, courses in sanitary engineering have been conducted at nearly 25 engineering centres which include the Universities of Nairobi, Lagos, Zaire, Kumassi, Ankara, Rangoon, Bandung, Bangkok, Chile, Bolivia, Peru, Lahore, Tehran, Rabat etc. Funds for training purposes are in many cases provided by UNDP and also by UNICEF for equipment and supplies.

Though methods of training vary in different countries, the type of personnel required is about the same and includes civil and sanitary engineers, health inspectors, water treatment plant operators, mechanics, plumbers, masons, carpenters, administrators, accountants and clerks. To train such a vast quantity of personnel also requires availability of professors, trainees, educational institutes, teaching materials and funds which are woefully inadequate in most developing countries. On the other hand, one has also to guard against merely classroom formal training with no "in-service" experience under supervision.

Considerable progress has been made, particularly in Latin America and in some countries in South-East Asia. However, concerted efforts by the respective governments in collaboration with International Agencies will be required for most countries for many years to come.

4.5 Technology

It can be safely assumed that in practically every situation related to planning and design of water supply projects the technical aspects and the technology involved are known and there is abundance of knowledge and experience in the field. All the same it often happens that systems are poorly planned and designed and the costs inflated to the ultimate detriment of the country concerned.

In many instances alternatives are badly selected with no economic and financial considerations not to speak of engineering aspects and the technology applied. This ultimately reflects in the equipment and materials used being far from conducive to conditions prevailing in the country. In this connexion the WHO International Reference Centre for Community Water Supply at The Hague through its thirsty collaborating institutions in as many countries ensures that the right knowhow and technology is made available to them on request.

The main criteria for the selection of systems should be the ability of the community to support the system both financially and otherwise. In other words, the systems should be planned and designed on the basis of what the population is capable of paying in terms of capital recovery and costs of operation and maintenance, and not left to the whims of the consulting firms and manufacturers of equipment whose interests may not be those of the community served.

5 Conclusion

This paper was intended to illustrate the existing global situation as regards community water supply and efforts made by International Agencies to collaborate with countries for the achievement of their targets for programme development and implementation.

Though great strides have been made in recent years conditions continue to be far from satisfactory and it is very unlikely that the global targets will be met. Some factors and constraints hampering sector development have been described above, and these are primarily: insufficient allocation of finance by governments, lack of trained personnel and inadequate external assistance.

Prospects for the immediate future are unfortunately not very bright. With the ever increasing escalation of prices, world wide inflation and political instability in many developing countries, with the consequent readjustment of priorities for assistance by the rich countries, a heavy onus has fallen on the countries themselves. Are they prepared to face the challenge? It is a question left open to be answered.

Ways, however, will have to be explored of making current investment go further and serve more people. This could perhaps be achieved by encouraging the use of simple technology, self-help projects, appropriate institutional structures, re-orientation of training and optimum utilisation of manpower.

It is also evident that International Agencies cannot isolate themselves from the world-wide financial trends. Rising costs have also affected International collaboration in programme development. However, one would hope this will only be a passing phase and it will not be long before the tempo is restored and stepped up to continue our long and fruitful collaboration with countries as their partners in development.

Development of water supply in the metropolitan city of Lagos

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Introduction

Water of satisfactory quality and in sufficient quantity is a prime requirement of society today. Impure or polluted water in these days of world-wide population explosion and unprecedented industrial growth is rapidly becoming the number one problem of advanced as well as emerging nations. The housewife is no longer satisfied with the quantity of water she can collect from the surface streams or the private shallow wells in her premises, and if she is, she definitely cannot rely on its quality. Not only does the remotest village in emerging countries have its own particular problems of supplying its inhabitants with adequate water, but so also do the great metropolitan cities of the world. The emphasis therefore in recent years has been to search for new raw water sources, treat the water thus obtained to acceptable standards and transport the treated water through pipelines to places where needed in sufficient quantity. In Lagos, the production and distribution of potable water became a public matter early in this century.

Social and economic position of Lagos

Metropolitan Lagos occupies a unique position in the Federal Republic of Nigeria. Part of Lagos recently delineated a special area still serves as the Federal Capital, while Ikeja, within sixteen kilometres of the city serves as the Lagos State Capital. Notwithstanding the fact that a new Federal Capital is now being planned, Lagos will still continue to serve as the commercial nerve centre of the country. Most of the sea-borne cargoes as well as the people coming in or going out of Nigeria pass through Lagos. The convergence of trade routes on Lagos led to its remarkable growth and has brought about a continuing migratory trend into it. In 1973, about 75% of the country's total imports and approximately 50% of the total exports (excluding crude oil) went through Lagos and Apapa ports.

Again, the importance of Lagos for the economy of Nigeria can best be described by the share of the city in relevant economic values and quantities of the country as a whole. Although there are no recent data available for Lagos Metropolitan Area, the figures for the Lagos State are reliable indicators because of the absolute dependence of the immediate hinterland on the performance of the city. The following table (Table 1) gives a comparison of some important economic indicators for Nigeria and Lagos State industries, wholesale and retail trades. All figures indicate a predominance of Lagos State in absolute quantities (numbers of establishments and employees, wages and salaries, sales) as well as the average per unit values (employees per establishment, wages and salaries per employee).

Because of the numerous roles which Lagos plays in the economic and social life of the Federation, it forms a target for job seekers with attendant problems of inadequate services notable among which is water supply.

TABLE 1

Industry and Trade in Nigeria and Lagos State, 1972

Economic Indicator	Nigeria	Lagos State	Lagos Nigeria (in %)
(a) *Industries*			
No. of Establishments	861	227	26
No. of Employees	61 000	26 000	43
Employees per Establishment	71	115	—
Wages and Salaries (million ₦)	62,9	30,2	48
W & S per Employee (₦)	1 030	1 160	—
Sales (million ₦)	1 607	551	34
Value Added (million ₦)	386	135	35
VA per Employee (₦)	6 330	5 190	—
(b) *Trades*			
No. of Establishments	794	207	26
No. of Employees	53 600	22 200	41
Employees per Establishment	68	107	—
Wages and Salaries (million ₦)	58,4	27,3	47
W & S per Employee (₦)	1 090	1 230	—

Bibliography: Economic Indicators, Vol. 9, No. 8, August, 1973, Federal Office of Statistics, Lagos, Nigeria.

All figures for establishments with 10 Employees and more only.

Existing industrial estates are expected to be almost completely occupied during the next few years and additional areas planned both for future industrial and housing development between 1975 and the year 2000 may practically be equal to the land area now developing or being developed.

Climate and geology

Lagos enjoys a littoral type of climate. All the year round, the temperature hardly falls below 18°C and averages about 27°C with a seasonal variation of about 5°C. Relative humidity drops with the rise in temperature to about 70% in the afternoon during the dry season. The annual mean rainfall of about 1 800 mm occurs primarily in May–July and September–October periods.

The Metropolitan Lagos area consists of outcrops of two main geological formations:

(a) The coastal plain sands which form the low gently sloping dissected uplands reaching in places a height of about 61 metres and

(b) Recent coastal deposits which form the extensive and swampy alluvial plains of the major rivers and creeks along the coast overlying the coastal plain sands.

Historical background

The history of Lagos water supply dates back to October 1910, when construction of Iju Water Works commenced under the leadership of Mr. H. F. Peet, who later became the Director of the Public Works Department. On 1st July, 1915, the Waterworks was publicly opened by

Lord Frederick Luggard, the then Governor General of the Colony of Lagos and Protectorate of Nigeria, who turned on the knob of Idumota Public Fountain and collected water in a silver cup from which he drank. There were over 300 such fountains located all over the city of Lagos at that point in time.

In 1915, when the Waterworks was commissioned, the design capacity was 11 250 m³ a day, the maximum daily consumption being 2 900 m³ for a population of approximately 100 000 inhabitants. Water demand rose steadily until 1938 when it reached 13 500 m³/d for a population of 156 000 people.

The headworks, located at Iju, 32 kilometres to the north of the City of Lagos, consisted of an intake structure on River Iju, a 900 mm cast iron raw water main, two steam-driven low lift pumps, four coal-fired Lancashire boilers, two horizontal flow sedimentation tanks and eight slow sand filter beds. The filtered water was stored in an 18 000 m³ tank and pumped to a 54 000 m³ distribution tank at Shaga about one kilometre from the works premises through a 700 mm diameter cast iron rising main by two steam-driven high lift pumps. From Shaga reservoir, a 700 mm diameter cast iron gravity main took off in a north-south direction and continued over a distance of 28 km. From this other feeder mains varying in size from 75 mm to 400 mm diameter branched off to form the distribution networks. Correction of pH is by the addition of hydrated lime, sterilization of filtered water is by the addition of chlorine.

The water supply authority, at its inception, was being managed as a division of the Public Works Department and the immediate control of the waterworks at Iju was under a Head Pumper, later passed on to a Chief Inspector and then to a Superintendent. The distribution centre was located in the city of Lagos. Supply was not metered except for industries and corporations.

With the expansion of the city to the mainland, the demand for water increased and to cope with this a 600 mm diameter cast iron gravity main was laid to Yaba Roundabout and commissioned in 1943.

The population of the city of Lagos and its environs experienced a phenomenal growth between 1945 and the early fifties and during this period districts on the islands were undergoing development while extensive development and resettlement schemes were also being experienced on the mainland districts. The original source of raw water—Iju stream—was found inadequate in meeting the demand for water and a new surface source—Ogun River—about 6 km from the treatment works was developed and commissioned in 1954, bringing the total production capacity to 50 000 m³/d.

Following the attainment of Independence in October, 1960, urbanization and industrialization created the necessity for the expansion of water supply despite the development of 1954. Further developments were embarked upon between 1954 and 1964 bringing the production capacity to 108 000 m³/d. The highlights of these developments were:

(a) Replacement of the old coal-fired boilers with two oil-fired horizontal boilers.

(b) New pumphouse at Iju Intake with four electric pumps.

(c) Additional intake and pumphouse on Ogun River.

(d) 900 mm diameter steel raw water main from Ogun to Iju.

(e) (i) sixteen vertical flow sedimentation tanks commissioned in 1960.
(ii) four vertical flow sedimentation tanks commissioned in 1964.

(f) (i) six rapid gravity filters, commissioned in 1960.
(ii) six rapid gravity filters, commissioned in 1964.

(g) Three electric high-lift pumps each with a capacity of 1 350 m³ per hour.

(h) 1 000 mm diameter steel treated water rising main from Iju to Shaga reservoir.

(i) 1 000 mm diameter steel gravity main from Shaga reservoir to Lagos.

Master plan and feasibility studies—1964

In 1964 a firm of consultants was commissioned by the Federal Ministry of Works and Housing to carry out a feasibility study of the Lagos Water Supply Expansion.

In brief summary, the projected total cost over the ten year period of 1966 to 1975 amounted to ₦36,8 million including ₦2 million interest during construction.

Forecast on Demand

The demand forecast was as follows:

Year	Population	Estimated average demand 1 000 m³	Maximum demand 1,5 × average demand 1 000 m³
1964	916 000	167*	250
1970	1,5 m	270	405
1975	2,1 m	388	582
1985	3,5 m	630	945

* In March 1964, Lagos Water Supply supplied 80 550 m³/d to an estimated population of 916 000. This was an average of 87,75 litres per capita per day as compared with the recommended design figure of 180 l/cap.d.

Recommended construction

The proposed construction to serve the design populations of 1970 and 1975 included:

(i) Impoundment on the Ofiki River, a tributary of River Ogun, to ensure a safe yield at Akute.

(ii) Construction of new raw water intake and an incombustible rubber dam at Akute.

(iii) Pumping and transmission mains.

(iv) Enlargement and modernization of the treatment plant at Iju.

(v) Construction of additional clear water pumping, transmission and storage facilities.

(vi) Provision of additional capacity in the gravity distribution system.

Additions and improvements to the distribution system

1. Increased storage capacity at Shaga from 27 000 m³/d to 81 000 m³/d, in the first five year period to 1970. A 56 000 m³ tank was to be added by 1975, and a third extension of 112 000 m³ by 1985.
2. Additional elevated storage of 30 000 m³ was to be provided in six steel tanks on towers of 27 m to 33 m in height.
3. The principal main through a water district grid system was to be served by a main of minimum of 300 mm diameter.

4. Interconnection of the new feeder mains with existing mains at principal intersections was to be made in order to increase the efficiency of the whole distribution system. During the maximum demand conditions, residual head in the system was anticipated in the range 16 m to 18 m.
5. Metered connections for all consumers were considered the best and most equitable method of apportioning charges for water consumed and in reducing water losses.

Although it would appear that the Federal Ministry of Works and Housing had approached U.S.A.I.D. to finance and implement the first part of the Consultant's recommendations, which was the immediate improvement of the system to 225 000 m^3/d, no effective negotiations were concluded. The result was that the next few years that followed witnessed a period of "water crisis".

Crash programme expansion

With the creation of 12 states in Nigeria, the Lagos State Government took over the responsibility for the Lagos Water Supply from the federal government on 1st April, 1968, and quickly focused attention on alleviating the acute water shortage in Metropolitan Lagos. In May, 1969, the Federal Ministry of Works and Housing commissioned another firm of consulting engineers through the U.S.A.I.D. to submit proposals for the implementation of Phase I of the original report. The proposal submitted in November, 1969 provided for an increase of production to 240 000 m^3/d and envisaged construction works to commence in January, 1971. Because of the time factor involved in this proposal, the Lagos State Government opted for a crash programme which allowed a parallel operation of design and construction of the project, thus shortening the implementation period.

Thus in late 1969, proposals were invited for the provision of engineering services for the crash programme for the expansion of water production to 200 000 m^3/d. The scheme chosen, estimated at ₦4,80 million, comprised:

(a) New intake and pumphouse on Ogun River at Akute.
(b) Modernization of existing horizontal flow sedimentation tanks.
(c) Modernization of existing rapid gravity filters and provision of additional filters.
(d) Construction of a sludge thickener.
(e) Additional vertical flow tanks.
(f) Construction of power house and provision of stand-by generators.
(g) Clear water pumping station, pumps and piping.
(h) Electrical equipment.

A study of the losses in the distribution network was also included in the scope of work of the consultants. The contract for the construction of civil works commenced in 1970 and, simultaneously, tenders for the supply and erection of filter equipment, pumps, electrical equipment, generators, pipes and fittings, were invited and reviewed. However, as a result of long delivery periods quoted by the majority of the overseas tenderers, a high level delegation was sent to Europe and successfully negotiated the shortest possible delivery times and better financial terms with the suppliers.

Throughout the construction period, minimum disruption to existing services was ensured and by December, 1972, shortly before the commencement of the 2nd All African Games in Lagos, water production had reached the figure of 150 000 m^3/d. The programme was finally completed in May 1973. This was followed by a systematic tackling of the bottle-necks in the distribution network.

Lagos water supply expansion programme Phases II and III

With the completion of the crash programme, it became obvious that a more permanent and comprehensive solution was necessary, and accordingly, in 1973, Phases II and III were designed.

Phase II envisages the construction of waterworks at Isasi, about 25 kilometres west of the city on the River Owo with a capacity of 160 000 m^3/d as well as improvements in the production capacity of Iju Waterworks. There will also be construction of additional overhead tanks in the distribution networks, and a low level weir at Akute on the River Ogun. Both improvements will provide 360 000 m^3/d in total by 1978. The above intermediate steps are designed to provide the stop-gaps required for the construction of the main scheme which represents Phase III.

Phase III envisages the construction of Adiyan Waterworks with a capacity of 950 000 m^3/d by 1994 to be achieved in three equal stages namely, 1980, 1985 and 1994. While the consultants were busy with the collecting of preliminary design data for Phase II, the Lagos State Government was commissioned by the Federal Government to provide a scheme capable of supplying 18 000 m^3/d for the Black Arts Festival Village and the Federal Housing scheme at Amuwo Odofin, adjacent to the village site. The project, which commenced in November, 1974, and was completed in December 1975, was constructed at Isasi on the River Owo and represents Phase II Stage I of the expansion programme.

Development concept report

Whilst the activities of Phase II Stage I were being vigorously pursued, the consultants' reports on water demand analysis, for Phases II and III, hydrological investigations (Phase II Expansion), and final development concept (Phases II and III), were submitted during the latter half of 1975. These were followed by the preliminary design for construction of Phase II Stage II.

Water resources

Among the resources of raw water investigated were:

(a) Groundwater
(b) Brackish water from the coastal lagoons, and
(c) Surface water from rivers.

In the selection of water sources for the development of Lagos Water Supply, the criteria used were:

(i) Early availability of water,
(ii) Relatively good knowledge of the technical parameters of the source,
(iii) Guaranteed security of the supply, and
(iv) Cost required for the development of the source.

Out of all the resources investigated, the source best fulfilling these conditions was surface water from rivers if reasonable storage could be made. River Ogun is the only river where adequate storage is feasible by the construction of a dam across its main tributary. Although in the case of the River Owo, geological conditions were found to be unfavourable for the construction of a dam, a weir against the intrusion of brackish water will make it possible to withdraw its total safe yield and it has been selected as a second source. A detailed study of underground resources was commissioned in 1975 to establish effective control of ground water abstraction in Metropolitan Lagos which is now in the hands of private commercial firms and institutions and also with a view to providing detailed and accurate data on the hydrogeology of the area for quick borehole construction take-off when an immediate water supply augumentation is needed.

Population projection and water demand analysis

The existing population projections prepared by different organizations and institutions vary considerably and reflect the poor statistical data available. For the future, three possible trajectories of Metropolitan growth were considered for Lagos as a whole. These present an extrapolation of current trends, and two different assumptions about the extent to which current rates may decline in time. Extrapolation of current trends implies that the population of Lagos Metropolitan Area will double every decade and reach a total of more than six million in 1985 and nineteen million by the end of this century.

This scale of growth may be regarded as very unlikely for two main reasons. Firstly, it would imply a constantly rising rate of immigration mainly from the neighbouring Oyo, Ondo, Ogun and Bendel States, which cannot be sustained over a period of time in view of the considerable difference that exists between the rate of growth of Lagos and the latter. It cannot increase for very much longer at the rate which would be required to sustain an overall immigration rate of 5,3 % per annum for Lagos. The scale of population loss is considerable in terms of the main origin of immigrants to Lagos and for Oyo, Ondo and Ogun States, it represents nearly 10 % of their estimated 1973 population.

Secondly, the evidence of growth of the cities in other parts of the Third World generally shows that the extremely high rates of growth which accompany the early phases of urbanization occur only for a short period of time. Consequently, it is necessary to consider possible levels of reduction in the rate of growth that has been experienced over the last twenty years. This largely involves a falling off in the rate of immigration, but it should be recognized that a reduction in immigration rates will also lead to a reduction of natural increase in the medium term as the proportion of young adults in the population is reduced. In the short term and even in the medium term, the effect of falling rates on the overall growth of population will be small relative to the scale of growth involved. For example, if the rate of immigration is effectively halved and the annual rate of population growth of Lagos during the 1973–1985 period is only 6 % as against the 8 % found in past trends, the present population of the State as a whole will still be doubled by 1985, to a total of about 5 million. In the absence of any other information pointing to a more drastic fall in rates, it is only realistic to plan for at least a doubling of the population over the 1973–1985 period and to reduce the growth rates only after 1985. From the evidence stated above, it seems logical to choose 5 million in 1985 and 9 million in 2000 as the basis for planning considerations (Table 2).

TABLE 2

Projected Population Growth for Lagos Metropolitan Area

	1973	1985	2000
	Thousands		
1973 Population extrapolated at 8% per annum to year 2000	2 400	6 000	19 000
1973 Population extrapolated at 6% per annum to 2000	2 400	4 800	11 600
1973 Population extrapolated at 6% per annum to 1985, and at 4% per annum between 1985 and 2000	2 400	4 800	8 800

Evaluation of future water demands

Assessment of the future water demand is based on the analysis of the past periods, evaluation of future population growth and assumed future average living standards of Lagos inhabitants with respect to both social and economic points of view. Future water demand is divided into three main categories depending on the type of the consumer, whether domestic, industrial or commercial. The figures indicating the water demand for a particular consumer category include all water losses occurring in the system after the main reservoir.

As a result of the rapid expansion of industries, the public water supply system was found inadequate and a considerable amount of water for industrial purposes is drawn from private boreholes. This situation cannot be accepted as normal, and it is assumed that industrial water demand will continue at an expected growth rate of 7 % per annum. As soon as the supply can cope with industrial requirements, the present high percentage of industrial borehole water supplies can be expected to be reduced; commercial and public water demand is estimated at 20 % of domestic demand. The development concept is based on the following water requirements contained in Table 3 below:

TABLE 3

Lagos Metropolitan Area Water Demand Projection

		1973	1985	2000
Projected Population persons		2 400 000	5 000 000	9 000 000
Domestic Water Demand	mgd	40,0	86,0	168,0
	1 000 m^3/d	182,0	390,0	763,0
Public and Commercial Water Demand	mgd	8,0	17,0	34,0
	1 000 m^3/d	36,0	77,0	155,0
Industrial Water Demand	mgd	4,0	15,0	40,0
	1 000 m^3/d	18,0	68,0	182,0
Average Daily Water Demand	mgd	52,0	118,0	242,0
	1 000 m^3/d	236,0	535,0	1 100,0

Water Charges can also be split into two components:

(i) Water supplied to the Lagos City Council Area is billed on a fixed charge of ₦80,00 per annum payable quarterly.

(ii) Metered usage at the rate of 55 Kobo per 1 000 m^3 for other users.

During 1974, the yearly average of water supplied to the City Council totalled 320,29 million cubic metres which would have resulted in a direct revenue of about ₦1,8 million at the rate of 55 Kobo per 1 000 m^3. This amounted to a subsidy of ₦1,00 million by the Ministry to the Lagos City Council. Water supplied to the L.C.C. was estimated at 65 % of the overall annual supply. The balance of the supply should yield a revenue of ₦970 000. However, actual revenue collected was only ₦383 000. From the foregoing, if the entire service area were charged on metered rates, the shortfall in revenue in 1974 was at least ₦1,50 million.

Water sales and profitability

The Water Division operates at a very substantial loss which is increasing each year. This can be clearly seen in the following table (Table 4) which shows the estimated trading positions for 1974/75 and 1975/76 financial years.

TABLE 4

	1974/75 Approved Estimate ₦1 000	1975/76 Approved Estimate ₦1 000
REVENUE	720	920
EXPENDITURE		
Salaries and Wages	593	970
Maintenance and Running Costs	2 000	2 000
Water Connection Services	90	90
	2 683	3 060
DEFICIT	1 963	2 140

The deficit for 1974/75 would be greater than the figure shown for the following reasons:

(a) Actual revenue collected totalled ₦597 000.

(b) Actual cost of payrolls amounted to ₦864 000.

(c) Expenditure did not include for

(i) Gross charges from other divisions of the Ministry or other government agencies for services provided.

(ii) No account was taken of depreciation on fixed assets or interests on loans.

Financial resources

Seventy-five per cent federal contribution to water supply includes the following capital projects:

(a) Water supply expansion Phases II and III,

(b) Mains extension and restructuring,

(c) Water supply metering,

(d) Consultancy fees.

Again, the analysis of the Five Year Development Plan (1970–1975) also reveals that availability of funds has been no constraint on the performance of the Water Division as contained in Table 5 below.

TABLE 5

National development programme 1970–75

	Authorized (₦1 000)	Spent (₦1 000)	% Spent
Nigerian Overall	1 263 532	593 980	47%
Lagos State Ministry of Works and Planning	184 955	49 884	27%
Water and Sewage	31 376	5 675	17%

General problems

As discussed earlier in this text, with the federal government participation in the Lagos Water Supply projects, finance is not a constraint. A major constraint is lack of sufficient high level manpower and the low calibre of the low-level and intermediate operatives with the result that all available engineers are overloaded with responsibilities. However, with the new concept of management, Management by Objectives (M.B.O.), introduced in the country in 1975, increased efficiency in the organization is being realized.

Illegal sales of water, falsified meter readings, illegal tappings and unauthorized diversion of mains supply are rampant although with the recent creation of "complaint centres" within the distribution area these shortcomings are expected to be substantially reduced. Such complaint centres also serve as feedback for information on pipe bursts, malpractices by water supply employees and private plumbers. The feedback is instantly monitored to the operational centres for necessary action. Another major set-back in development, operation and maintenance is inadequacy of materials. For instance, there are only two firms in the country which produce asbestos cement pressure pipes. There are also few foundries. In this respect even when pipes are available, metallic joints and specials may not be available in sufficient numbers. This situation could have been tolerable under normal population growth but not with the present dimension of population explosion.

Existing primary networks are tailored along a north–south axis without booster stations in the system. Until very recently, Metropolitan Lagos had expanded only from the south towards the north hence the later settlements are constantly under low-pressure. The present development concept of water supply expansion has, however, taken care of such problems.

Conclusion

Since the execution of the crash programme in 1973, water supply in Metropolitan Lagos has been given greater priority than in the past. A global and more realistic approach through the long term concept report already accepted by Government has been staged for execution. At 1975 price levels, the sum total of the programme was estimated at ₦380 million exclusive of tertiary distribution networks.

It is obvious that a review of the present water charges is desirable at least to make the organization viable in respect of operational costs. Whether the organization remains an arm of the Ministry or operates as a separate entity in future, it is paramount that it should continue to render effective services to the consumers.

Water supply problems in Jakarta

by Irwin Nazir

Indonesian Water Supply Association

1 Introduction

The state of development of water supply facilities in most cities in Indonesia has been viewed as far from adequate to serve the rapidly growing population and its expanding activities.

Practically in almost all cities in Indonesia, water supply conditions had received little attention and facilities were not properly maintained during a number of years following the outbreak of World War II. This situation had led to deterioration of the then existing facilities, primarily pipelines and accessories of the distribution network, shortening their useful life expectancies and, in many cases, causing substantial decreases in capacity. Illegal service connections, beyond control of the local water authorities caused the conditions to become worse.

More serious problems regarding water supply have been encountered in the large cities where urbanisation has resulted in a high rate of population increase, with which are associated the development of concentrated areas of exceptionally high population densities, and of populated centres in the outskirts, exerting demands for water beyond the design capacity and the reach of the distribution system. People living in such areas must rely upon private water supply systems, or initiate self-help water supply projects, to meet their demand for water or make-up water, where the supply from the public water supply system is inadequate. Many of these private water supply systems are drawn from sources of questionable conditions.

Considerable efforts have been made to attract greater concern of the people regarding their participation in safeguarding potential water resources, considering that private water supply is still indispensable until every household can be served by the municipal water supply system, or before they can be served with adequate quantities.

Although the urgent need for potable water has been recognized, and the Government of Indonesia has embarked upon investment programmes to improve the situation, the limited funds in the budgetary allocation provided for the development of this sector of basic services and the very great demand for water supply improvements in the country have imposed difficulties regarding the selection of priorities and in establishing the extent of the development in each city that has been considered. Aside from the requirements of the very large number of cities, also to be considered in the development programme are the requirements of about 81% of the total population of more than 126 000 000, living in the rural areas. Urban water supply development has reached a total production of only 17 000 l/s, while it has been estimated that only less than 5 000 000 of the rural population will have access to potable water supply by 1976. Financial assistance, in most cases in the form of loans, has been provided by foreign governments and international lending agencies to help speed up the development of water supply in the country. However, considerable funds will still be required to finance further developments in the future.

In the administrative structure of the Government of Indonesia, four Ministries are concerned with the development of water supply. These are: the Ministry o Finance, Ministry of Health, the Ministry of Public Works and Power, and the Ministry of Interior. Water supply development in large and medium size cities is administered by the Ministry of Public Works and Power, while the Ministry of Health is more concerned with developments in rural areas and smaller towns, where initiation of water supply projects is required for the improvement of health conditions, and with quality control. Operation and management of water supply systems is left to the Ministry of Interior, through the local authorities. Although there is no delineation regarding the above mentioned responsibilities, it seems that this concept is being applied in the country. Other Ministries may also initiate their own water supply projects. However, such projects are normally attached to main projects developed by those Ministries.

The Indonesian Water Supply Association (*PER*satuan *P*erusahaan *A*ir *M*inum *S*eluruh *I*ndonesia—*PERPAMSI*)

The Indonesian Water Supply Association was established in April 1972, by the decision of the First Conference of Municipal Water Supply Enterprises, attended by representatives from 54 cities of Indonesia. This association was created in view of the similarity of problems faced by municipal water supply enterprises, and of the need for collaboration to develop autonomous business organizations based on sound principles of economics.

By the decree of the Minister of Interior of June 23, 1975, this association has been given recognition as a "semi-official professional organization", and placed under a board, chaired by the Minister of Interior, which will provide the association with general and technical directives. Other members of this board are: the Minister of Public Works and Power, the Minister of Health, and the Minister of Finance. By that decree, the Indonesian Water Supply Association becomes coordinatively affiliated with the "Inter Indonesian Municipality Organization" (*B*adan *K*erja *S*ama *A*ntar *K*ota *S*eluruh *I*ndonesia—*BKS–AKSI*).

2 The development of water supply for Jakarta

Jakarta City

Jakarta, which was founded in the 16th century and became later the capital city of Indonesia, occupies a land area of 657 km^2 on the northern coastal region of the Island of Java. This relatively flat coastal plain is dissected by rivers, originating at the mountainous area south of the territory, of which the Ciliwung River is the largest.

Average yearly rainfall varies from 2000 mm near the coast to 4000 mm in the mountainous region, concentrated during the wet season, which lasts generally from November till May. The wettest month is generally January. The dryest month is August, with an average minimum rainfall of $\pm 3{,}5\%$ of the yearly total. The number of inhabitants of Jakarta, which was 300 000 in

1920, and 800 000 just before World War II, has grown explosively to 4 576 009 in 1971 (as recorded at the time of the last census). The estimated population of 1973 is 4 915 265.

The Jakarta Water Supply Company (Perusahaan Air Minum Jaya)

The Jakarta Water Supply Company was established by the Decree of the Governor of the Special Territory of the Capital Jakarta in 1968, and has since then taken over the management of the water supply system from the Municipal Government.

Water Supply for Jakarta

The Jakarta main water supply dates from 1908, when water was drawn from 15 deep wells located in the southern part of the present city. The total production of these wells was 200 l/s (17 280 m^3/d).

In 1922, an additional supply of 300 l/s (25 920 m^3/d) was provided to the city, which had, at that time, an estimated population of 300 000. The source of this new supply is a spring at the foot of Mount Salak, about 65 km south of Jakarta. No treatment, except for disinfection, is applied to the water, considering its superb quality for human consumption. Unfortunately, the limited rate of supply from this spring necessitated recourse to other sources to meet the requirement of the city.

Further development of the main water supply system took place in 1953 with the initiation of construction of a water purification plant within the city. This plant, known as the "Pejompongan Plant" started its production in 1957 with a capacity of 2000 l/s (172 800 m^{3}d). Raw water is drawn from a canal (i.e. the "Banjir Kanal") which was originally constructed to divert flood flows from the Ciliwung River. With this plant in operation, the total rate of delivery of the then existing system was 2500 l/s (216 000 m^3/d), supplying 1 950 000 inhabitants of the city. The Pejompongan Plant was expanded with an additional capacity of 3000 l/s (259 200 m^3/d). Construction of the expansion facilities commenced in 1963. In 1970, the plant allowed an additional rate of delivery of 2 000 l/s to the city, while the 3000 l/s design capacity was fully utilized in 1975.

With a total capacity of 5 500 l/s (475 200 m^3/d), on an "average per capita supply" base, the quantity of potable water that could be provided seemed just adequate to meet the average minimum demand for domestic usage. This would have been the situation under satisfactory conditions of the distribution system. However, the present conditions of the system do not allow even distribution of the available water to all parts of the city. Neither does the distribution network cover the whole city, as is indicated in the attached figure. Based on a study carried out in 1972, the estimated population served by the system at the time of study was 48%, of which 55% (or 26% of the total population) was served by direct connection, and the remainder (22% of the total population) through water vendors. Of the area served, there are still parts that are receiving intermittent supply or poor supply.

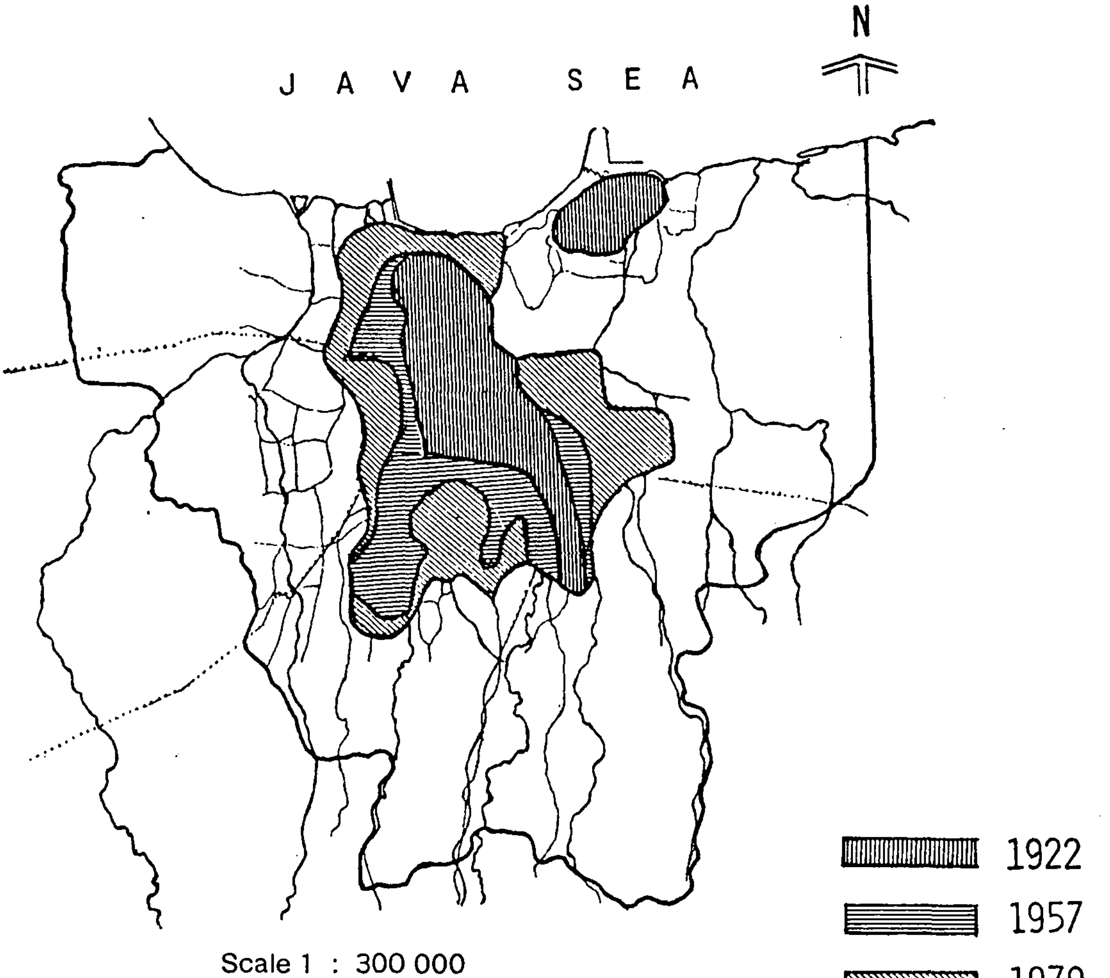

Figure 1 — Jakarta water supply service area

Supplementary water supply facilities have been constructed by the Water Company in parts of the city, outside the area served. During the period from 1968 through 1970, 35 deepwells were constructed. These wells are producing a total rate of flow of 70 l/s (6048 m^3/d). People who do not have access to water provided by the company have to maintain and rely upon private water supply systems. Some factories and housing estates use deep wells, while individual households have their supply from shallow wells. Privately owned deep wells have later been put under the control of the Water Company.

In the Master Plan for the Jakarta Water Supply System, several project stages have been proposed, which include effective utilization of the existing production capacity and the construction of new treatment plants and pipelines to meet the requirements of the city up to the year 2000. It has been estimated that to serve approximately 90% of the projected 8 300 000 people in the year 2000 (directly and indirectly), a production of 17,7 m^3/s (1,53 million m^3/d) will be required. This Master Plan is being used by the Water Company as a guidance in developing the water supply system for Jakarta.

3 Problems associated with the development of water supply for Jakarta

Under the conditions that exist at present, the Jakarta Water Supply Company has to make every possible effort to gradually improve the situation. Because of the interrelationship of the various problems that have to be solved, and realizing that any action taken to solve a certain problem may have a bearing outward the jurisdiction of the company, a careful study has to be conducted before any attempt is made to bring a solution to the problem.

Of the many problems that have been encountered, the most pertinent, and those which are considered as being of international interest, are described in this chapter.

Financial Problems

Major expansions of the water supply system of Jakarta have been financed from loans provided by governments of foreign countries and international lending agencies. Loan agreements are arranged by the central government, through the Ministry of Finance. Under this arrangement, interest and loan principal are paid back by the central government from general revenues.

Even for a large city, such as Jakarta, it is still difficult at present to generate significant revenues from water users which not only cover operation and maintenance costs, but also leave sufficient amounts for system expansions and to pay back loan principal and interest.

The condition of the existing system, primarily the distribution facilities, requires a major portion of the revenues to finance rehabilitation and replacements, and to cover maintenance costs. Considerable amounts have been spent on replacements to minimize water losses and hence increase the quantity of water that could be distributed to the consumers.

Other problems faced by the water company are those associated with the establishing of the pricing scheme. First, the difficulties in ascertaining the budget required to improve the condition of the system and to what extent this budget will have a bearing on the rate of water-use, and how this bearing should be distributed through time. Would it be considered fair to place this budget on the account of the present consumers? To solve these problems, a very elaborate inventory and accounting programme has to be conducted, which will certainly bring forward a variety of new problems. The second problem relating to the pricing of water is more of a socio-economic nature. The significant variation among water users in their income and ability to pay, the kind of water usage, and the benefits received by those supplied with water, are among the factors that have to be considered. To conduct an income survey would hardly be possible, while complete data may not be available at the revenue office. This situation necessitates the setting of an equal unit price of water for household usage, which may impose a heavy burden on the low income families, although these may be more fortunate than families who have to buy their water from vendors at higher prices. Quite a number of consumers of this income category are behind with their payments, facing the risk of being cut off from the supply. On the other hand, to the very rich consumers, who may not even care on whatever level the price of water is set, the established household rate may induce wastage of water, although a two-fold rate has been set for excessive quantities.

All the .factors of consideration that have been mentioned are also regarded in establishing other charges that are associated with water usage. Among these charges, the different rates of initial connection fee may be cited as an example. The philosophy behind the charging of initial connection fees stems from the fact that the selling price or the value of a premise would increase remarkably, to the benefit of the owner, when this premise is connected to the city distribution system, and that benefit could best be contributed to the funds required for financing the overall development of the water supply system. This concept of charging initial connection fees has lately become a subject of discussion regarding its justification.

Another source of income of the Water Supply Company is the sale of water from privately owned artesian wells. If the discrimination of charges for usage of water supplied by the city water supply system is based on the kind of usage, the basis of charging well owners for the water drawn from their wells is the location of the well with respect to the service area of the city distribution system. In the present pricing scheme, water from wells located beyond the reach of the city distribution system is charged 25% of the rate of water supplied by the water company, while the rate within the service area of the city distribution system has been set at 50% of the rate of city water. Charging for water from privately owned wells has been in effect only recently, primarily for the purpose of controlling and preventing excessive withdrawal of groundwater, which is of course justifiable. The discrimination of water rates, however, should be viewed from other points.

Managerial and Social Problems

The change of status of the water supply authority in Jakarta has given more autonomy in managing the water supply system. Without disregarding the considerations that relate water supply to social needs of the community, management of water supply has since then been based on the concepts of business organization. Although this connotes something materialistic, it has been considered timely that a water supply undertaking be self sustaining.

The main problem faced by the managing staff of the Jakarta Water Supply Company under its present status as a (local-) government enterprise is to find the most appropriate system of management, under the existing conditions. It seems that the company has inherited the problems that have been faced when the water supply system was administered by the Municipal Water Supply Department, and that the phrase (if there is such): "the property of the government is also the property of the people" seems still to be the principle of a number of consumers, in the sense that one has not to pay for what one owns.

Naturally, the company realizes that part of the problem of management is being caused by deficiencies in the services extended to the consumers, primarily the quality of service, such as: the limited, or intermittent, supply due to the shortage of production, or because of excessive usage of water in areas where the supply conditions are favourable; inadequate pressure in the supply lines because of leakages and illegal service connections, which further creates tendencies for the installation of in-line pumping facilities by consumers and thereby affects the supply to others; and still many other shortcomings.

The deficiencies created by the shortage of production and the improper conditions of the system, the attitude of a number of consumers, the insufficient amount of revenue that could be generated to rehabilitate the system, and the inadequate number of personnel of the company that are required to control the situation, could be considered as elements of what we may call the vicious circle of problems which has rendered the efforts of the company to increase revenue from water-use less effective. Although releases have been issued, through which cooperation of the consumers has been requested to optimize utilization of the existing facilities it seems that other measures need to be developed to remedy the situation.

As the raising of revenue is considered as the most important thing to improve the service to the consumers, the company has given serious thought to problems related to the big amounts of unpaid bills and the collection of bills. Regarding the latter, the Jakarta Water Supply Company has involved private organizations in the collection of bills, on a contractual basis. Certain parts of the service area are allocated to these organizations, and incentives are paid at rates that depend on the amounts of revenue collected by the organizations. At present, this way of collecting bills is still considered as a trial method, and only a part of the area has been included. This method, naturally, reduces the number of personnel under direct employment of the company, which to some extent minimizes the problems associated with management of personnel. This method has so far been considered as the only solution because it is still uncommon for the majority of the people to have bank accounts. With less employees involved in the financial division, more room could be made available to recruit employees of the technical division, who are required for rehabilitation and maintenance to reduce the amount of unaccounted-for water, hence increasing the quantity of water that could be sold.

Another problem which could also be regarded as a social problem and to which a solution has still to be sought, is that of maintaining the quality of raw water in the river. Suspended matter and floating debris in the Banjir Canal water, which were not removed after screening, have frequently been the cause of failure of the desludging equipment of the Pejompongan plant. It seems very difficult to appeal for people's conscientiousness and their participation in not dumping any refuse into streams, although this is partly due to the inadequacy of facilities that could be provided by the municipality.

Considering the distance of flow of the raw water source through urbanized areas, considerable high loads of wastes must have been taken up. At present, a substantial reduction in strength of the pollution load could be attained by dilution. However, this situation should be watched carefully because the problem may become serious in the future if the existing regulations for the disposal of waste matter into public waters could not be enforced effectively, and considerable funds will then be required to restore the conditions of this potential source of raw water supply.

Staffing of the Company and Training of Personnel

Continuous assistance from foreign countries, primarily in technical fields, has been received since the development of the first Pejompongan purification plant in 1953, and it may take a few years before all activities associated with the water supply for Jakarta could be placed entirely in the hands of local staff. Of course, some of the expertise is assigned in accordance with the requirements set by the lending countries and foreign agencies, when providing their loans. It is, however, a fact that the current staff of the Jakarta Water Supply Company is inadequate to handle all the problems which emerge from the conditions of the system, and problems that have come into existence when new technology has been brought into the country. In this respect, the concept of a turn-key project, such as had been the case with the first Pejompongan purification plant, could be critized in the sense that the level of technology that is attached might not be appropriate to the available local expertise at that time. Only through intensive in-service training, have local engineers been able to accomplish proper operation of the plant equipment.

With regard to technical personnel to man the existing plants, the experience from the past has eliminated the difficulties of training of new personnel. The problem is whether the company is able to recruit engineers, considering the limited number of graduates from local technical universities. Difficulties have been encountered in finding a place where staff members of the company could be trained in other fields to enable them to solve other aspects of the problems. A most appropriate place would be a city where the conditions are similar to that of Jakarta.

The Problem of Technology Level

The development of water treatment technology has reached such a state that almost no difficulties will be encountered in treating raw water, of reasonable quality, to make it acceptable for drinking water. This technology has been transferred from the countries where it has been developed, to almost any developing country, Indonesia being one of them, through training programmes, assignment of foreign expertise to developing countries, scientific publications, foreign contractors, etc.

The Experience of Jakarta, and probably also in other developing countries, revealed that the problems regarding the application of advanced technology are mainly associated with (1) the availability of skilled personnel to operate the plant and to maintain proper conditions of the facilities, (2) the availability of equipment, or parts for replacements, which can be purchased easily, and (3) the initial cost of the plant.

The problem of skilled personnel has been pointed out in the preceeding paragraph, and no further discussion on this problem will be required here. With regard to the availability of replacement parts, careful consideration should be given to the selection of type of equipment or instruments, because most of the items are not fabricated in the country and have to be purchased elsewhere. Repair of malfunctioning instruments may not be possible locally and those instruments may have to be sent to other countries for repair, or factory technicians may have to do the repair work in situ. This of course necessitates the allocation of additional funds to cover transportation or travel expenses. Another matter to be considered is that dependability on foreign countries may necessitate the provision of additional investment on spare equipment to avoid loss of continuity of operation of the plant, which again increases the initial cost.

Initial cost is indeed the most important factor to be considered during the planning phase of a water supply project. Based on rough estimates, a so-called modern purification plant is more costly than the conventional-type plant (note that advanced technology is generally related to modern plants), partly because of the imported parts or equipment, and partly other expenses that are generally attached to such projects, such as travel expenses and fees for foreign construction supervisors. Considering that the higher initial cost, on a unit-of-water basis, the higher will be the user-charge, it should be considered necessary to keep the initial cost of a plant as low as possible. Modern plants have been claimed to be of greater efficiency than the conventional-type plants. This is true from the operation and process points of view, land area requirements, and the number of personnel that is needed to operate the plant. The question is whether it would be justified to attain the additional efficiency (relative to that of the less costly plants) at the additional cost. The price of land in some places may be high, but labour in most developing countries is less expensive than in the industrialized countries. Also, less qualified personnel are needed in the case of the conventional-type plant.

Emergency and Complementary Projects

It has been indicated in the preceeding chapter that a Master Plan for the Jakarta Water Supply System has been prepared. If all the projects that have been proposed in the Master Plan could be implemented, Jakarta could then be considered as a city with an adequate supply of water. According to this Master Plan, new water purification plants will be constructed: one plant of 4000 l/s capacity which, in addition to the currently available capacity, will satisfy the demand up to 1980, and one plant of 7400 l/s capacity to satisfy the requirements up to the year 2000. Both plants will be constructed in phases, as will be the construction of the distribution facilities.

During the last two years, certain parts of the city have reached a state of development which requires immediate supply of water. Some of these parts, although included in the projected service area of the proposed purification plants, will not be served earlier than 1990. Considering such a situation, the Water Supply Company has studied the possibilities of initiating emergency projects to supply the needs of the people in those areas, and other areas which may suffer from shortage of water during the years before the proposed plants could be put into operation. This idea of initiation of emergency projects was strengthened by the delay in the implementation of the projects proposed by the Master Plan.

The basic concept of the emergency projects is the development of "independent" water supply systems, consisting of a "moveable" water purification plant and a distribution system, in areas which will be served by the main water supply system of the city in the future. When the areas served by such a plant have received the required supply from the main system, the purification plant will be dismantled and the moveable parts, or units, of the plant will then be moved to other locations. The nature of the projects require careful considerations regarding the selection of areas and the duration of the project to render it economically feasible.

Design plans have been prepared for the installation of a purification plant with a capacity of 400 m^3/h. Additional units will be installed when the experiments with the first 400 m^3/h plant have shown satisfactory performance, and the method which has been applied to solve the temporary problem of water supply has been proven successful.

This type of project could of course be applied as a complementary supply facility to serve remote areas outside the planned service area of the main system. In this case also it is required to study whether this independent system is economically feasible, or whether it will require subsidy from the main system. Considering the existence of a discriminating pricing scheme, the amount of revenue generated from such a project depends, among other things, on the type of occupancy of the proposed areas.

Unfortunately, this concept may not be applicable to certain parts of the city due to unavailability of sources of raw water within economic distance. For such areas, a more feasible solution has to be sought.

4 Conclusions

The explosive growth of the population of Jakarta and the present condition of the distribution system are two major causes of the problems of water supply. New water purification plants have been constructed to increase the rate of production of potable water. However, the poor condition of the distribution system does not allow effective utilization and distribution of the added quantities.

Insufficient amounts of revenue that could be generated, partly due to the high percentage of losses and unaccounted-for water, have produced financial problems, constraining the efforts to improve the conditions of the system, while the limited funds allocated by the central government to the water supply sector and the enormous demand for water supply improvement throughout the country may necessitate giving a lower priority to Jakarta. Therefore, other sources of funds need to be sought to finance the implementation of development plans. Such an unfavourable financial situation makes it difficult for the Jakarta Water Supply Company to manage the water supply system based on economic principles. Although it has been given the authority to do so, there are still many factors that have to be considered, amongst which are the economic conditions of the majority of the people, and the social nature of water supply service.

No serious problems associated with the application of modern technology have been encountered, as far as the requirements for skilled personnel to operate plant facilities are concerned. However, justification from the financial point of view will still be needed, prior to the application of such technology, taking into consideration that, in this aspect and until the near future, Indonesia is not a manufacturing country.

Résumé

La détérioration de l'état des ouvrages de distribution d'eau due au manque d'entretien pendant un certain nombre d'années après le début de la Deuxième Guerre mondiale et la croissance explosive de la population de Djakarta peuvent probablement être considérées comme les causes majeures des problèmes que doit surmonter le service des eaux de Djakarta pour assurer à la ville une alimentation en eau adéquate. Bien que la production d'eau potable ait été accrue par la construction de stations de traitement, toute l'eau produite ne peut pas être effectivement utilisée étant donné l'état actuel du réseau de distribution.

Des nombreux problèmes liés au développement de l'alimentation en eau de Djakarta, les problèmes financiers ont donné le plus de souci étant donné l'importance des investissements exigés pour améliorer la situation et pour exploiter convenablement le réseau. Le présent rapport décrit les diverses sources de revenus, les problèmes qui se sont présentés et les facteurs pris en compte en établissant les prix de vente et autres charges liées à l'utilisation de l'eau.

Les deux problèmes principaux relatifs à la détermination des prix de vente de l'eau sont: les bases d'établissement du plan de tarification, et les conditions socio-économiques. Ces dernières, et d'autres déficiences dans le service et le réseau, sont considérées comme la cause des problèmes de gestion du réseau de distribution d'eau; elles affectent en outre le volume des recettes qui pourraient venir de la vente de l'eau. Sont inclus également dans la discussion les problèmes relatifs à la qualité de l'eau brute, qui peuvent devenir plus graves dans l'avenir si l'on ne prend pas de mesures positives.

En ce qui concerne le personnel du service des eaux de Djakarta, on s'est attaché à intéresser des experts étrangers au développement du service des eaux de Djakarta, en liaison notamment avec l'assistance financière apportée par les gouvernements étrangers et les banques de prêts internationales, et avec le niveau de technologie appliqué.

D'autres discussions ont été incluses sur les divers problèmes associés à l'emploi de ce que l'on appelle la technologie moderne de purification de l'eau. Des considérations spéciales ont été faites sur les aspects financiers de cette technologie.

Les importants investissements nécessaires pour développer le réseau de distribution d'eau, le problème d'accroître les revenus provenant de la vente de l'eau et des charges liées à l'utilisation de l'eau, et l'aide limitée qui pouvait être accordée par le gouvernement central ont retardé la réalisation des programmes de développement. La situation a amené à penser qu'il faut modifier les étapes du programme prévu et mettre en route des projets d'urgence qui demandent de plus faibles investissements.

Le rapport conclut que les problèmes auxquels est confronté le service des eaux de Djakarta sont surtout des problèmes financiers, les autres problèmes résultant de cette situation défavorable.

International Standing Committee on Water Quality and Treatment

Subject 1 (a)

Polycyclic Aromatic Hydrocarbons in Water

by Prof. Dr. J. Borneff

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As early as 1915 the carcinogenic potency of coal-tar was proved in animal experiments by Yamagiwa and Ichikawa. In the 1930's the search for the substances responsible succeeded in isolating polycyclic aromatic hydrocarbons, among which were compounds exhibiting varying degrees of carcinogenic potency. Applied to the skin or subcutaneously, only a few µg of the more dangerous substances resulted in fatal neoplasms in animal experiments. Oral application required higher doses and also solubilization by means of detergents, fats, or oils.

Man is also endangered by polycyclic aromatic hydrocarbons. This is true, for instance, for occupationally induced cancer of the skin. Of course, no data have yet been obtained with respect to minimum doses and, furthermore, there is no evidence of the effects from oral ingestion. However, it is the task of preventive medicine to limit the carcinogenic content of food to the utmost. This is relatively simple in regard to food additives; they can, for instance, be eliminated by restrictions, but foodstuffs themselves are more complicated. Therefore the yearly intake of polycyclic aromatic hydrocarbons by man amounts to 3–4 mg from fruits, vegetables, and bread etc., 0,1 mg from vegetable fats and oils, and about 0,05 mg from smoked meat or fish and drinking water.

Existence of PAH

Although water does not take the first place among the contributors of polycyclic aromatic hydrocarbons, its control is necessary since it is an essential part of the diet for which no replacement exists. Water, consumed or utilized by man, has to be obtained from different sources. Therefore, with the exception of the relatively uncontaminated groundwater sources, attention has to be devoted to surface water and waste water. Attention also has to be given to the by-products of sewage purification, the sewage sludges, in cases where they are to be used as fertilizers on cultivated soil. Wedgwood and Cooper pioneered the study of carcinogens in water. They qualitatively detected benzo(a)pyrene and other PAH in industrial waste water from a gas works. Our own research, begun in 1954, indicated that PAH from natural sources and domestic sewage proved as important a source as PAH of industrial origin.

At the beginning of these experiments it was assumed, that polycyclics in water were solubilized in the presence of fats, oils, detergents, or that they were in water by means of solid material (e.g. soot). The first part of our program was concerned with the study of particles. We checked, for instance, sandfilters of a lake-waterworks and discovered other PAH in addition to benzo(a)pyrene. The amount of carcinogenic substances within the mud was up to several mg per kg. In suspended particles from the Rhine river an almost equal quantity of carcinogenic substances could be traced. However, set in relation to the water volume, the strongly contaminated Rhine river carried about 200 times higher a concentration than the lake waterworks. Comparative tests showed, that the quantity of the sedimentary particles per m³ river water was not the only important factor. Therefore, one cannot judge the water quality just by its content of suspended and flocculated material. Further hints concerning the possible origin of carcinogenic substances came from the microscopic examination of waterworks sandfilter muds. They contained a lot of algae. Therefore we went fishing in the Lake of Constance for phytoplankton to isolate the source of benzo(a)pyrene, and we found the algae had only concentrations of 1/10th of the sandfilter muds. Thus we concluded the major portion of carcinogens is attached to other particles in the mud.

Since dust and soil other than algae were the major components in the sandfilter muds, we expanded the investigations of Blumer to study the occurrence of polycyclics in soil. Besides benzo(a)pyrene there were about 30 different polycyclic aromatic hydrocarbons in every soil sample. Together with analyses from other authors we can state the following:

(1) The absence of benzo(a)pyrene from soil is an exception,

(2) uncultivated soil contains about 1–10 µg/kg and fertilized soil about 100 µg benzo(a)pyrene, and

(3) considerably higher values were found in areas exposed to strong pollution by aerosols. As an example, 1 000 µg/kg benzo(a)pyrene were found close to a railroad track (not electrified) and 200 000 µg/kg in soil from an oil refinery.

However, not only benzo(a)pyrene is traceable in superficial soil stratums, but also PAH were found in material of borings from a depth of 170 m and about 100 000 years old (i.e. in ground surely not changed by civilization). This leads to the conclusion that industrial contamination is not the only source of pollution. The synthesis of these substances seems to occur mainly via pyrolysis, but production in plants and bacteria has also been demonstrated. In contrast, types of bacteria exist which can assimilate polycyclic aromatic hydrocarbons. Undoubtedly, city street surface runoff is one of the main sources of PAH in surface water. Road-tar-dust contains up to half a gram of carcinogenic substances per kg. Apparently a continuous charge of PAH comes from sewage input into a water-course. The PAH occur mainly in particles or at least are adsorbed onto particles. According to our tests, in raw sewage there are up to 100 000 µg or more carcinogenic substances in corpuscular form within 1 m³ water. Furthermore, we found some were dissolved. Initially, we believed PAH were solubilized by detergents, but soon we realized that the high concentration of detergents necessary to solubilize PAH does not occur in rivers and lakes. We ascertained, using improved analytical techniques, that polycyclics are dissolved even in unobjectionable ground water, which contains no detergents or mineral oils. Therefore, one can only speak in terms of low solubility.

Another source of PAH is rain water. Zoeteman *et al.* (1975) reported, that precipitations of six urban areas during 1973 and 1974 contained about 545 µg PAH/m³. Compared with the analyses (n = 26) of the River Rhine during 1973 (average value 270 µg/m³) rain water is relatively highly contaminated.

Precipitations seeping into the ground will release traces of PAH from the soil in addition. Now we can conclude, that available data on the occurrence of PAH in water are as follows:

- unobjectionable groundwater and unpolluted lake water contains: 10 to 50 µg/m³ (ng/litre)
- river water, slightly polluted contains: 50 to 250 µg/m³ (ng/litre)
- river or lake water, fairly polluted and rainwater contains: 250 to 1000 µg/m³ (ng/litre)
- waste-water, pollution may reach more than: 100 000 µg/m³ (ng/litre).

(These values comprise 3.4-benzopyrene, 3.4-benzofluoranthene, fluoranthene, 1.12-benzoperylene, 11.12-benzofluoranthene, and indeno-(1.2.3.-cd)-pyrene.)

We should mention that PAH in tap water is not necessarily equivalent to PAH in the original well. This result was observed by testing well waters and tap water from the end of the water supply pipe. At this locus the values were almost 10 times higher than in the well itself. Thus an increase of PAH occurred in the supply network. A contributing factor could be the paint within the water pipes. An analysis of the paint used for the pipe construction showed some mg/kg of carcinogenic material. Further tests showed that measurable amounts of PAH were observed, when water was passed over the pipes for several months. These results are confirmed by Fielding (1975).

The Fate of PAH

Our knowledge about the fate of polycyclic aromatic hydrocarbons in water can be summed up as follows:

In groundwater concentrations are normally stable at 10 to 50 µg/m³. This stability is due to the absence of significant numbers of micro-organisms capable of biodegrading these substances. Chemical decompositions have not been found. Theoretically it should be possible to remove almost all benzo(a)pyrene from drinking water by intensive treatment processes, but as long as foods contain amounts of carcinogens which are up to 100 times that of water, there seems to be little reward in it.

Surface waters exhibit natural purification processes. One of the most important of these is the sedimentation of suspended materials. Obviously sedimentation processes are influenced by the velocity of currents, therefore lake and reservoir waters usually contain lower amounts of polycyclic aromatic hydrocarbons than rivers. In addition to sedimentation, a fat soluble part of the polycyclics goes into the lipoid substances of plankton, and reaches the sediment as dead plankton material. Although polycyclics may be photo-oxidized, the influence of ultra-violet rays is of no practical consequence. This may be concluded from the fact that at the water surface (30 cm) of Lake Zürich there was a higher concentration of polycyclics, than at a depth of 30 m (36 : 26 µg/m³, average of 60 analyses).

Municipal and industrial sewage purification plants employ processes which are similar in principle to the natural ones just discussed. When currents are reduced, carcinogenic particles become subject to sedimentation. Biological treatment induces polycyclics to adsorb to micro-organisms which afterwards can be removed. In this way two thirds of the polycyclics will be eliminated. If the sewage is subject to chemical processes (e.g. chlorination) an extensive oxidation and destruction of carcinogenic characteristics will result. Waste-water treated accordingly will show, finally, similar concentrations of polycyclic aromatic hydrocarbons to normal river water.

Major amounts of the polycyclics exist in *sewage sludges*. Therefore, sludges have to be disposed of in a way that causes no further danger, or renewed pollution. For this reason sludges should not be burned, because their burning would probably add to air pollution. The best way of removal is to use it as fertilizer in agriculture. To our present knowledge there is little chance of accumulation of polycyclics in edible plants. A final opinion can only be given when pending control tests, under defined conditions, are evaluated. It is suggested that limited restrictions ought to be observed and it seems advisable not to fertilize potatoes and other edible root vegetables with sewage sludge.

Drinking water purification processes must be of the highest order. If surface water is utilized and treated by river bank filtration, effects on water quality are to be expected, but they will only be satisfactory if the river bank consists of very fine sand. The best results in river water purification were achieved by flocculation systems (iron salts + chlorine), supplemented by ozonization, active carbon filtration, and final chlorination. Correct procedures result in a residual content of polycyclics similar to that normally found in natural groundwater. Lake water usually requires less intensive purification than river water, but analytical checks should be employed to control the removal of polycyclics.

Method of PAH-analysis

1 Principle

The PAH are extracted from the water with organic solvent, separated, after a cleanup procedure by two dimensional thin layer chromatography, and identified and quantified by their fluorescence in the UV-light. Evaluation may be accomplished by;—visual comparison (semi quantitative) (section 5.1.); elution of the substances and measuring their fluorescence in solution (section 5.2.1.); measuring fluorescence directly from the TLC-plate with a scanner (section 5.2.2.).

2 Apparatus

Wide necked cans, 5–10 l capacity (glass, stainless steel or aluminium, containers may be used, but no plastic materials!); stirrer (3 000–5 000 r.p.m.) capable of stirring 10 l, with explosion proof motor; lubricants must not come in contact with the water; vacuum rotary evaporator, with 1 l evaporating flask; equipment for thin layer chromatography; separatory funnels, 1 l capacity; beakers, funnels, micropipettes; small conical flasks, about 10 ml capacity, glass stoppered; drying oven; UV-lamp with filter 365 mµ; if available: Fluorometer with scanner for direct measuring of TLC-plates.

3 Reagents

All solvents should be of high purity. Cyclohexane is additionally purified by distillation or by percolation through a column with active Al_2O_3. The residue of 600 ml of the cyclohexane should not contain any of the PAH when chromatographed on thin layer.

Cyclohexane

Benzene (e.g. Benzol f. d. Fluoreszenz-Spektroskopie, Fa. Merck, Art. Nr. 1785)

n-Hexane

Methanol

Ethanol (99%)

Diethylether

Sodium sulphate, anhydrous

Aluminium oxide powder (for TLC)

Acetylated cellulose (40% acetyl) (for TLC)

Aluminium oxide for column chromatography

Reference substances: (Manufacturer: Firma Ferak Berlin E. Gründemann oHG, 1 Berlin-West 47, Friedrichsbrunner Straße 3–5)

Fluoranthene

3.4-benzofluoranthene (benzo[b]fluoranthene)

11.12-benzofluoranthene (benzo[k]fluoranthene)

3.4-benzopyrene (benzo[a]pyrene)

1.12-benzoperylene (benzo[ghi]perylene)

Indeno(1.2.3-cd)pyrene

3.1 Stock solutions of the PAH:

A stock solution is prepared of each of the 6 substances by dissolving 10 mg in benzene and making up to 100 ml in a volumetric flask (= 100 µg/ml).

Care must be taken when weighing the substances, some of them are carcinogenic! Moreover, if contamination occurs in the laboratory this may lead, later on, to incorrect results. The solutions should be kept in dark, glass stoppered bottles, they are stable for several years, if care is taken that no solvent evaporates and thus the concentration is altered (this may be controlled by measuring absorption or fluorescence).

3.2 Standard solution:

5 ml of the stock solution of fluoranthene, and 1 ml each of the other 5 stock solutions are mixed in a small glass stoppered flask. This solution contains 50 ng of fluoranthene, and 10 ng each of 3.4-benzofluoranthene, 11.12-benzofluoranthene, 3.4-benzopyrene, 1.12-benzoperylene, and Indenopyrene in 1 µl.

4 Procedure

4.1 Extraction:

5 (–10) l of water sample are collected into the thoroughly cleaned can. In the laboratory, 300 (–600) ml of purified cyclohexane are added and stirred with a fast stirrer for 10 min. The phases are allowed to separate (preferably overnight), the cyclohexane layer is then carefully taken off (with a beaker) into a 1 l separating funnel to remove remaining water which has been carried over with the procedure. The cyclohexane solution is then filtered through sodium sulphate into a graduated cylinder. The amount of cyclohexane recovered should be 250 (–500) ml. The solution is concentrated to a few ml in a vacuum rotary evaporator, the concentrated extract transferred to a small conical glass stoppered flask, the evaporating flask is rinsed 2–3 times with small amounts of cyclohexane which are added to the extract. The content of the small flask is then further concentrated to about 0,5 ml. It is also possible to extract directly 2 × 2 500 ml water in a separating funnel by shaking. However by changing the method it must be proven that the recovery rate exceeds 80%.

4.2 Cleanup procedure

0,5 g of aluminium oxide, basic, activity II (according to Brockmann), (i.e. containing 3% water, if the purchased material is activity I, or 4% if it is activity 0) is placed in a small column (about 6 mm in diameter and 80 to 100 mm long, tapering off at the lower end) above a small plug of cotton wool. The column is rinsed with 1 ml of cyclohexane, the extract placed on the column, the conical flask rinsed with 0,5 ml cyclohexane, which are added to the column, too. The eluates up to now are discarded. Then a further 0,5 ml of cyclohexane and 3 ml of cyclohexane-benzene mixture (1 + 1) are used to elute the PAH. They are collected into the small conical flask and evaporated to about 0,1 ml for TLC.

If heavily polluted water is examined, only one aliquot of the extract is used for the cleanup and further determination.

When first introducing the method, the cleanup procedure should be tested with reference substances because different materials and other circumstances may affect the elution.

4.3 Thin layer chromatography (TLC)

4.3.1 Preparation of plates:

For the preparation of 5 TLC-plates 28 g of aluminium oxide powder, 12 g of acetylated cellose are mixed well with 65 g ethanol (using a stirrer or mixer) and the plates coated with this mixture using a coating apparatus. As soon as the plates are superficially dry, they are activated for 30 min at 130°C in a drying oven, then stored in a dessicator until required.

4.3.2 Chromatography procedure

The whole of the extract, if clean water is being examined, or an aliquot is applied in small portions as one spot in one corner of the TLC-plate about 1,5 cm from the two sides. The plate is developed in the first direction with a mixture of n-hexane + benzene (90 + 10 vol.) for 30 min. After drying the plate is turned by 90° and developed for 60 min. in the second direction with a mixture of methanol + ether + water (40 + 40 + 10 vol.). During the whole procedure the plates should be protected from light. For reference chromatograms, suitable amounts of the standard solution are applied to thin layer plates and developed as above.

5 Evaluation

5.1 Estimation by visual comparison

If no fluorometer is available, amounts of PAH may be determined semi-quantitatively by comparing the TLC-plate under the UV-lamp with reference chromatograms. In this case a series of TLC-plates, with 1–100 µl of the standard solution applied, are developed. Under the UV-lamp the spots on the sample plate are identified by their position and their fluorescence colour and the amount of each substance is estimated by comparison with the reference plates. For greater ease in handling, the developed plates may be sprayed with a preserving dispersion (e.g. " Neatan " by Merck) which must have no fluorescence of its own; the thin layer can then be drawn off the glass plate and fixed on a piece of black paper. Reference chromatograms preserved in this way may be used up to two months if kept in the dark, but must be replaced earlier if used frequently because of fading away of the fluorescence.

5.2 Quantitative determination by measuring the fluorescence

5.2.1 Elution of substances from the TLC-plate and measuring the fluorescence in solution

The spots of the substances in question are located under the UV-lamp, scraped off individually and eluted with benzene using small sintered glass filter funnels. The solution is filled up to a definite volume and the fluorescence measured at the following wavelengths:

Fluoranthene 465 nm

3.4-benzofluoranthene 454 nm

11.12-benzofluoranthene 434 nm

3.4-benzopyrene 431 nm

1.12-benzoperylene 420 nm

Indeno(1.2.3-cd)pyrene 505 nm.

The excitation wavelength is 365 nm.

For each substance a calibration curve is set up by preparing, from the stock solution, a series of standards with concentrations ranging from 0,005 to 0,5 µg/ml and measuring their fluorescence. To cover the whole range, two different instrument settings (sensitivity, slit width) will have to be used for high and low concentrations. A fluorescence standard must be used for calibration of the instrument.

Due to the background fluorescence values measured may be too high, unless the whole fluorescence spectrum is registered and appropriate correction made. On the other hand, losses occur during chromatography and elution which will be different in each laboratory and will have to be determined individually. For further details refer to the literature.

5.2.2 Measuring fluorescence directly from the TLC-plate with a scanner

Working principle of the scanner: the exciting radiation (in our case 365 mµ, filtered out from a Hg-lamp as light source) is directed onto the thin layer plate through a fixed slit, the resulting fluorescence reaches the fluorometer by a suitable optical device. The thin layer plate moves past the slit at a constant velocity, the intensity of fluorescence is registered by a recorder. The area of the registered peak corresponds to intensity of fluorescence and size of the spot, and therefore is proportional to the amount of substance in the spot.

Prepare two reference chromatograms with 2 and 5 µl of the test solution. Depending on the instrument used, mark the spots of the substances in a suitable way. Place the plate into the scanner and choose the slit length of the scanner according to the size of the spot to be measured. Adjust the plate so that the slit is just about in the middle of the spot. Now set the wavelength of the fluorometer according to the table below and open the slit of the fluorometer until fluorescence intensity is about 50%. Note the slit width. (Instrument sensitivity should be set at a high level and must not be changed thereafter). Now locate the plate so that the scanner slit is just in front of the spot, start the scanner motor and register the fluorescence intensity on the recorder.

Repeat the procedure for every spot on the reference plate.

Wavelength setting for the PAH:

Fluoranthene	462 nm
3.4-Benzofluoranthene	452
11.12-Benzofluoranthene	431
3.4-Benzopyrene	430
1.12-Benzoperylene	419
Indenopyrene	500

Then measure the spots on the sample plate, making sure that the instrument settings for each substance are the same as with the reference plate (particularly the slit width of the fluorometer!). Choose the reference chromatogram according to the fluorescence intensity of your sample. If the intensity is too high to be measured under the conditions of the reference plate (i.e. exceeds 100%) the slit width should be reduced and both sample and reference plate measured under the new conditions. If the difference is too large, a reference plate must be prepared with a higher concentration. It is not advisable though, to have too large amounts on the plate, and in cases where pollution of the water is to be expected, only an aliquot of the extract should be examined on thin layer, and if necessary a second plate with a suitable amount for measuring can then be prepared.

6 Calculation

The area of the recorded peak is proportional to the amount of substance present. It can be determined in different ways (as in gas chromatography), one way is to measure the peak height, and the peak width at half the height and multiply the two. This method gives quite good results. Of course, an electronic integrator can be used, if available. The amount of substance on a sample plate then is:

$$A\ (\mathrm{ng}) = \frac{B \times C}{D}$$

where B = amount of substance on the reference plate (ng),

C = peak area measured from the sample plate,

D = peak area measured from the reference plate.

If an aliquot of the extract has been used, multiply A by the appropriate factor to find the amount contained in the whole extract, which represents the amount in 10 l of water; divide by ten and you have the concentration in ng/l.

The concentrations of the 6 substances are added to give the concentration of PAH in the water. It must be borne in mind that this figure does not give " total " PAH concentration, but it is a good and easily comparable representation of the whole group.

7 Other Detection Methods

It must be mentioned, that other methods of detection may be used. With gas chromatography a large number of PAH can be determined. For separation of the compounds a column of high efficiency must be used and a careful cleanup prior to GC is necessary. Column chromatography followed by paper chromatography was widely used until TLC was introduced as a routine laboratory procedure, the latter method being less time consuming and more sensitive. The advantage of combined column and paper chromatography is that larger amounts of extract can be chromatographed and therefore substances of low concentration and weak fluorescence, compared with the main components, can still be detected.

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Résumé

Les PAH carcinogènes sont des substances nuisibles chroniquement actives dans l'eau potable. Ils sont partiellement d'origine biologique et partiellement produits par des processus pyrolytiques. Les PAH se trouvent dans les eaux d'égout à concentration très élevées et peuvent servir d'indicateurs de pollution fécale. Il est impératif que les PAH soient éliminés de l'eau potable ou réduits au faible niveau où on les trouve dans l'eau souterraine (30 $\mu g/m^3$). Un tiers des PAH dans les eaux de rivière se trouve sous forme particulée, un autre tiers est finement dispersé et le dernier tiers est dissous. Les stations de traitement modernes peuvent éliminer deux tiers des PAH par décantation, coagulation et filtration. Les PAH résiduels peuvent être enlevés par oxydation ou traitement au charbon actif. L'efficacité des divers procédés de purification a été testée lors d'essais en laboratoire et en station pilote. Le contrôle des analyses à plusieurs stations de traitement a vérifié l'efficacité pratique du traitement de purification. A notre avis, l'analyse de PAH est un complément valable aux analyses classiques de l'eau potable. Elle est aussi recommandée par l'OMS et en 1976, cette analyse a été rendue légalement obligatoire en Allemagne. Si on trouve plus de 250 $\mu g/m^3$, l'eau doit être rejetée comme douteuse. Une norme plus basse sera envisagée dès que l'on disposera de plus de données.

Toxicity of Nitrates in water and their removal

by Dr. P. S. Fuller

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Water nitrate has been shown to cause illness and sometimes death in infants, and it is suspected that it may be a cause of illness and death in the last decades of life.

The infant illness is methaemoglobinaemia and the illness in later life suspected to be induced from nitrate is cancer. In neither of these cases does nitrate itself act as the pathogenic chemical. It must be changed to and absorbed as nitrite in the first case and changed to nitrite before or after absorption before it can take part in the formation in the second case of one or other of the proven carcinogenic nitrosamines.

Although it is accepted that severe methaemoglobinaemia in infants has, in some cases, been caused by certain nitrate waters, all infants on similarly high nitrate waters do not necessarily become ill. Likewise only a few adults have developed a cancer that might be connected with their intake of high nitrate waters. Why, when on waters of similar nitrate content, some humans become ill while others remain free of overt symptoms has stimulated much interest and important work into the aetiology of these diseases and into the co-factors concerned with their production.

On the other hand much work in this subject has also been directed to producing evidence which will provide a scientific basis for determining safe limits for nitrate in water supplied to the whole population.

Physiology and Pathology of Methaemoglobinaemia

Life is dependent on sufficient oxygen being carried from the lungs by the blood to all organs of the body. Most of the oxygen is carried within the red blood cells by the pigment haemoglobin and the oxygen is loosely and reversibly attached to the iron atoms of the haemoglobin molecule.

If the iron atoms in haemoglobin become oxidised the pigment changes to methaemoglobin. Its colour is changed and it loses its ability to combine reversibly with oxygen and thus becomes useless for oxygen carriage. While the nature of the iron-oxygen bond is incompletely understood (Weiss 1964) it is usually considered that the iron in haemoglobin is in the ferrous state and becomes oxidised to the ferric state in methaemoglobin. Normally only about 1% of the red cell pigment is in the form of methaemoglobin and it is constantly being formed from and reduced back to haemoglobin. The reduction is maintained by a complex system of enzymes (Jaffe and Newmann 1968).

This equilibrium ensuring the presence of only some 1% of methaemoglobin can be upset in three ways—by an abnormality of haemoglobin; by an abnormality of enzymes, or by the overwhelming of the enzyme system by some chemical or drug.

In the first case, advances in knowledge have been made during the last few years in regard to haemoglobin. Its exact atomic structure was worked out by Perutz *et al* in 1968 and it has been possible to define a variety of abnormalities in the molecule. Some of these can give hereditary forms of methaemoglobinaemia and are known as haemoglobins (M). At least five have been described (Nagel and Bookchin 1974) and with some abnormalities the patient is more susceptible to the induction o cyanosis by drugs. Also, foetal haemoglobin is more easily oxidised than adult haemoglobin and the infant is born with a predominance of haemoglobin (F).

Secondly, Jaffe (1969) states that where congenital methaemoglobinaemia is not due to abnormal haemoglobins it is usually due to a decreased enzyme activity, particularly of the NADH methaemoglobin reductase system.

When aberrations are homozygous, congenital methaemoglobinaemia appears but when they are heterozygous there is normally no methaemoglobinaemia but individuals may be more susceptible to drugs and chemicals that may cause the condition.

In addition, Ross and Desforges (1959) found that the erythrocytes of cord blood reduced significantly less methaemoglobin than did those of adult controls and interpreted this as evidence of a transient deficiency of DPNH dependent methaemoglobin reductase.

Thirdly, it has been known for many years that methaemoglobinaemia can be brought about by the action of various chemicals and drugs. The condition came into prominence during the 1914/1918 war among workers in explosive factories. Many chemicals used in industry and many drugs used in medicine can produce this effect. Nitrite formed from nitrate in water or food is in this group.

Some individuals are more susceptible to the effect of nitrate and other chemicals. Indeed, Cohen *et al* (1968) found soldiers in Vietnam, who on standard doses of primaquine that did not affect the rest of the army, developed a methaemoglobinaemia ranging from 20–32%.

Methaemoglobinaemia not only causes a change in the colour of the skin to brownish-blue or greyish-blue (Bosch *et al* 1950) but also causes poor oxygenisation of the tissues. In severe cases this may cause death but the symptoms, when caused by chemicals, are readily treatable by the removal of the cause and by methylene blue or ascorbic acid. Therefore it is important that the Health Authority be alerted whenever there is a likelihood of methaemoglobinaemia occurring through the incidence of high nitrate water. Also, prompt notification may save misdiagnosing the cyanosis and extensive investigation for heart disease.

Defects other than plain lack of oxygen have been found in association with high nitrate water; thus, in hereditary methaemoglobinaemia due to DPNH methaemoglobin reductase deficiency, an associated mental retardation has been described. Jaffe and Hsieh (1971) do not consider it to be a cause and effect association but that some congenital defect is responsible for both conditions. Petukhov and Ivanov (1970) compared school children on water containing 105 mg/l NO_3 with elevated blood methaemoglobin with control children on drinking water of 8 mg/l NO_3 and normal blood methaemoglobin and studied psychophysiological reactions to light and sound irritation. The children whose blood obtained 5,30 per cent methaemoglobin presented more pronounced signs of fatigue than those of the control group and these investigators consider that their data suggests that nitrates probably affect the central nervous system.

Shuval and Gruener (1972) suggest caution in extrapolation to humans of experimental studies with sodium nitrite in rats. They showed that nitrites can pass through the placenta of rats and cause raised levels of methaemoglobin in the foetus. They also found changes in the blood vessels of rats chronically exposed to high levels of nitrite in their drinking water.

Surveys and Field Studies

The main body of knowledge about the way water nitrate affects infants, and also the basis for recommendations on standards, rests on the important surveys and clinical observations made during the years since Comly (1945) first described the condition. These cannot be condensed within this short paper and have been recently reviewed by the International Standing Committee on Water Quality and Treatment of the International Water Supply Association (Aqua **1**, 1974).

Nitrosamines

The possible hazard to health of adults from high ingestion of nitrate is that, reduced to nitrite, it may play a part in the body in the formation of nitrosamines. These are known to be highly carcinogenic for animal species and there is no reason to suppose that man is resistant to these compounds, but proof of this has to rest on epidemiological studies (Mirvish 1975).

Nitrosamines can be formed in the urinary bladder if the nitrate or nitrite concentration in the urine is high enough and if bacteria are present. This has been demonstrated in vivo in rats, as has absorption from the bladder of this substance into the blood-stream (Hawksworth and Hill 1974).

The incidences of cancer show wide geographic variation and it is likely that chemical carcinogens are responsible for such human cancer, thus the epidemiological approach is being vigorously pursued.

A study in England to see whether there was any change in the incidence of malignant disease in an area where there was a high nitrate concentration in drinking water compared Worksop, which until recently had contained high concentrations of nitrate in the supply (90 mg/l NO_3), with nine low nitrate control towns. This revealed that Worksop had an increased death rate from gastric cancer and the authors concluded that more detailed epidemiological studies of the relationship between nitrate consumption and the incidence of gastric cancer would be valuable (Hill *et al.* 1973).

Chile has a high mortality from stomach cancer and death rates show a peculiar geographic pattern of high and low risk areas. Data on the use of nitrates throughout the country demonstrated a high correlation between death rates from this cause and the cumulative per-capita exposure to nitrogen fertilisers. This was reinforced by a lack of association with socio-economic conditions (Armijo and Coulson).

Epidemiological studies in Colombia showed that the mountainous areas have an abnormally high incidence of gastric cancer while that for people from the coastal areas is low. It has also been shown that in the three areas considered to be of the highest risk for gastric cancer there are water supplies with up to 145 mg/l NO_3. In further studies high urinary nitrate has been detected in one of these, the high nitrate area of Guitarilla (Hawksworth *et al.* 1974). These studies in different geographical areas have found an association between high water nitrate and a form of cancer. This does not amount to cause and effect, indeed it could be coincidental. However it does not seem sensible to allow an increase in drinking water of any substance until it is proven to be harmless.

London

In England, sources of drinking water with high nitrate content have been uncommon and usually associated with small supplies from underground. Therefore it has been practicable to supply, in these areas, bottled water of low nitrate content for bottle-fed infants. However, if a main river source was concerned, the substitution of bottled water would be difficult and, in London, the Metropolitan Water Board concluded that to make such arrangements for its large population was not practicable and that all distributed water should not contain more than the 50 mg/l NO_3 recommended by the World Health Organisation. Since nitrate is not significantly removed by the conventional treatment used, the limit of nitrate concentration must be set at 50 mg/l in the raw water to ensure that treated supplies conform to the standard.

The River Lee is the source of some 12% (2,5 m^3/s) of London's supply. In 1970 it was observed (MWB 44th Report) that while the nitrate content varied, the last three 10-year averages had shown a steady upward trend which would eventually overtake the standard. But this trend started to increase sharply in the winter of 1971 and from the autumn of 1972 remained almost continually around or above 50 mg/l until May 1973. This resulted in a considerable loss of abstraction from the Lee.

The concentration of nitrate in the River Thames did not go up to this extent and supply was maintained by diluting and blending Lee water with water of lower nitrate concentration, either from the Thames or from storage reservoirs. This situation, coupled with plans for encouraging the expansion of the population of towns draining to the Lee, led to detailed studies of nitrate contributions and proposals for possible remedial measures being made by the then separate water, river and drainage authorities with the Water Pollution Research Laboratory and the Department of the Environment.

Nitrate contributions to the Lee for the period 1966–1971 were examined in detail so that the effect of future changes could be assessed. Two main sources of nitrate were sewage works effluents and land runoff; under dry weather conditions, sewage effluents were the main nitrate source whereas land runoff dominated during wet winter conditions.

For an average flow year the relative contributions of nitrate to the river by sewage effluents and land runoff were 30% and 70% respectively at 1973 populations.

The largest effluent in the catchment is from Rye Meads Works (0,8 m^3/s, 1973), approximately 15 km upstream of the water supply intake at Chingford. It was calculated that, by decreasing the effluent nitrate concentration to 65 mg/l NO_3 the nitrate concentrations at the intake would not exceed 50 mg/l NO_3 under dry weather flow conditions.

Denitrification

Concurrent with the investigations into nitrate sources potential methods of nitrate removal were being surveyed.

The Water Pollution Research Laboratory had conducted literature surveys of denitrification methods together with pilot scale experimentation (Bailey and Thomas 1975) and recommended that at the Rye Meads Sewage Works denitrification could be achieved by modifications to the plant.

Activated sludge

Basically the method is the removal of air diffusers from the first pass of a four pass plug-flow diffused air activated sludge unit with mechanical stirring in this first pass to keep in suspension the activated sludge and form an "anoxic" zone. In this first pass the oxidation of carbonaceous matter in the settled sewage occurs by utilisation of nitrate oxygen from activated sludge returned from the final sedimentation tanks. In the remaining three passes, with normal air diffuser systems, the remaining carbonaceous matter and ammonia is oxidised. In theory, 50% of the nitrate could be removed by this method since half the oxidised nitrogen is recirculated. From commissioning in January 1974 the process has run successfully and the theoretical 50% denitrification has been achieved. The work to April 1974 has been summarised (Cooper *et al*).

Since its inception in April 1974 the Thames Water Authority, responsible for the whole water management cycle, has vigorously promoted the denitrification studies. Alternative methods of achieving anoxic zones in the activated sludge plant have been investigated and by mid-1975 seven modified units were in operation at the Rye Meads Works, treating 75% of the works flow. By summer 1975 an overall decrease of 25% in the nitrate concentration had been achieved which, had it occurred in the low flow year of 1973, would have maintained average Chingford intake nitrate nitrogen concentrations below 50 mg/l NO_3 throughout the year.

Other denitrification studies have been on water after abstraction from the Lee.

Static Bed Flooded Biological Filter

A filter based on the experiments of St. Amant and McCarty (1969) was set up in November 1974 to determine the effectiveness of the method on Lee water and in different climatic conditions. Water is abstracted from an aqueduct and delivered via 150 mm diameter pipework to the lower inlet of a tank 4,5 m. high and 2,6 m. in diameter. It is passed through a system of small diameter distribution orifices to flow upwards through a 25–40 mm ballast filter media 3,2 m. in depth and then through a 150 mm diameter pipe connected to the top of the tank. Methanol is pumped into the influent pipework just prior to entry to the tank.

Nitrate removal rates of greater than 80% have been constantly achieved for rates of flow in the order of 10,42 l/s (theoretical retention time 10 minutes) on influent nitrate concentrations of 33,2–88,6 mg/l NO_3. The hydraulic rate represents an upward velocity in the filter of 2 mm/s. The water temperatures have fluctuated between 10–23°C without significantly affecting filter efficiency.

The influent dissolved oxygen concentrations have been within the range 6–16 mg/l.

The major advantages of this method have been that the bacteria remain fixed to the media and hence no recycling of the sludge is required but, periodically, flushing is needed to remove some of the biomass. Disadvantages have included flow channelling within the filter and the production of sulphides under these anaerobic conditions in high water temperatures.

Fluidised Bed Filter

A pilot scale plant of similar capacity to the static bed filter has been constructed with a distribution system based on an upward velocity of 7 mm/s and will become operational during 1975.

A small scale filter of this type has been in operation for some months. This filter is 2 m high and 1 m in diameter. The water is passed through a distribution unit at the base of the tank upwards through graded ballast and finally through an 0,75 m/depth of sand (particle size 0,5 mm diameter) on upward velocity of 5,56 mm/s. The expansion of the sand bed has been 100%.

The filter efficiency rates have been satisfactory with greater than 80% removal rates being achieved on influent nitrate concentrations of 45-65 mg/l. The retention time is approximately six minutes.

It now seems likely that with the present load of nitrate in the Lee the recommendations of World Health Organisation European Standards can continue to be met.

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Sommaire

Une forte teneur en nitrate des eaux potables est une des causes reconnues de la méthémoglobinémie chez les nourrissons, et est soupçonnée d'être une cause du cancer à un âge plus avancé. Pour être cause des deux maladies, il faut que le nitrate se transforme en nitrite avant ou après l'absorption. La méthémoglobinémie peut se manifester chez les sujets où il existe une anomalie congénitale de la molécule d'hémoglobine ou des systèmes enzymatiques par lesquels la méthémoglobine est réduite en hémoglobine. Elle peut se manifester aussi quand les systèmes enzymatiques sont perturbés par certaines substances chimiques parmi lesquelles se trouve le nitrite. Certains individus ont une sensibilité plus prononcée à de telles substances que l'ensemble de la population.

La méthémoglobine n'a pas la capacité de l'hémoglobine d'apporter l'oxygène aux tissus et d'autres défauts ont été décrits associés à la méthémoglobinémie.

Le danger pour les adultes reste la possibilité que le nitrate joue un rôle dans la formation dans le corps de substances cancérigènes, les nitrosamines, ce qui a été démontré chez les animaux. Des études épidémiologiques en Angleterre, au Chili et en Colombie ont montré une association statistique entre un haut niveau de nitrate et une forme de cancer. Ceci est une indication que le sujet mérite des recherches plus approfondies.

Les eaux des captages de faible importance qui ont une haute teneur en nitrate peuvent être remplacées par l'eau en bouteille pour les nourrissons, ce qui devient moins pratique dans les grands services d'eau. Le niveau du nitrate dans le Lee—rivière fournissant 12% du total de l'eau potable à Londres—a montré une tendance à augmenter, et en 1971, 1972 et 1973 a parfois dépassé 50 mg/l. Par conséquent l'utilisation de cette eau a dû être restreinte bien que la distribution fût maintenue grâce à une dilution avec de l'eau d'une concentration plus basse.

Après une étude des origines du nitrate, la Thames Water Authority a examiné et vigoureusement appliqué des méthodes de dénitrification. Les résultats les plus encourageants, qui ont obtenu une dénitrification de 50% à une station de purification des eaux d'égout, ont été obtenus par la supression des diffuseurs d'air de la première phase d'une installation à boues activées avec maintien d'une agitation mécanique dans cette première phase pour garder en suspension la boue activée en vue de former une zone anoxique.

En utilisant un lit filtrant biologique immergé, on a obtenu en permanence un enlèvement de 80% du nitrate avec une vitesse ascendante de 2 mm/s. En ajoutant de l'alcool méthylique à l'eau de rivière traitée, un lit filtrant fluidifié d'une vitesse ascendante de 7 mm/s sera mis en service en 1975.

Flow-through system for the continuous monitoring of raw water using trout

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Introduction

Systematic water quality control of raw water for drinking water is of the utmost importance for the protection of human health. The increasing pollution of these waters, which occurs at many places all over the world, has led to the development of very advanced methods for pollution detection. Most pollution monitoring programmes have been oriented solely towards chemical and physical parameters. It is difficult, however, to predict the toxicological quality of a complex, continuously changing, water from chemical and physical analysis alone. In particular, unexpected contaminants may go undetected because the number of parameters which can be measured continuously by chemical and physical methods is very limited. Furthermore mixtures of toxic compounds may have synergistic or even antagonistic effects which cannot be measured by instruments. In addition there are many practical problems using chemical/physical methods which are for the most part due to the limited sensitivity of the sensor and the need for frequent and intensive cleaning. Biological monitors may overcome most of the disadvantages of chemical/physical monitors. However it should be pointed out that biological monitors cannot be used alone without chemical/physical monitors since biological monitoring systems can only indicate that a toxic condition exists but in most cases not the identity of that toxic condition.

The integrated use of both types of monitors can lead to the effective control of surface water and contribute to a safe drinking water supply.

This paper describes the construction and practical application of a biological monitoring system which is suitable for monitoring surface water.

Requirements for continuous monitoring systems

A biological monitoring system for surface water should use water organisms continuously present in and exposed to that water. If the water contains a pollutant to which the water organism is sensitive, the organism will react to this situation in a particular way. A monitoring system that uses one or other reaction of a living organism to a toxic substance must meet a number of requirements:

1. The monitoring system should be sensitive to the greatest possible number of toxic substances.
2. The test organisms, their reactions to a toxic substance and the measuring method must be suited to continuous and automatic measuring whereby, in case of emergency, an alarm will be put into operation.
3. The biological variation which is inherent in the use of living organisms should be excluded as far as possible in order to avoid false alarms.
4. During normal situations the test organisms must be able to remain in the monitoring system for a long time without this influencing their behaviour, metabolism and sensitivity to toxins.
5. The monitoring method must be such that slight interference from the outside cannot influence the measured result.
6. The surface water to be controlled must flow continuously through the pilot plant.
7. The test organisms must be easily obtained, bred and kept alive and standardisation of the organisms must be possible.
8. The monitoring method should be as simple as possible.

The monitoring system

The continuous monitoring system is shown schematically in Fig. 1. This system is a combination of methods applied by Zahner (1), Junke and Besch (2) and by Vivier (3).

The advantages of each of these methods have been used and are supplemented by personal experience. As rainbow trout (Salmo gairdneri R.) meet all the requirements mentioned in the previous paragraph they have been used as test organisms. Moreover, trout in flowing water possess a strong positive rheotaxis which makes the trout very suitable for monitoring in a flow-through system. Where surface water with low oxygen content is used, the water should be aerated shortly before flowing into the flow-through system.

The plexiglass flow-through basin in which the trout are kept has a length of 2 m, a width of 620 mm and a height of 250 mm; the water depth is 150 mm.

The turbulence of the incoming water flow is damped by means of specially constructed baffles, so that the water in the swimming area of the fish has almost the same velocity over the whole cross-section. The central part of the test basin consists of three separate canals (160 mm wide) each containing one trout and one monitoring system.

The flow velocity of the water is variable and can be adapted to the fish length. The rule of thumb for trout is that if the fish has a length of X mm the flow velocity may not be much more than $\frac{1}{2}$X mm/sec, in order to prevent tiredness from a prolonged stay in flowing water. In order to prevent stress, the trout should be able to turn easily. In this case the trout should have a length of 150 to 160 mm. The maximum flow velocity may then be 80 mm/sec which corresponds to a water flow of about 21 m^3/h.

When a toxic substance in the water reaches the trout in the flow-through system they may react in two different ways:

(a) the fish will detect the toxic substance and try to swim away from it. This is possible only by turning and trying to avoid the poison by following the stream;

(b) the fish will not perceive any toxic substances and will show no evading reaction. Sooner or later their condition will be affected and they will consequently no longer be able to swim against the stream.

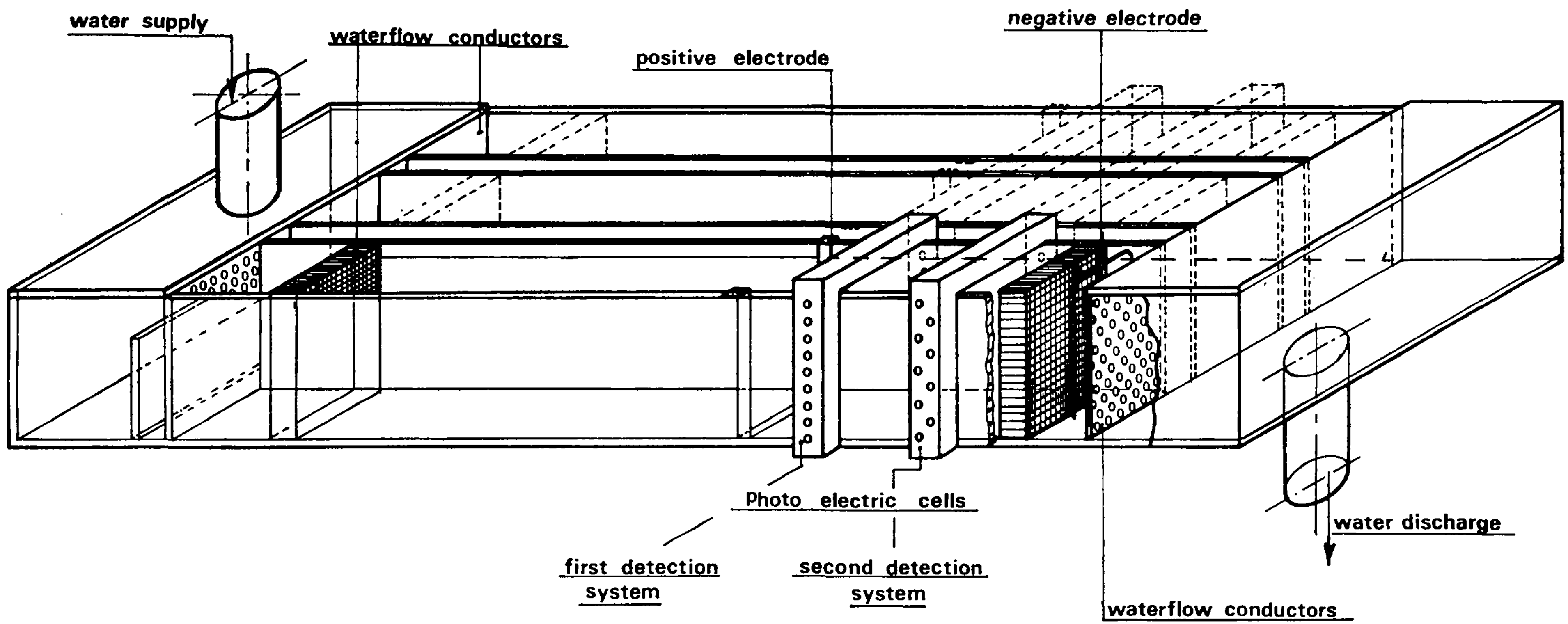

Figure 1—Flow-through system for continuous monitoring of surface water with trout. For practical application, two units, using 6 trout, are recommended.

The very sensitive evading reaction and their inferior condition will lead to a downstream movement of the fish, which can be used for the indication of toxic substances. This principle is applied in the method of Vivier (3) by means of photo-electric cells. The first requirement here is that an alarm signal will be given only when a toxic substance is present. For this purpose it is necessary that the number of passages of the fish across the beams of the photo-electric cells from causes other than the influence of toxic substances should be reduced to the lowest possible level. This is achieved by means of electrical impulses given as soon as a fish interrupts the beam of the photo-electric cells. The low tension electrodes (see Fig. 1) have been fixed in the basin at both sides of the photo-electric cells, the positive electrode being situated upstream of the photo-electric cells. This is very important, since fish in an electric field swim towards the positive pole. After experiencing the electrical irritation the fish will try to stay in the part of the basin situated upstream of the photo-electric cells.

The electrical irritation stops 10 seconds after a passage past the photo-electric cells. These 10 seconds are normally enough to send a fish upstream again. The electrical irritation must stop after a short period since the positive electrode is placed upstream from the photo-electric cells and in the case of a continuous stream of electric pulses the fish are irritated before they pass the photo-electric cells. It is however possible for a fish to sustain the 10 seconds irritation and remain in the downstream part of the test basin. This excludes such a fish from further monitoring. To prevent this situation, independently of a passage past the photo-electric cells, every two minutes a train of electric impulses is given off for 10 seconds. If a fish experiences this unexpected electrical irritation he immediately swims back upstream. Based upon model experiments with specific toxic substances it appeared that in a few cases a fish, after being seriously affected by a poison, will pass the photo-electric cells only once or twice, remain in the downstream part despite the electrical irritation and die.

This behaviour makes a second detection system necessary which indicates that a fish is present in the extreme downstream part of the test basin for longer than a certain time period. This second detection system also uses photo-electric cells.

Indication of alarm

When a toxic substance is present in the water and either affects the condition of the fish or makes them take evasive action, which is registered by the first detection system, the fish will consequently pass the photo-electric cells which in turn will repel them by electrical irritation. Subsequently, the fish will pass the photo-electric cells again, will again be repelled—and so on. This will increase the number of passages past the photo-electric cells considerably and operate an alarm. The behaviour of each of the three trout is registered separately on a printer. In order to prevent an alarm from causes other than a toxic substance, the electronic switch has been designed in such a way that the alarm will operate only when at least 2 fish in the same measuring period of 15 minutes pass the photo-electric cells more often than normal.

Parallel to the first detection system, the second one will operate an alarm only when at least 2 fish remain in the extreme downstream part of the monitoring system for longer than five minutes. The period of five minutes has been chosen because in this period, independent of a passage past the photo-electric cells, a fish in the downstream part of the test basin will be electrically irritated twice. If a fish remains in the down-stream part despite two electrical irritation periods, this is abnormal behaviour.

Since it is not excluded that a fish may pass the photo-electric cells more often than normal and die before another fish reacts to the toxin with an increased number of passages past the photo-electric cells during one of the subsequent measuring periods, it is necessary that the combination of the first and second detection system can operate the alarm. This is illustrated in Fig. 2.

Practical experiences with Rhine water

The monitoring system has been in operation on Rhine water for two years already, and has come up to expectation.

The mean number of passages past the photo-electric cells of one trout in 24 hours has appeared to be about 5, which is very low. The alarm level, which could be decreased down to 4 passages per 15 minutes by at least two trout has not been reached once.

The sometimes very turbid Rhine water did not interfere with measurements. In addition the rainbow trout behaved normally in the Rhine water in the temperature range between +4°C and +26°C (extreme water temperatures during the test period). The trout are fed twice daily with Trouvit but the trout also feed themselves with natural food. During their stay in the monitoring system the trout increase in length and weight. In a normal routine procedure, every month new trout are placed in the monitoring system in order to prevent adaptation to the test system and the Rhine water. During this one-month period and even up to a three months stay in the system, the trout show no signs of stress or other changes in their behaviour.

Internal and external macroscopic investigation and extended histopathological research showed no deviations or abnormalities.

Experiments with toxic substances

The fact that during the test period of the flow-through system on Rhine water no alarm situation occurred makes it necessary to study the reactions of the trout to specific substances. In a closed, all glass (except the stainless steel pump) flow-through system with circulating water a broad range of toxic substances are tested. Some of the results obtained are shown in table 1.

TABLE 1. Time lapse between first alarm and death of test fish caused by toxicants in a closed circulating flow-through system.

Toxicant	Dosage mg/l	Time until 1st alarm (h)	Time until death (h)
$CuSO_4$	1,5	2,5	13
$HgCl_2$	1,0	29	30
Na_2SeO_3	100	4	16
Lindane	0,06	0,75	6
Lindane	0,15	0,40	2

Water temperature: 12°C
Test water: standard water (4)

The first results show that for the chemicals tested, except for $HgCl_2$, the time from dosage until the first alarm is considerably shorter than the time until the fish die. More chemicals, however, must be tested before the sensitivity of the flow-through system is known.

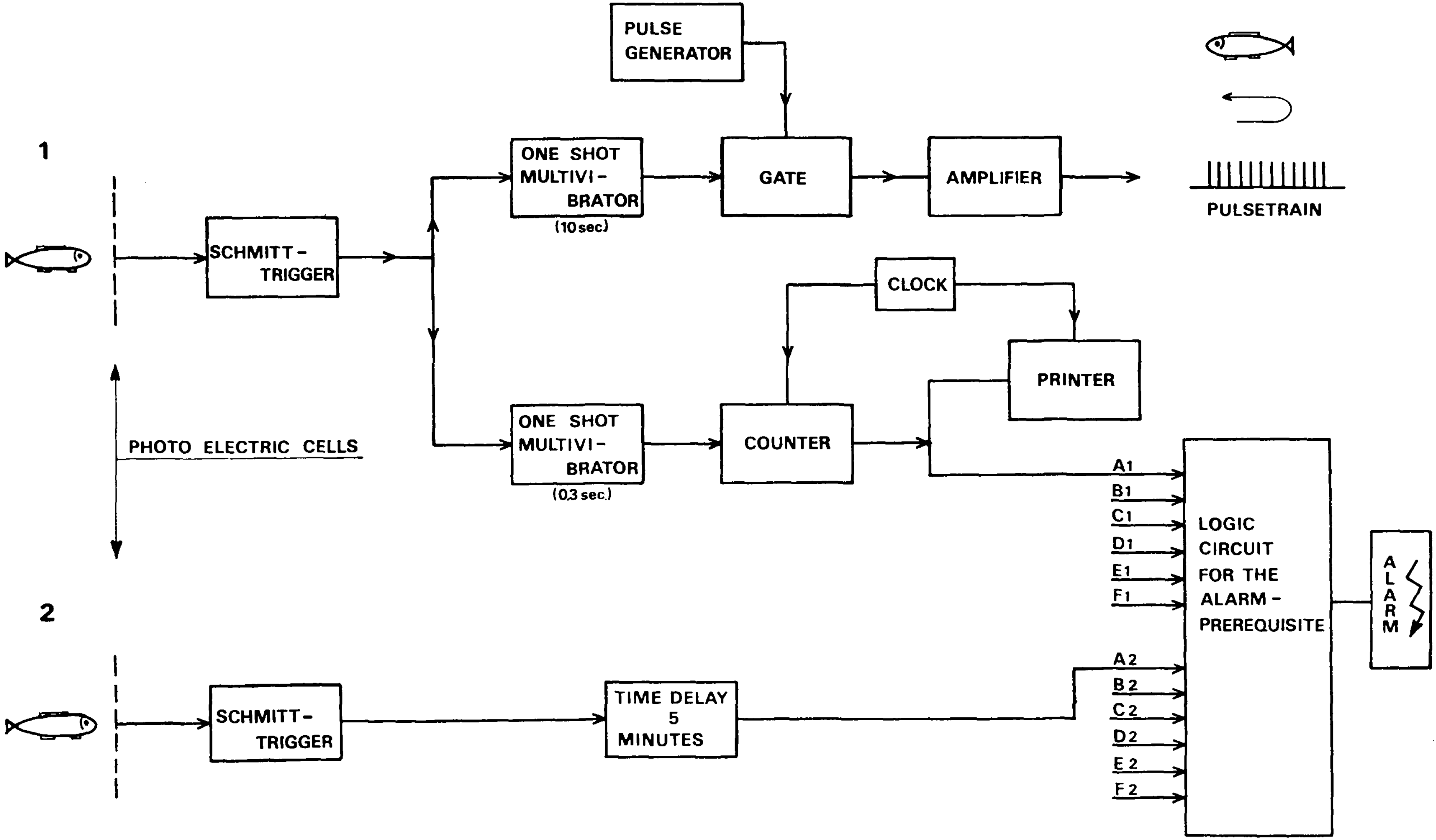

Figure 2—Electronic system for surveillance of behaviour of test fish. The upper part of the scheme represents the first detection system (1) monitoring avoidance reactions and decrease in condition of the test fish. The lower part represents the second detection system (2) registering death or complete inability to swim.
Both systems, either apart or in combination, operate the logic circuit which contains the alarm prerequisites. If an alarm prerequisite is exceeded the alarm signal is switched on. For further explanation see text.

Practical application

In the case of an alarm it should, in the first instance, be visually determined whether the trout show deviant behaviour as a result of an unknown cause (for instance an acute Myxobacteria infection on the gills). If most or all of the trout show deviant behaviour it will probably be caused by the water quality. In order to exclude several causes which directly influence the metabolism of fishes and which have no direct toxicity for men it is necessary to measure either continuously or immediately after an alarm, the pH, temperature, oxygen content and ammonia content. If these values are normal it is likely that a toxin caused the alarm. In the case of an alarm, samples should be taken for chemical investigation to establish the identity of the toxic pollutants and the proper authorities should be warned.

Continuous-flow basins can be used at several places to monitor water quality. Waterworks processing surface water could take advantage of this monitoring system by placing basins some kilometres upstream (depending on the flow velocity of the water) from their intake. Another possible application would be to site these flow-through basins at strategic points on larger rivers. The fish monitoring system could also be applied at inlets to storage basins as well as upstream of the intake points to the waterworks from storage basins. One very good application of a flow-through basin could be at outlets from factories and sewage purification plants so that discharges of toxic waste water could be indicated quickly and stopped.

Discussion

The flow-through system described in this study has a number of advantages, the most important ones being:

1. the absence of stress and abnormal behaviour of the test fish
2. three different reactions of the fish to a toxic substance can be registered, either singly or in combination
3. false-alarms are almost completely prevented
4. the electronic equipment is simple because it registers clear reactions of the test fish.
5. disturbances from the surroundings hardly interfere with the measuring procedure.

From the long term testing of this flow-through system on Rhine water it is obvious that this system is suitable for practical application to surface water. The fact that model experiments with specific toxic substances, using loss of positive rheotaxis as an indicator of abnormal behaviour, can give an early response makes it acceptable that such systems are suitable as early warning systems for the presence of acute high concentrations of toxic substances. However it is necessary that more and different types of toxic substances are tested for their effects upon fish before flow-through systems based upon loss of positive rheotaxis can be applied in practice with a high degree of security.

The biological monitoring system described in this study, although perhaps less sensitive than other biological systems, is sensitive enough for the applications mentioned above. It is shown in a literature study carried out by the Environmental Protection Agency in the USA that for 90% of 40 randomly selected substances, the critical concentration selected as a result of aquatic toxicity is more restrictive than that for acute oral ingestion by mammals even after application of a 100 fold safety factor. For the remaining 10% of the chemicals a 10 fold safety factor is found (5). These results are in accordance with the results of Jung who found, based upon a literature survey, that for 97% of almost 1000 chemicals, water organisms are more sensitive than mammals (6). From these results we may conclude that in most cases an early warning of the presence of an acute high concentration of toxic substances in the water is indicated by the flow-through system described above.

At this moment biological monitoring systems based upon other reactions of fish are under development. Some of these systems (7, 8) are very promising under excellent laboratory conditions, but await their application under practical circumstances using surface waters with a high degree of turbidity. Although these systems may be more sensitive, changes in water quality and growth of small particles on electrodes can interfere strongly with the process.

The automatic biological warning system described in this study has been developed primarily for water undertakings using river water as the raw material for the preparation of drinking water. If an acute high concentration of a toxic substance is present in the river water the water-intake must be closed down in time to prevent dangerous, high concentrations of toxic substances from reaching the purification plant or even the drinking water consumer. Whatever biological warning system is used, there is always a delay between the presence of a toxin and the reaction of the fish. This time delay must be long enough to register any abnormal behaviour of the test fish and to close down the water intake. For this reason it is advisable that the water undertaking has a certain raw water buffer capacity at its disposal between the warning system and the purification plant and a larger storage basin to continue the production of drinking water when the raw water intake is closed.

Summary

An automatic biological warning system has been developed for the rapid detection of acute high concentrations of toxic substances in surface waters. The flow-through system is based upon the positive rheotaxis of trout in a water flow. The electronic monitoring of fish behaviour is carried out in such a way that avoidance reactions, decrease in condition and death of the test fish, either apart or in combination, can operate the alarm. Measures are taken such that a false alarm is prevented almost completely. Long term testing using Rhine water and model experiments with specific toxic substances suggest that this flow-through system is suited for practical application.

Acknowledgements

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Résumé

Un système automatique de contrôle biologique pour l'eau de surface est développé pour le détection rapide des substances toxiques d'une concentration élevée.

Le système est basé sur la propriété de la truite d'être capable de maintenir sa position quand elle nage à contre courant (rheotaxis positive). L'enregistreur électronique du comportement du poisson est construée à une manière que la reaction d'évitement, l'affaiblissement physique et le mort des poissons peut (à part ou en combinaison) donner une alarme. Des mesures sont pris pour prévenir presque totalement une alarme fausse.

Expériences pendant longtemps avec l'eau du Rhin et des expériments avec des spécific substances toxiques indiquent que le système de contrôle est propre pour l'application pratique.

Surveillance biologique automatique de l'eau puisee pour une alimentation en eau potable

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1 Introduction

L'évolution économique actuelle se traduit par la multiplication et la généralisation de l'emploi de substances susceptibles d'altérer la qualité de l'eau; nombre d'entre elles (en particulier celles issues des synthèses chimiques) possèdent une activité biologique élevée (toxicité, stimulation, altération de certaines fonctions), un comportement métabolique particulier (accumulation) et peuvent donner à l'eau ou aux organismes des goûts ou odeurs désagréables même à très faible concentration.

Il devient donc nécessaire d'assurer une surveillance continue de la qualité des eaux des milieux aquatiques, notamment en amont de certaines utilisations prioritaires (alimentation en eau potable) et de celle de certains effluents susceptibles de contaminer gravement ces milieux, afin de détecter très rapidement toute pollution toxique aiguë grave et de prendre immédiatement les mesures qui s'imposent.

Le nombre des polluants potentiels, la variabilité et la complexité de leur structure chimique rendent illusoire la mise au point de détecteurs physicochimiques universels susceptibles de déceler en permanence et instantanément dans les eaux la présence de toutes ces substances.

D'autre part, les interréactions entre les polluants (synergie, antagonisme) intervenant dans leurs effets globaux sont encore très mal connus.

Par contre, le comportement de certains organismes aquatiques peut être utilisé pour une détection globale, suffisante pour répondre aux objectifs ci-dessus.

2 Bases et contraintes physiologiques et techniques d'un dispositif automatique de surveillance

Le dispositif à mettre en oeuvre ne pouvant dans la majorité des cas être l'objet d'une observation humaine constante doit être capable de déclencher, si nécessaire et automatiquement, un signal d'alarme éventuellement transmissible à distance. Cette contrainte est d'ailleurs la plus difficile à satisfaire.

Le poisson a été choisi comme organisme test en raison des facilités d'utilisation et de sa sensibilité à l'ensemble des substances toxiques pour l'homme, et aux principaux paramètres indicateurs de la qualité des eaux (pH, teneur en oxygène, température, etc...). Cependant, d'autres organismes aquatiques (crustacés notamment) manifestent une sensibilité plus élevée à l'égard de certains polluants (Jung, 1975).

Divers essais (Barbier, 1973) ont montré qu'un échantillon de poissons soumis à un certain effort physique (nage) manifestait une incapacité à résister au courant en un temps nettement inférieur à celui nécessaire à l'apparition de mortalités pour une même proportion des sujets (50%) et pour une concentration donnée d'un toxique; la figure 1 illustre cette constatation. La détection basée sur la détermination de l'incapacité du poisson à résister à un courant d'eau est donc plus précoce que celle basée sur la mortalité.

Une sensibilité encore supérieure peut être obtenue si la détection est basée sur les effets des pollutions sur certains indicateurs des principales fonctions physiologiques du poisson (rythme cardiaque ou respiratoire). Des appareils très sophistiqués ont été mis au point sur ce principe (Morgan, 1974, Cairns *et al.*, 1974) mais leur coût, leur complication, les difficultés d'exploitation, les rendent pratiquement inutilisables pour des industriels ou d'autres utilisateurs qui ne disposent pas des installations et des personnels très spécialisés et qualifiés nécessaires pour leur mise en oeuvre.

Les organismes à retenir doivent répondre aux critères suivants:

- —disponibilité sur le marché, donc faisant l'objet d'un élevage ou d'une exploitation commerciale généralisée,
- —aptitude à vivre sans mortalité notable dans l'espace restreint du dispositif envisagé,
- —relative facilité de manipulation et de maintenance,
- —sensibilité convenable,
- —comportement constant facilitant l'automaticité de l'alarme.

L'espèce présentant le maximum d'avantages est la truite arc-en-ciel (*Salmo irideus* Gibbons) qu'on peut se procurer très facilement et qui possède une bonne sensibilité aux divers toxiques. Toutefois, un convénient important de son emploi est dû à sa température létale relativement basse (environ 26°C en 48 h). Pour les températures excédant 23–24°C pendant des périodes assez longues (cas de nombreuses rivières de plaine), il y a donc lieu d'envisager soit une substitution d'espèce, soit une régulation thermique du dispositif ce qui constitute malheureusement une complication supplémentaire.

Il faut cependant noter que dans la gamme de température convenable, la truite peut fort bien vivre dans l'eau de rivières dites à cyprinidés (sa faible densité dans ces milieux étant due, en l'absence de pollution, à la concurrence des autres espèces et à des conditions défavorables de reproduction).

S'agissant d'un test visant à déterminer globalement un certain indice de toxicité, il n'est nullement obligatoire de recourir aux espèces de poissons normalement présentes à la station considérée; la répartition de ces espèces obéit, en effet, à divers facteurs écologiques indépendamment de toute pollution ou toxicité des eaux.

Dans le cas où la température des eaux s'oppose à l'utilisation de la truite, il est possible de la remplacer par la carpe (*Cyprinus carpio* L.) d'un ou deux étés qui répond elle aussi dans une mesure acceptable aux critères définis plus haut; elle est malheureusement moins sensible dans l'ensemble que la truite aux diverses formes de pollution.

3 Principes de fonctionnement et modalités de réalisation

Le fonctionnement du détecteur est basé sur le comportement global du poisson (truite arc-en-ciel ou carpe) laissé libre dans un bac où est établi un courant d'eau de vitesse réglable prélevé dans le milieu à contrôler.

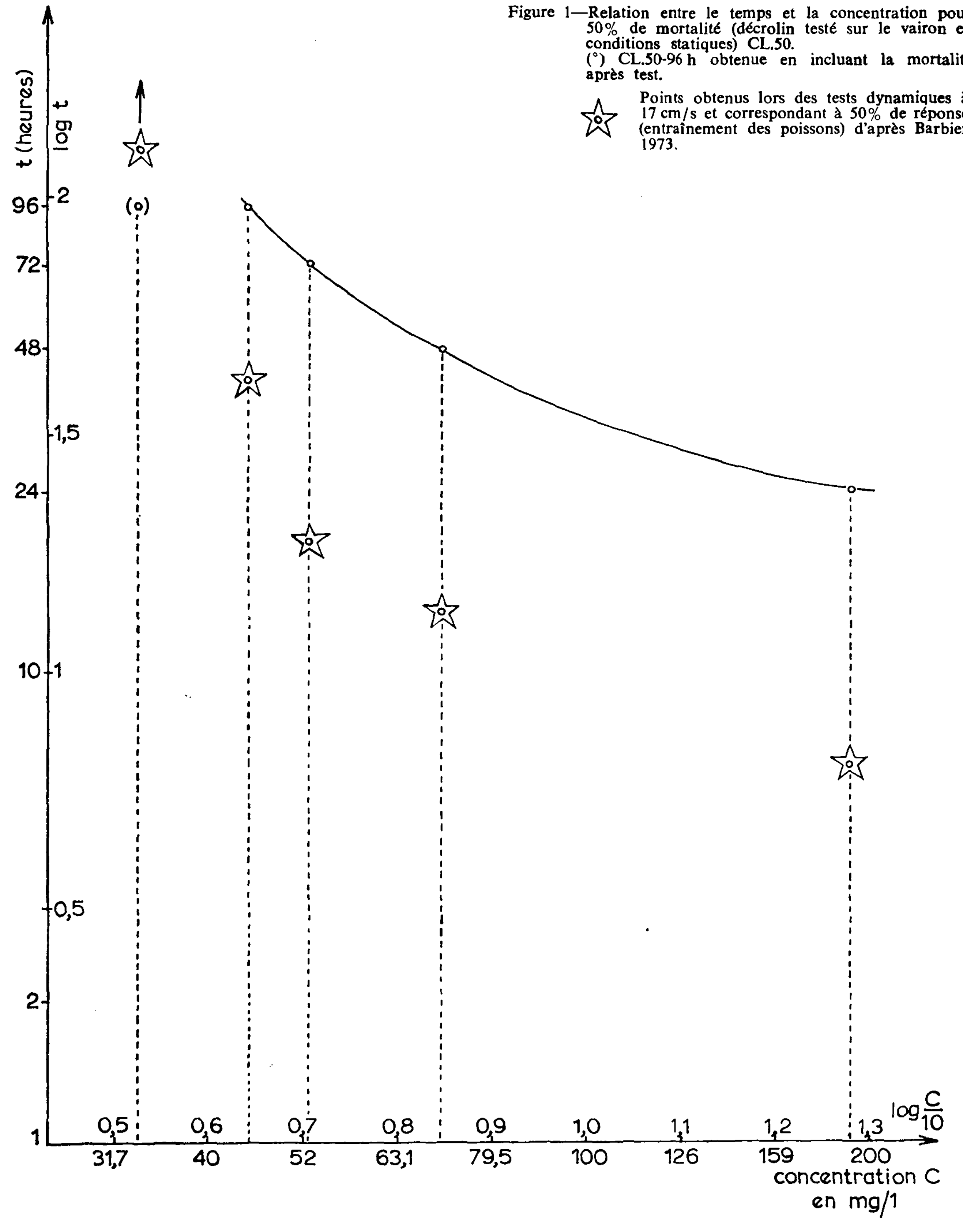

Figure 1—Relation entre le temps et la concentration pour 50% de mortalité (décrolin testé sur le vairon en conditions statiques) CL.50.
(°) CL.50-96 h obtenue en incluant la mortalité après test.
☆ Points obtenus lors des tests dynamiques à 17 cm/s et correspondant à 50% de réponse (entraînement des poissons) d'après Barbier 1973.

En temps normal, le poisson par rhéotaxie nage contre ce courant pour se maintenir dans la partie amont de l'aquarium; un champ électrique approprié disposé en aval de l'emplacement qui lui est réservé s'oppose éventuellement à ses tentatives actives ou passives de dévalaison. Ce champ électrique n'est pas permanent, il est établi automatiquement lorsque le poisson franchit les limites du secteur amont dans lequel il doit normalement se tenir. Ce franchissement est relevé par des détecteurs appropriés (cellules photo-électriques).

En cas de pollution de l'eau circulant dans l'aquarium, l'intoxication du poisson se traduit d'abord par une réaction d'agitation, ou de fuite, ensuite par une diminution de ses performances physiques le rendant incapable de lutter contre la vitesse du courant d'eau. Ces mouvements l'amènent à quitter le secteur de l'aquarium où il doit normalement se cantonner.

Les décharges électriques (de faible voltage) déclenchées par le passage du poisson vers l'aval l'obligent à revenir vers l'amont.

Les tentatives actives ou passives (sous l'influence du courant d'eau) de fuite vers l'aval du poisson se traduisent ainsi par la mise sous tension du barrage électrique à une fréquence égale à celle de ses mouvements.

A partir d'un certain stade d'intoxication, le poisson, fatigué par ces mouvements répétés et l'action des polluants contenus dans l'eau, se laisse entraîner par le courant d'eau malgré les sollicitations du barrage électrique.

Le principe de fonctionnement est mis en oeuvre selon diverses modalités en fonction de la situation du détecteur, de son mode d'exploitation et des objectifs poursuivis.

3.1 Nombre de poissons utilisés

Les études de toxicité montrent qu'il existe chez une même espèce de poissons des différences individuelles importantes sur le plan de sa sensibilité aux toxiques.

Les travaux préalables à la mise au point du détecteur et relatifs à la comparaison des sensibilités en milieu statique, d'une part, avec nage forcée, d'autre part, ont révélé la présence, dans certains lots de poissons testés, d'un ou deux individus nettement plus résistants que la moyenne aux effets d'entraînement du courant (Barbier, 1973).

Il est donc souhaitable de tester simultanément plusieurs poissons et d'enregistrer leur comportement individuel: un nombre de 3 individus s'est révélé jusqu'à présent suffisant de ce point de vue.

3.2 Déclenchement de l'alarme

L'alarme peut être déclenchée à partir des détecteurs commandant la mise sous tension du barrage électrique (Poels, 1975) ou par un autre jeu de cellules disposées en aval du barrage électrique enregistrant le passage du poisson lorsque celui-ci est irrémédiablement entraîné vers l'aval malgré l'action du champ électrique.

Le premier principe conduit à des réalisations généralement plus sensibles mais plus complexes car la commande de l'alarme ne doit intervenir qu'au-dessus d'une certaine fréquence correspondant à un comportement anormal du poisson.

Cependant, une version simplifiée peut être envisagée pour certaines utilisations comme la détection des pollutions accidentelles à l'amont des piscicultures (Leynaud, Barbier et Savary).

Le deuxième principe (fig. 2) conduit à une installation plus simple (il suffit de détecter le premier passage du poisson) mais légèrement moins sensible, encore qu'une augmentation de la vitesse du courant dans l'aquarium compense ce handicap dans une certaine mesure. Ce type de détecteur est employé en particulier dans les postes de contrôle situés en amont de certaines utilisations ne bénéficiant que d'un contrôle périodique et non d'une présence humaine constante tels que le poste de contrôle de Boran-sur-Oise. (Vivier, 1972).

3.3 Caractéristiques hydrauliques et dispositifs annexes

Selon les modalités de réalisation, la vitesse du courant d'eau dans l'aquarium se situe entre 80 et 200 mm/s.

Afin de limiter le débit nécessaire à l'installation, la largeur du canal de contention des poissons peut être réduite jusqu'à 80 mm. Dans le même but, les canaux contenant les 3 poissons tests peuvent être disposés en série.

Dans les cas où, malgré ces précautions, le débit disponible serait trop faible, il est possible d'en recirculer une partie au moyen d'une pompe auxiliaire ou d'assurer une vitesse de courant suffisante par un autre dispositif tel que hélice, roue à aube, etc. . . . (Barbier, 1973).

Il est facile de réaliser un prélèvement d'eau simultanément au déclenchement de l'alarme par mise en route d'un échantillonneur ou plus simplement par ouverture d'une vanne électrique temporisée.

Les matières en suspension peuvent être retenues dans un décanteur placé en amont du circuit, un déversoir situé dans la partie aval du canal permet de régler la hauteur d'eau dans le canal et d'éliminer les voiles superficiels susceptibles de perturber le fonctionnement des détecteurs.

4 Réalisation d'installations fonctionnelles

En France, plusieurs installations fonctionnelles ont été réalisées notamment par le Syndicat des Communes de la Banlieue de Paris pour les eaux (Compagnie Générale des Eaux, régisseur) pour la protection des prises d'eau de Mery-sur-Oise sur l'Oise et de Choisy-le-Roi sur la Seine.

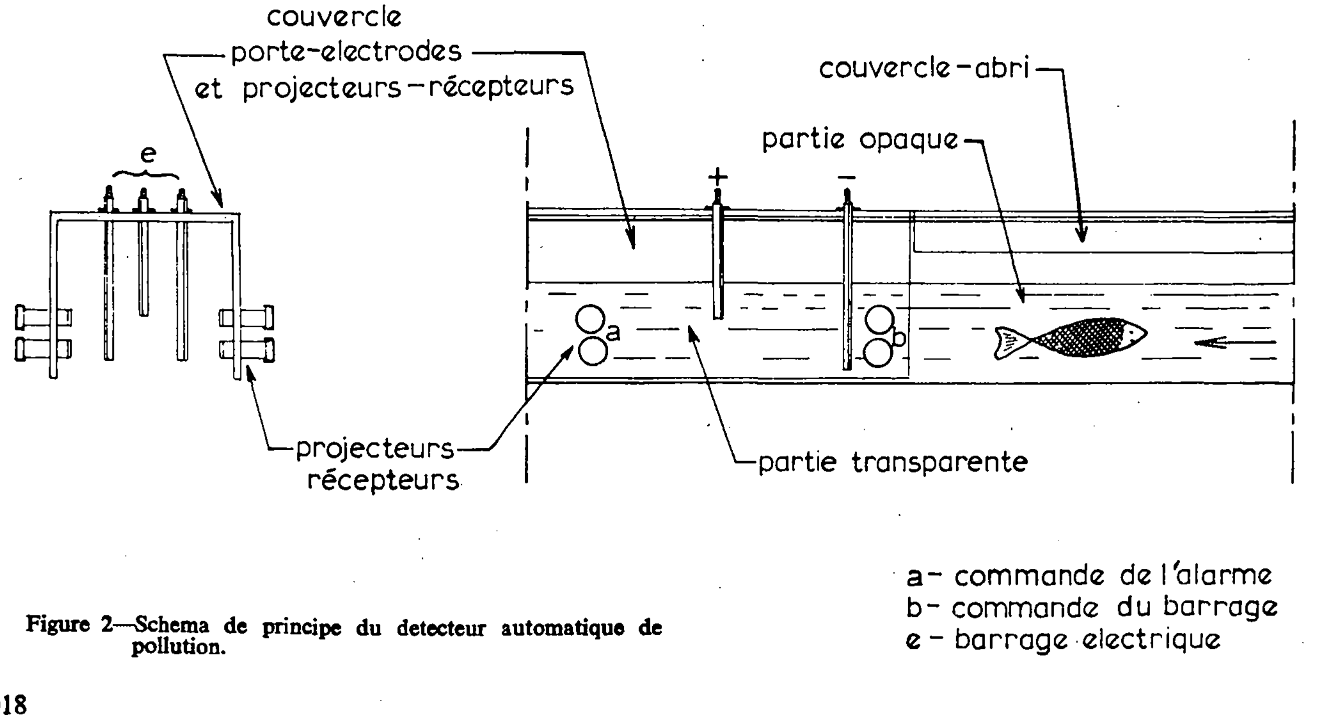

Figure 2—Schema de principe du detecteur automatique de pollution.

A cet effet, des postes de contrôles ont été installés en amont de ces prises d'eau respectivement à Boran-sur-Oise (15 km) et à Ablon (6,8 km).

Ces postes comprennent une prise d'eau en rivière, une installation de pompage, des préleveurs automatiques, des appareils de mesure en continu de certains paramètres (O_2, pH, température, turbidité, conductivité) et un détecteur biologique.

Le détecteur biologique comprend trois aquariums en série contenant chacun une truite arc-en-ciel de 200 à 250 grammes correspondant à la taille des poissons livrés à la consommation (0,20–0,25 m).

L'alarme est déclenchée lorsque le poisson est entraîné en aval malgré le barrage électrique au niveau du deuxième rang de cellules photoélectriques.

Cette alarme est retransmise à l'usine par ligne téléphonique séparément pour chaque aquarium.

Les poissons tests se comportent généralement bien dans le canal d'observation à condition de leur réserver à l'amont une zone d'abri (au moyen d'un couvercle approprié) qui les soustrait à la vue du personnel assurant l'entretien de l'installation . . . ou des visiteurs.

Lors de leur introduction dans le canal, il leur suffit de quelques minutes pour s'adapter à la contention.

Les poissons sont renouvelés toutes les semaines à l'occasion des opérations d'entretien; dans ces conditions, il n'est pas indispensable de les nourrir pendant leur séjour dans les aquariums. La vitesse du courant d'eau est de 0,15 à 0,20 m/s.

Dans l'ensemble, le fonctionnement de ces détecteurs biologiques s'est révélé satisfaisant pour la détection des pollutions aiguës.

5 Conclusions

Le test biologique basé sur le comportement du poisson complète très utilement les moyens de détection des pollutions actuellement disponibles compte tenu de sa signification globale.

Moyennant quelques connaissances de base, les opérations de maintenance qu'il nécessite sont relativement réduites et il est utilisable dans des installations fonctionnelles de type industriel.

Toutefois, si dans l'ensemble le poisson est plus sensible que les mammifères à la grande majorité des substances toxiques, il n'est pas possible d'atteindre (sauf par des dispositifs très sophistiqués et complexes) un niveau de sensibilité correspondant aux seuils de pollution jugés admissibles dans le cas d'une contamination chronique par l'eau de boisson.

C'est ainsi que les concentrations limites (très faibles) en métaux lourds pour l'eau de boisson envisagées pour les futures normes ne peuvent être détectées par le test poisson.

Ce test simplifié permet cependant une surveillance globale des milieux aquatiques et la détection des pollutions aiguës particulièrement néfastes pour les installations de distribution d'eau potable.

6 Résumé

La multiplication des substances nouvelles susceptibles de polluer l'eau utilisée pour les distributions publiques nécessite la mise à disposition de ces industries, d'un moyen de détection global des altérations graves de la qualité des eaux permettant un contrôle continu et automatique.

Ce contrôle peut être réalisé au moyen d'un test biologique utilisant le poisson (truite arc-en-ciel). A cet effet, un appareillage relativement simple a été réalisé dans lequel le poisson est contraint à se maintenir dans la partie amont d'un aquarium et à lutter contre un courant d'eau de vitesse modérée.

L'intoxication du poisson test sous l'influence d'une pollution aiguë de l'eau distribuée dans l'aquarium se traduit par son entraînement vers l'aval malgré les impulsions d'un barrage électrique de faible tension. Cet entraînement détecté par des cellules photoélectriques permet le déclenchement d'un signal d'alarme.

Des installations fonctionnelles basées sur ce principe ont été réalisées pour le contrôle en continu des eaux de la Seine et de l'Oise en amont de prises d'eau pour les distributions publiques.

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Basic criteria for water purification plants of small capacity

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1 Introduction

With the aim of developing a widespread scheme for supplying drinking water to a large number of small inhabited centres, scattered over the vast territory of the former Portuguese colony of East Africa, now the People's Republic of Mozambique, a study was carried out on the standardization of projects for water purification plants. The most important aspects of the study are presented in this paper.

This study was completed and delivered in 1973.

According to the terms laid down by the authorities who commissioned the study, the projects were to be designed in such a way that:

(a) they could serve villages with a population of between 200 and 3 500 inhabitants;

(b) they might be used in quite a general way, and easily, in each specific case;

(c) they should, in particular, incorporate the technical resources and manpower available in the territory;

(d) they should use very simple equipment which could be easily operated and maintained by unskilled personnel;

(e) they should make use of chemical products existing in the territory;

(f) they would not require the use of electric power.

Such a programme, to be applied over a surface area of nearly 800 000 km², was an attractive challenge as regards technical methods and imagination, to which the authors devoted all their enthusiasm, achieving, in their opinion, indubitably positive results.

We believe that we can present this study as an example of technology at the service of rural communities, with a minimum of obligations to the commercial interests of manufacturers and suppliers of foreign equipment, by taking advantage of all local potentialities, on a course of technological independence as followed by the undeveloped countries in solving their basic sanitary problems.

2 Structure of the study carried out

The study basically covers 4 type-projects, relating to 4 output rates for drinking water, each divided into two alternative purification schemes, and consists of:

(a) General and specific Descriptive Memorandum for each alternative purification scheme and for each rate of production of drinking water;

(b) Overall and detailed constructional drawings;

(c) Technical specifications for civil construction and for the equipment;

(d) Instructions for the operation of the purification plants.

The study is organized in such a manner that, in the light of each specific case, given the population to be served and with the definition of the alternative purification scheme that is most suitable, the type-project applicable is immediately selected and also, consequently, the Descriptive Memorandum, the corresponding Technical Drawings and Specifications, as well as the instructions for running the purification plant.

With the study already done, it is possible for the central services which in the People's Republic of Mozambique are responsible for water supply, to have permanent stocks of equipment (pipes, valves, dosing apparatus, motor-pumps, etc.) and of chemical treatment (sodium carbonate, aluminium sulphate and sodium hypochlorite), thus rationalizing the management of the requirements concerned for the whole territory.

Table

Chemical and physical composition of the raw waters examined.

	Number of samples	Sample values: Arithmetic mean	Maximum	Minimum
Turbidity (t.u.)	59	18,0	75,0	1,0
Odour	55		hydrogen sulphide	odourless
Colour (Pt–Co scale)	52	6,0	50,0	5,0
Temperature (°C)	6	25,0	29,0	23,0
pH	82	7,0	10,9	4,1
Specific conductance (μs/cm)	67	1260	23000	0,0
Hardness ($CaCo_3$) (mg/l)				
total	84	127	897	6
carbonate	51	46	239	0
non-carbonate	47	105	766	2
Alkalinity ($CaCo_3$) (mg/l)				
total	61	98	532	3
phenolphthalein	53	4	102	0
Dissolved Oxygen (O_2) (mg/l)	13	7,2	9,0	4,1
Solids (mg/l)				
suspended	11	74,9	591,2	1,7
dissolved	30	211,9	966,0	3,3
Total silica (mg/l)	54	38,4	128,7	9,5
Albuminoid Nitrogen (N) (mg/l)	16	0,1	0,1	0,0
Ozidizability (O_2) (mg/l)	73	13,4	394,0	0,5
NH_4^+ (mg/l)	70	0,2	1,9	0,0
Na^+ (mg/l)	67	49,0	674,0	2,1
Ca^{2+} (mg/l)	85	23,6	160,0	1,3
Mg^{2+} (mg/l)	83	15,4	123,7	0,2
Fe (mg/l)	83	0,4	35,1	0,0
Mn (mg/l)	66	0,5	35,1	0,0
Cl^- (mg/l)	83	95,0	1292,8	1,1
NO_2^- (mg/l)	63	0,2	2,7	0,0
NO_3^- (mg/l)	82	3,8	44,3	0,0
HCO_3^- (mg/l)	64	111,6	649,6	10,0
SO_4^{2-} (mg/l)	83	33,5	268,2	0,5

3 Basic data

In order to be able to assess the quality of the raw water to be used, a large number of physical and chemical analyses were carried out on samples taken from all over the territory. The parameters examined and the maximum, minimum and average values obtained, are shown in the Table.

The study was prepared to assess the average characteristics of the raw water, although it allows for eventual alterations to be introduced in purification (namely as regards the reagents to be used and respective doses), in such cases in which the parameters of characterization of the raw water might show important differences in relation to the average values considered.

The degree of purification ensured in any case would make it possible to comply with the International Standards of the World Health Organization for drinking water.

In establishing the number of output rates for drinking water, account was taken not only of the population of the various villages, but also the type of water distribution, i.e. whether piped to the home or obtained from public drinking fountains. On the basis of data provided by the authorities who commissioned the study, regarding the living habits of the people and urban structure of the villages, it was considered that, in general, three-quarters of the population of each inhabited centre would be served from public drinking fountains, with 50 l/cap d,

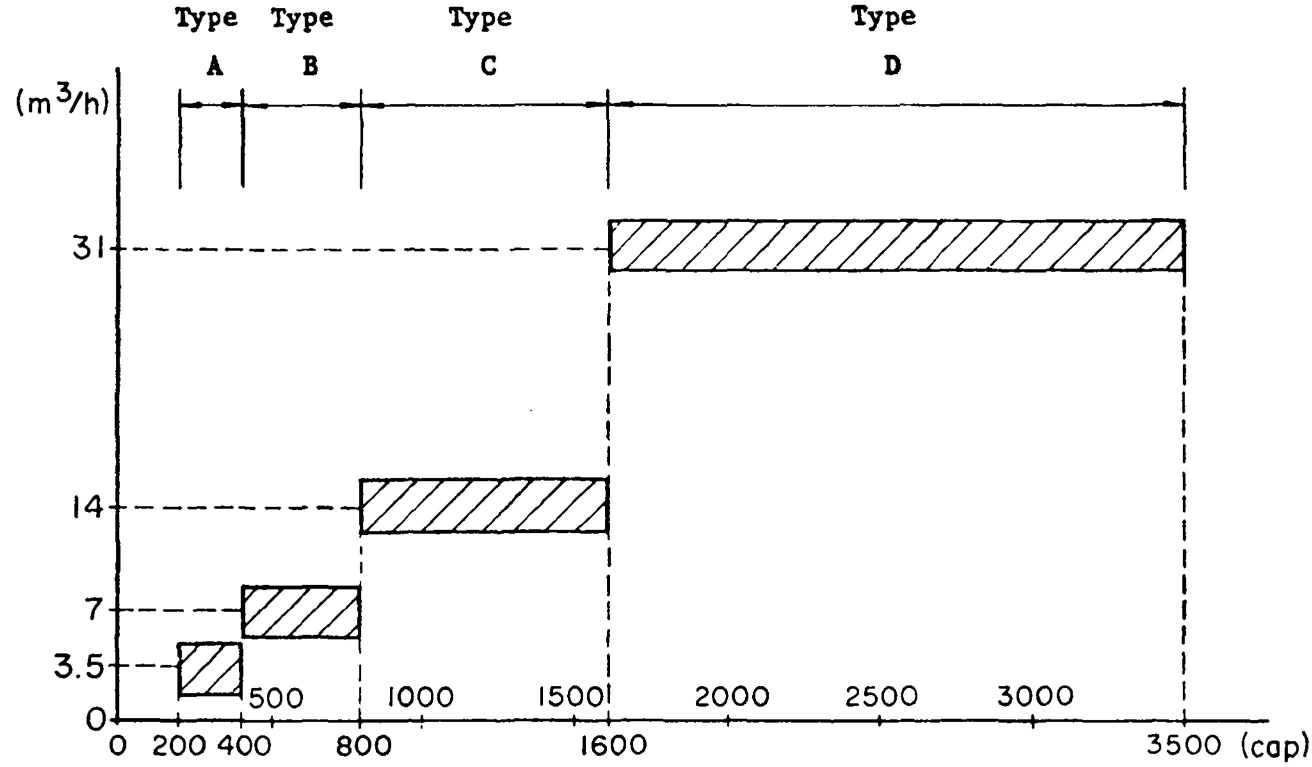


Figure 1—Production capacities and range of population served by the various type-purification plants.

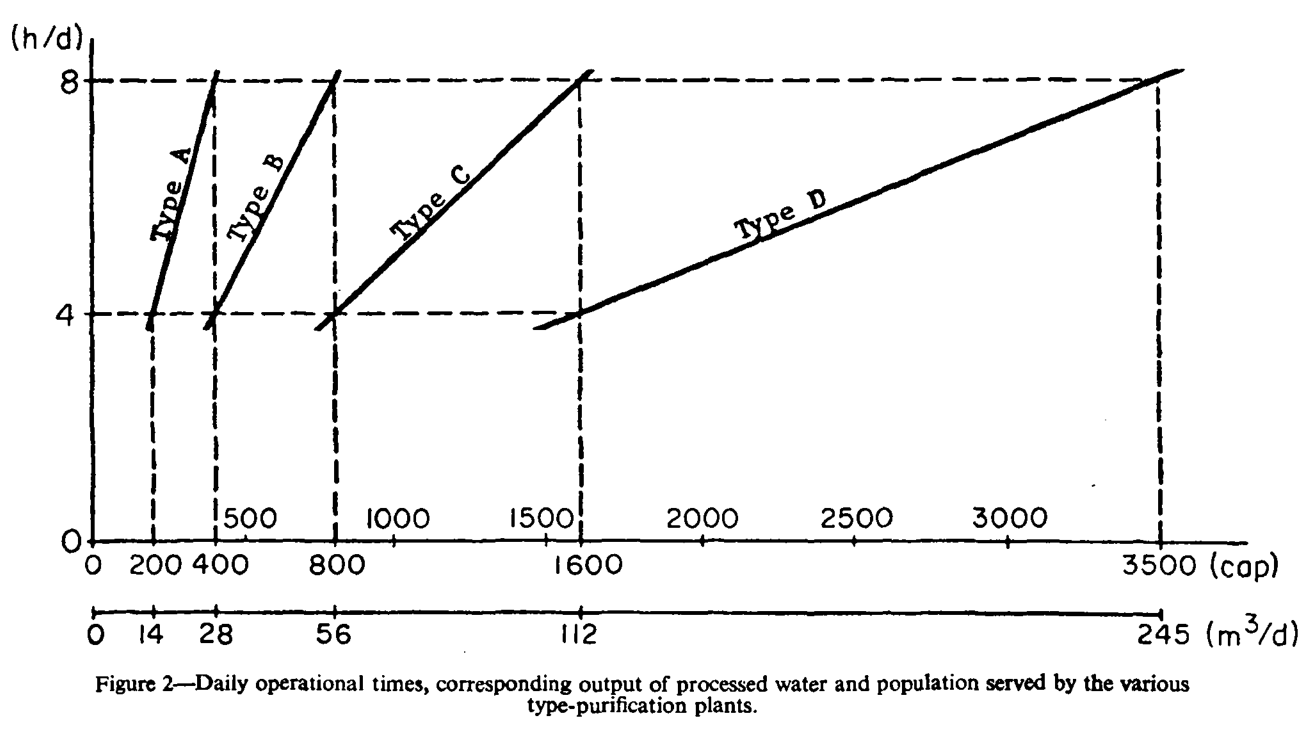


Figure 2—Daily operational times, corresponding output of processed water and population served by the various type-purification plants.

and that the remainder would be served at home, with 150 l/cap d.

Four output rates for drinking water were therefore worked out: 3,5 m³/h, 7 m³/h, 14 m³/h and 31 m³/h. Each rate was made to correspond to a type-treatment plant, such plants being conventionally designated Type A, Type B, Type C and Type D, respectively. The population ranges capable of being served by each type-purification plant, allowing only for a variation in the number of working hours per day of the plant, up to a maximum of 8 hours with a minimum of about 4 hours, are 200 to 400; 400 to 800; 800 to 1600 and 1600 to 3500 respectively, as indicated in Figures 1 and 2.

Each type-purification plant has a water tower with sufficient volume for coping with the daily needs of the maximum number of inhabitants in the respective population range.

4 Alternative purification schemes

For all types of purification plants, the following two alternative purification schemes were considered, as shown in Figure 3:

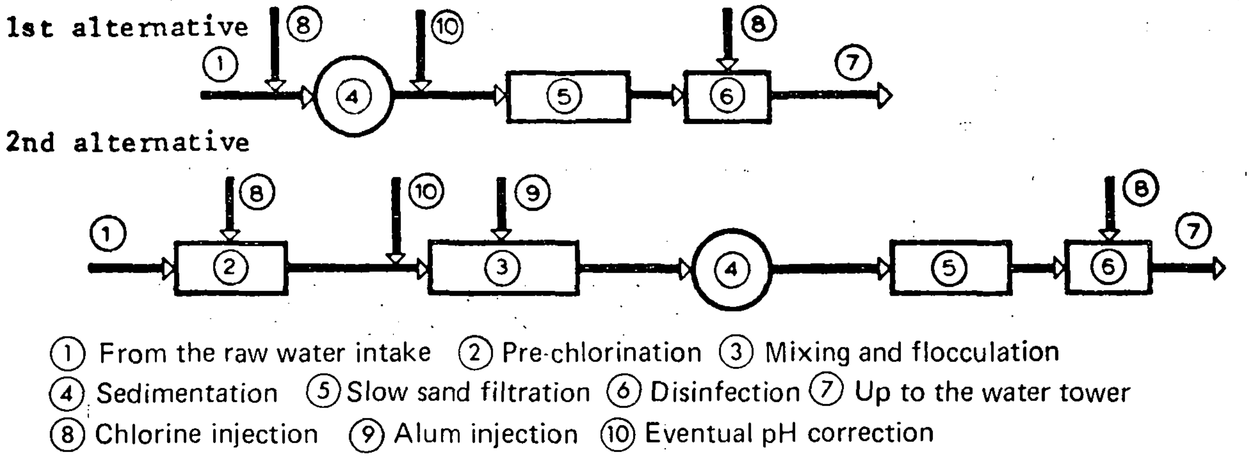


Figure 3—Alternative purification schemes.

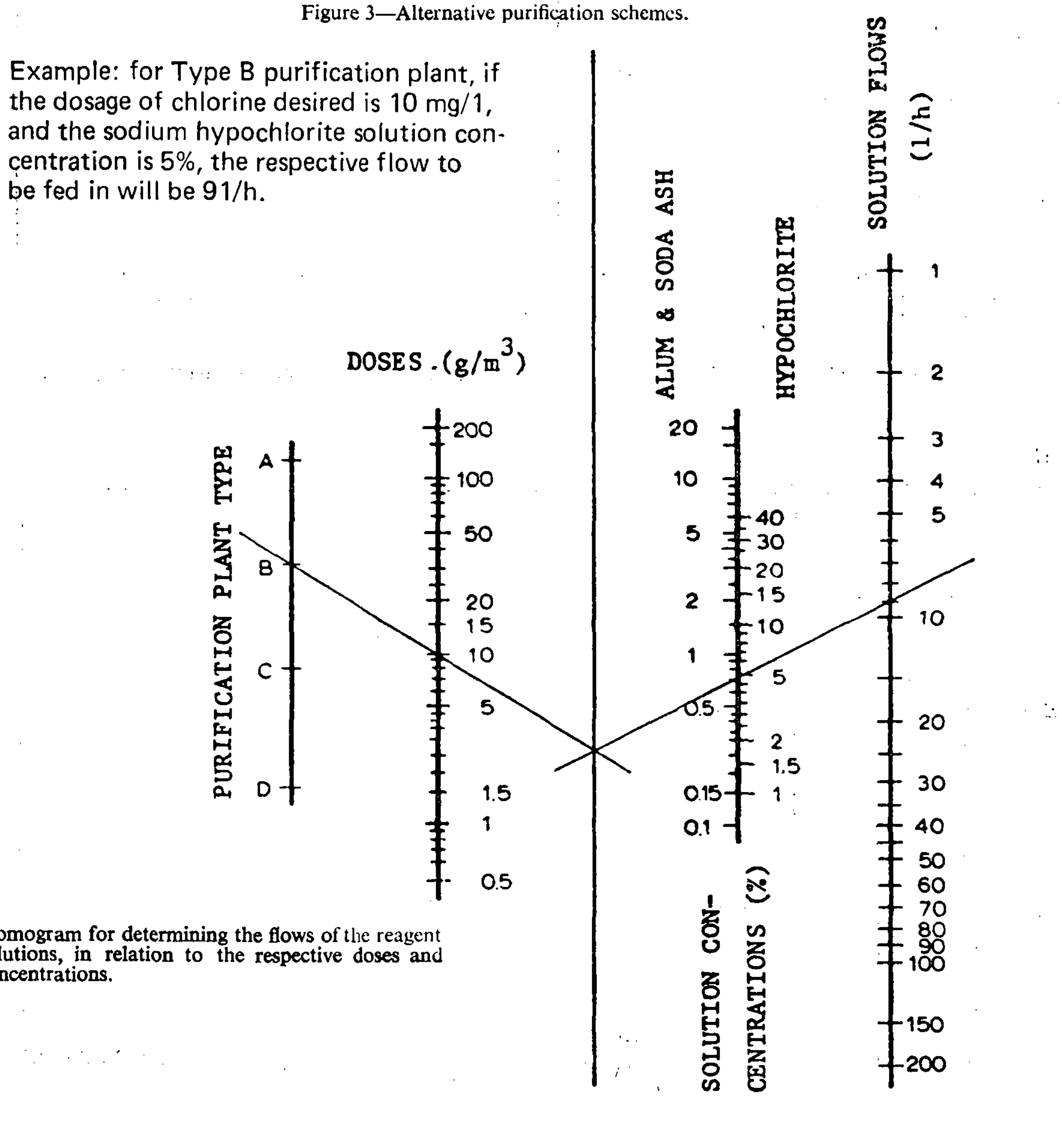


Figure 4— Nomogram for determining the flows of the reagent solutions, in relation to the respective doses and concentrations.

1st alternative

Pre-chlorination, eventual pH correction, plain sedimentation, slow sand filtration, disinfection.

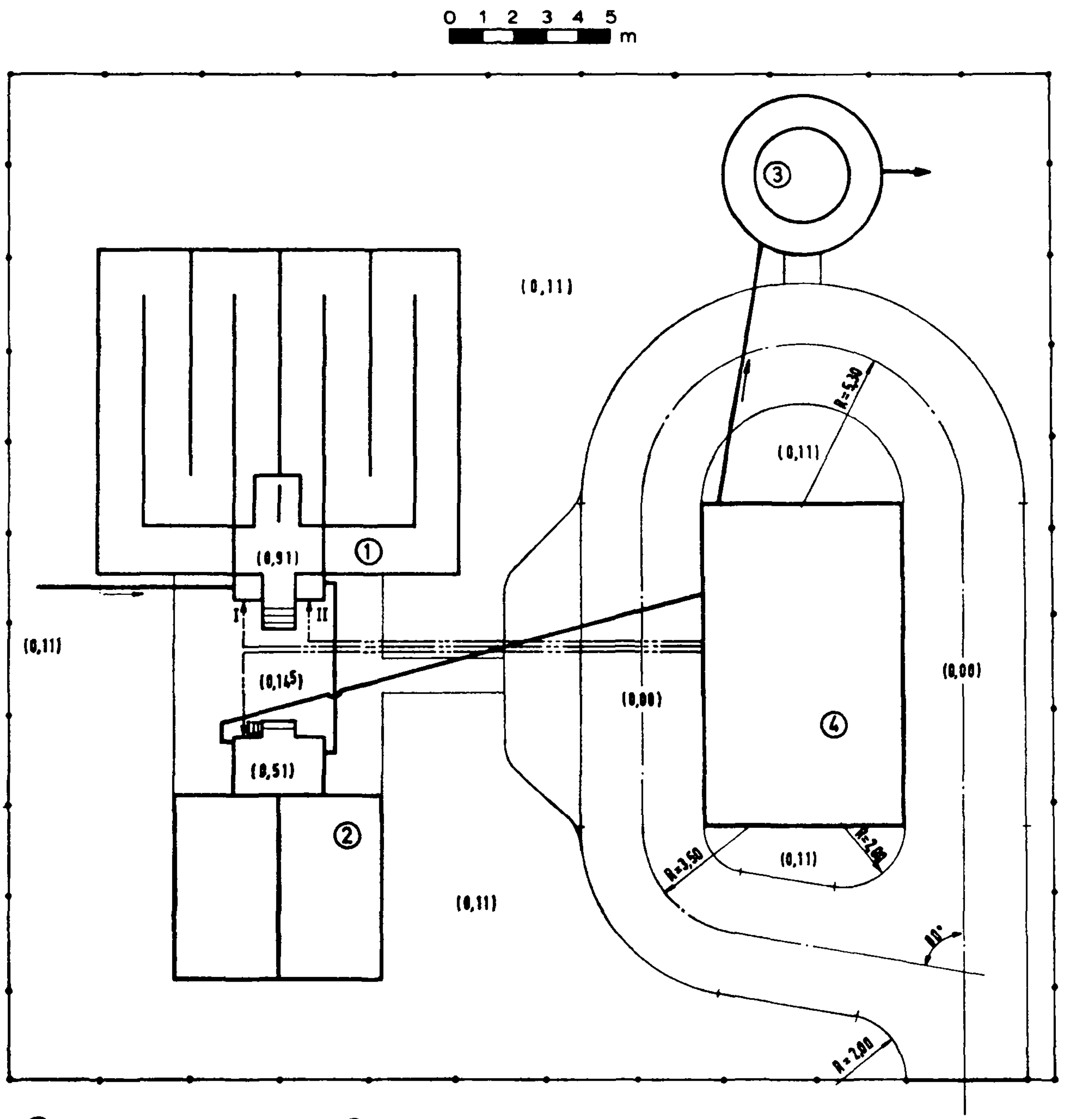
2nd alternative

Pre-chlorination, eventual pH correction, chemical coagulation, slow sand filtration, disinfection.

The first alternative would be recommended for cases where plain sedimentation could successfully ensure a convenient reduction in the load to be applied on the slow sand filters, and also where it would be justifiable to adopt the simpler purification scheme.

In any case, either of the alternative schemes is very easy to run. Resorting to slow sand filtration simplifies the purification processes, by dispensing with any washing equipment, apart from taking advantage of the reduction in the biological load provided by such filters.

The general appearance of the purification plants is as shown in Figures 5 and 6 which, although they refer to Type A purification plant, are identical, apart from the dimensions, for the other types of purification plant.



① Plain settling tank ③ Water tower

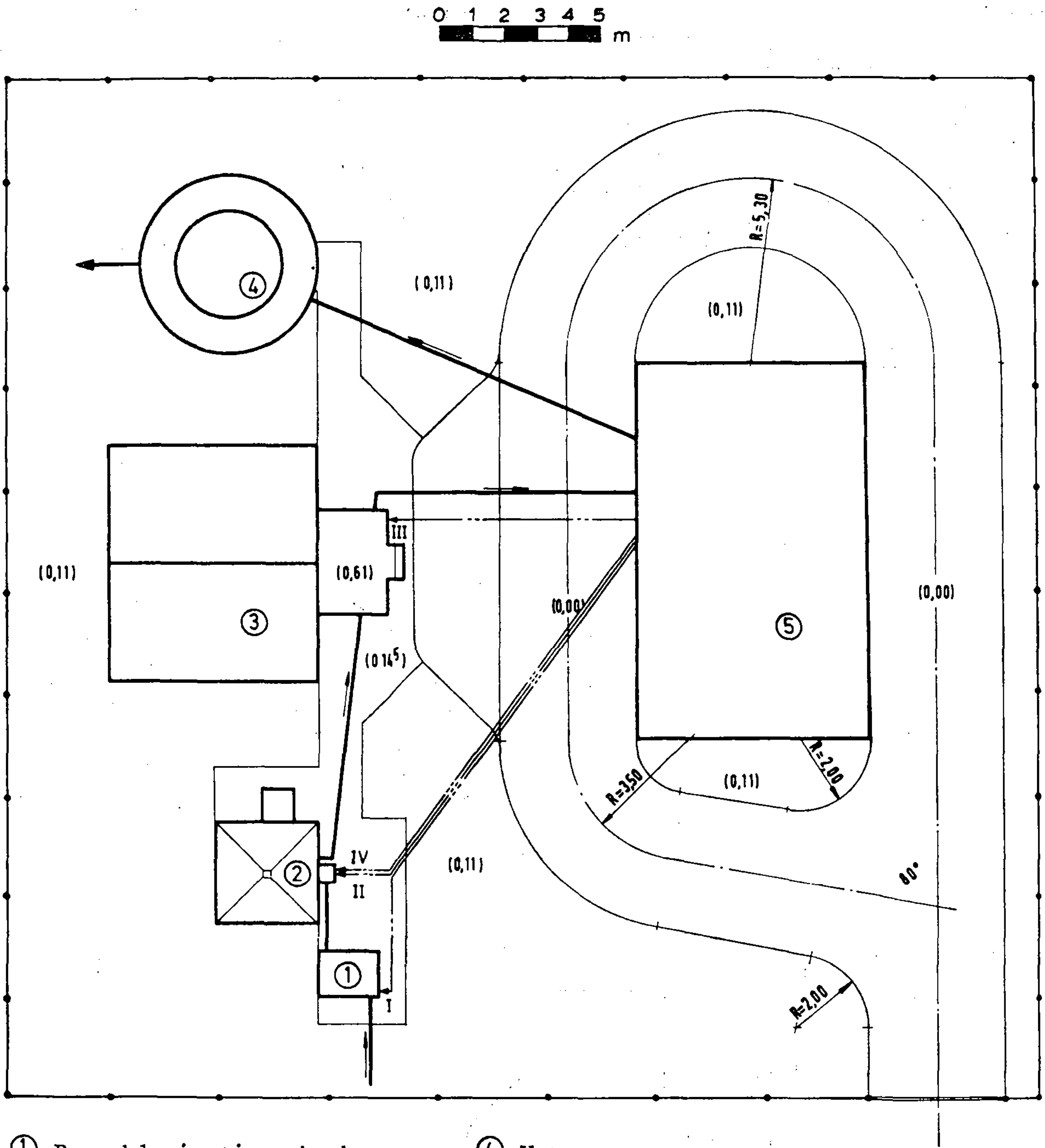
② Slow filter ④ Chemical and pumping building

——— Water pipes ———·——— Chemical solution lines

( ) Level I, II, III Chemical injection points

Note: Similar to the other type-purification plants, except as regards dimensions

Figure 5—Layout for Type A purification plant, showing alternative with plain sedimentation.



① Pre-chlorination chamber
② Vertical upflow settling tank
③ Slow filter
④ Water tower
⑤ Chemical and pumping building
I, II, III, IV Chemical injection points
——— Water pipes
——·—— Chemical solution lines
( ) Level

Note: Similar to the other type-purification plants, except as regards dimensions

Figure 6—Layout for Type A purification plant, showing alternative with chemical coagulation.

5 Dimensional criteria for the purification plant components

As regards the operations of the units involved in the two alternative purification schemes, the main components to be considered are: plain settling tanks; vertical upflow settling tanks; slow sand filters.

The plain settling tanks were dimensioned according to the following criteria: Reynolds Number of between 500 and 2000 (streamline flow); Froude Number higher than 10^{-5}; retention time of about 30 h; hydraulic load of 0,03 m^3/m^2 h; water depth 1 m.

In order to ensure values for the Reynolds and Froude Numbers within the above mentioned limits, the plain settling tanks were divided into longitudinal strips, by means of separating walls. The retention time chosen is among the highest quoted in the technical bibliography of this branch of engineering, but the increase in turbidity re-recorded at many points during the rains, which is characteristic of the torrential regimen of the rivers in the territory, made such a value advisable as a safety measure. The hydraulic load and water depth chosen, are lower than what is generally recommended in the technical bibliography, but the respective values were adopted for the same reasons that justified the choice of a long retention time.

The dimensions of the vertical upflow settling tanks were fundamentally defined as a function of the following criteria: a retention time of 3,5 h; hydraulic load between 0,4 and 0,9 m^3/m^2 h; water depth varying between 3 and 6 m.

The slow sand filters were dimensioned on the basis of the following criteria: filtering speed of 0,1 m^3/m^2 h; total depth of water above the sand 1,2 m; total height of filter material 1,85 m; effective diameter d_{10} of 0,35 to 0,45 mm; coefficient of uniformity U less than 1,7.

It should be noted that owing to the limited output capacity of drinking water by the type-purification plants planned, the adoption of high safety margins in dimensioning the various components does not lead to significant absolute increases in the respective construction costs. Furthermore, such safety margins were made necessary by the general application characteristic of the various purification plants to all the cases of inhabited centres to be supplied with drinking water.

6 Materials and equipment envisaged

All the purification components were planned in concrete, whenever possible without the use of reinforcement. The buildings are made of brick masonry roofed with asbestos cement. All of the materials used in civil construction are locally made, and the building techniques can be mastered by personnel without special qualifications.

The equipment includes:

(a) valves of various types, all hand-controlled;

(b) asbestos cement pipes;

(c) water meters;

(d) gravimetric dosing apparatus for the various reagent solutions, associated with constant-level tanks;

(e) containers made of polyester reinforced with fibreglass for the various reagent solutions;

(f) motor-pump groups, petrol-driven, for pumping the purified water up to the water tower and, as the case may be, for pumping the raw catchment water to the purification plant;

(g) colorimeters for pH and residual chlorine.

The need for importing foreign equipment is thus reduced to a minimum, and virtually confined to the motor-pump groups and other components as yet not manufactured in the territory.

7 Chemical products used

An effort was made to use the minimum amount of chemical products, and those chosen (sodium carbonate, aluminium sulphate and sodium hypochlorite) are easy to handle and do not call for special care in preparing the solutions concerned.

The main reasons for choosing sodium carbonate instead of lime, which is the most commonly used substance, were in order to avoid the slaking operations required for the lime available in the territory, and in order to use solutions instead of suspensions, with the obvious advantages as regards preparing and dosing the reagents.

The study includes graphs, such as that shown in Figure 4, on the basis of which, and in accordance with the Operational Instructions, for each product, for each concentration and each dosage it is easy to calculate the solution flow to be used.

8 Operational instructions

The instructions for Operation included in the study, were intended to provide the foremen of the various purification plant with the minimum knowledge required to make them, in the current cases of the daily routine of running the installations, independent of any outside help.

These foremen would have to be given a brief training course, during which they would be acquainted with the terms and aims of the Operational Instructions. For this purpose they would have the support and technical assistance of the central services of the authorities which in the People's Republic of Mozambique are responsible for water supply.

The Operational Instructions, drawn up in fairly simple language, without the inclusion of technical terms and explanations which might require the foremen of the purification plants to have special qualifications, consist of the following:

(a) general description of the installations and of each component in particular;

(b) explanation of the purification phenomena involved;

(c) indications as regards the treatment chemicals, namely concerning their purpose, manner of preparing the solutions and variation of the doses as functions of the concentrations and of the results desired;

(d) instructions for starting up and stopping the plant, with an indication of the sequence of operations to be carried out;

(e) standards of summary control of the quality of the water;

(f) daily routines and those at longer intervals, relating to hygiene, cleaning and conservation.

Resumé

Dans cette communication sont présentés les aspects les plus frappants d'une étude de normalisation de projets de stations de traitement d'eau élaborée pour un grand nombre de petites agglomérations éparpillées dans le vaste territoire de la République Populaire de Mozambique.

Les projets ont été conçus de façon à ce:

(a) qu'ils soient destinés à des localités de 200 à 3 500 habitants;

(b) qu'ils soient utilisés en général et de façon simple dans tous les cas;

(c) qu'ils incorporent de façon déterminante les moyens techniques et la main-d'oeuvre disponibles sur le territoire;

(d) qu'ils emploient un équipement très simple d'exploitation et de manutention accessible à un personnel non qualifié;

(e) qu'ils utilisent des produits chimiques existants sur le territoire;

(f) qu'ils dispensent du recours à l'énergie électrique.

On a opté pour des schémas de traitement incluant la pré-chloration, la filtration lente et la désinfection et en plus, alternativement, la décantation simple dans certain cas, et la coagulation chimique dans d'autres cas.

On a fixé 4 échelons de production d'eau potable, de 3,5 m^3/h, 7 m^3/h, 14 m^3/h, et 31 m^3/h, auxquels on a fait correspondre une station de traitement-type, chacune d'elles pouvant desservir, selon la durée quotidienne de fonctionnement variant d'à peu près de 4 à 8 h, respectivement, des agglomérations de 200 à 400 habitants, de 400 à 800 habitants, de 800 à 1 600 habitants et de 1600 à 3500 habitants.

L'étude a compris pour chaque projet-type, à part le mémoire descriptif, une collection de dessins constructifs, des spécifications techniques et des instructions d'exploitation.

Nous pensons pouvoir présenter cette étude comme un exemple de la technique au service des communautés rurales avec un minimum de liens à intérêts commerciaux de la part des fabricants et fournisseurs d'équipment étrangers, en tirant partie de tout le potentiel local dans le sens d'une indépendence technologique des pays sous-développés pour la résolution de leur problèmes sanitaires de base.

Pipes and pipelines: Design criteria and experiences in the uses of various materials

by R. Y. Bromell

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1 Preface

This report has been drawn up following a review of information supplied by members of the International Standing Committee on Water Distribution:

Mr R. J. Weiss	Austria
Mr M. Chalet	Belgium
Mr T. Tchachev	Bulgaria
Mr K. C. Hassabis	Cyprus
Mr P. Kieler Jensen	Denmark
Mr J. Liimatainen	Finland
Mr R. Chappey	France
Dr H. Tessendorff	Germany
Mr E. C. Reed	Great Britain
Mr R. Gurevitz	Israel
Dr Ing. Pierluigi Martini	Italy
Mr W. C. Wijntjes	Netherlands
Mr G. A. Longe	Nigeria
Mr Janczewski	Poland
Mr A. M. Santos	Portugal
Mr R. J. Laburn	South Africa
Mr D. Pedro Grau Berdaguer	Spain
Mr D. Antonio Renedo	Spain
Mr Olle Niste	Sweden
Mr E. Shaw Cole	U.S.A.

In addition the author acknowledges the valuable help received from a large number of water undertakings, companies and individuals, some of whom are listed in Appendix A.

This paper is intended to cover pipes having internal diameters between 100 mm and 1 000 mm. For convenience they are designated between 100 mm and 300 mm as being small, over 300 mm and up to 600 mm as medium, while those pipes greater than 600 mm are defined as large. Pipes having an internal bore of less than 100 mm are not considered.

2 Introduction

Since the days of the Roman Empire aqueducts have been used to convey water from the source of supply to the point where it is to be used. Some of these aqueducts are still in existence and one can but wonder how many pipelines that are being laid today will still exist after the next two thousand years. A large proportion of capital investment in water supply is buried in pipes and careful consideration has to be given to the selection of the most appropriate material.

In order to assess performance it is necessary to examine pipe design, trench bedding requirements, trench loading, impact loading, hydraulic pressure resistance, permissible deflection, factors of safety, types of joints, resistance to corrosion and chemical action, flexibility of installation as well as the storage and handling of materials. After taking into account the cost of making repairs and the economics of making future connections there is little doubt that the final decision will depend upon these factors moderated by a strong element of personal choice.

3 Design of pipelines

3.1 General

Nearly 100 years ago pipes were thick, rigid and subject to little disturbance. Since that time operational requirements have become far more exacting due to higher pressures, tighter control of leakage and greater trench loadings. Over the same period pipe manufacturers have improved their technology, high strength materials have become available and economies have been achieved by a reduction in wall thickness. Thick walled pipes were capable of withstanding all vertical loads likely to be applied in normal installations, but thin walled pipes of today mean that consideration must be given to the supporting effect of the soil. Pioneer work on the application of soil mechanics to the design of pipelines was undertaken by Spangler, Marston and Schlick and showed how the bedding around a pipe can influence structural strength.

3.2 Rigid pipes

Asbestos cement, grey iron and prestressed concrete pipelines may be regarded as being rigid conduits that fail by rupture of the pipe walls at small deflections. The ability of a rigid pipe to carry an external load is directly related to structure ring strength which can be determined by the three edge bearing test. This takes into account the lateral pressure exerted against the sides of the pipe by the backfill material, the degree of compaction and the shape of the trench bottom. There are many conflicting standards for excavated and selected material for pipe embedment and a personal selection from various authorities is shown in Figure 1. Bursting pressures consist of known operating pressures plus surge or water hammer overpressures that require careful calculation and vary with the pipe diameter, pipe material, configuration of the pipeline and the method of operation. Surge suppression devices are frequently used to restrict the magnitude of water hammer.

3.3 Flexible pipes

Ductile iron, steel and pipes manufactured from various plastics materials tend to have a low inherent strength and some more than others deflect under vertical loadings. As the sides move outwards passive soil pressures are induced in the embedment and the development of more flexible pipe materials places a greater reliance on the backfill to withstand the vertical loads. Reference is made to the selected recommended embedments for ductile iron, steel and plastics pipes as shown in Figure 1. Smith discussed this point in greater detail

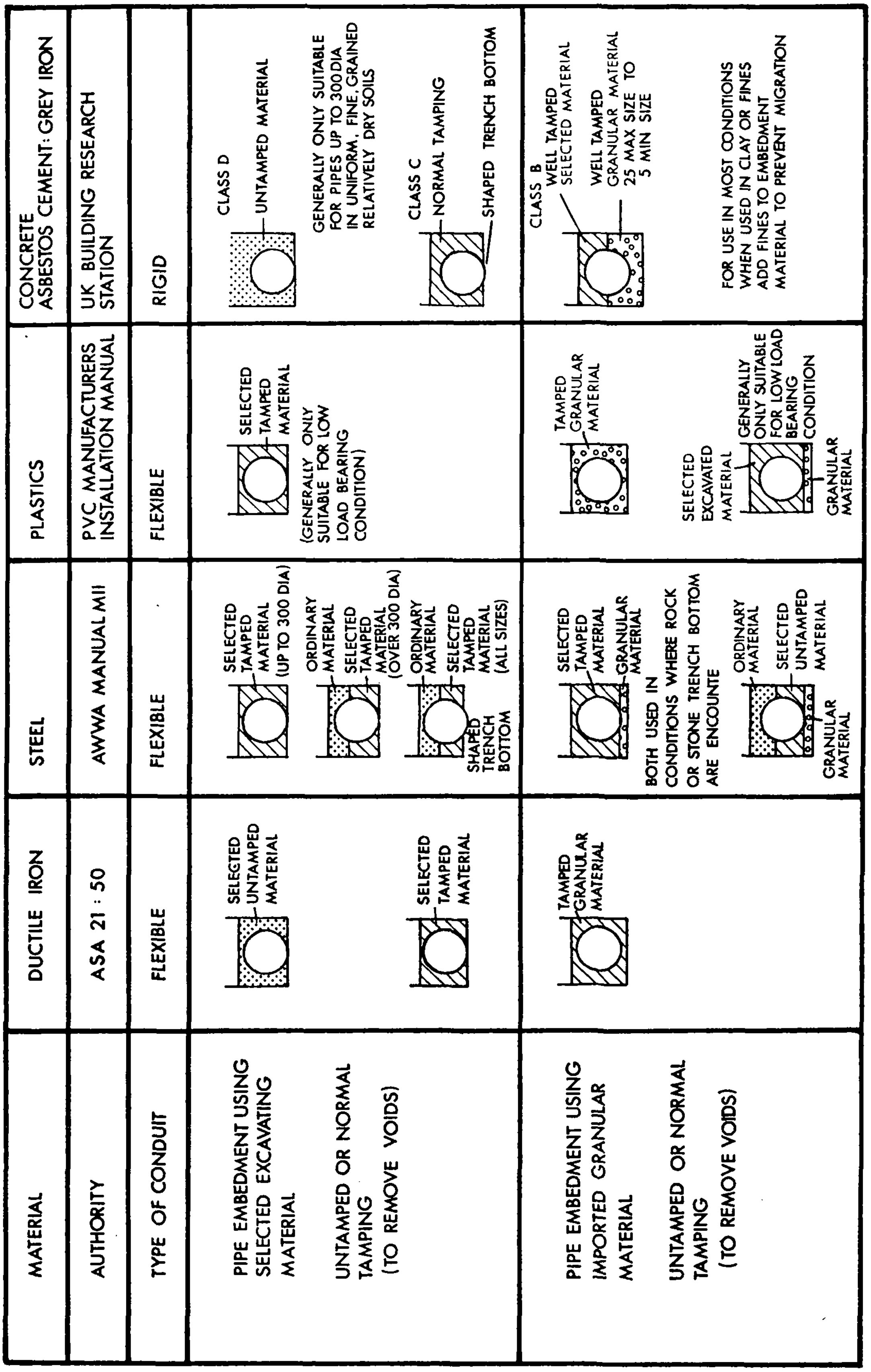

Figure 1—Recommended embedment for pipes of Differing materials.

earlier this year and suggested that the ASA 21.50 recommendations were very conservative. External loadings require deflection, bending, compression and buckling analysis. Uneven bedding, variations in backfill, temperature fluctuations and internal pressures give rise to longitudinal stresses. Checks are made for stress corrosion, cyclic strength and impact properties. It is generally accepted that the effect of transitory loads on flexible pipes is less than on rigid pipes, but some authorities assume the full rigid pipe load until more reliable facts are available.

The U.K. Construction Industry Research and Information Association (CIRIA) is about to publish a report on the design and construction of thin walled buried pipes where the wall to thickness ratio is such that the soil structure interaction is essential for stability. The pipe materials considered cover plain steel, corrugated steel, unreinforced plastics and reinforced plastics. It is understood that from a review of existing design methods, the recommended design procedure includes guidance on the suitability and properties of various soils to be used as backfill. The aim of this report is to show how to gain the full economic advantage from the soil/structure interaction.

3.4 Soil conditions

Where it is possible there are obvious advantages in using excavated spoil instead of having to import material for backfill around a pipeline. Most coarse grained soils are suitable for embedment provided stones larger than 30 mm diameter are not placed in contact with the pipe although larger stones up to 80 mm diameter may not be allowed in the remainder of the embedment. The best material is probably a mixture of soil with fines and graded gravel that will compact down into a dense mass. Examination of a route for a proposed pipeline is by samples taken from trial excavations and submitted to chemical examination together with a soil resistivity survey to ascertain whether the soil may be considered aggressive to certain pipe materials.

If there is a change of direction or internal diameter or a valve or a connection in a pipeline then the dynamic and static unbalanced forces arising from water velocity and internal pressure should be resisted by suitable anchorages. Special consideration should be given to pipes on steep slopes and to pipelines laid in bad ground having poor bearing pressure. Pipelines with fixed joints such as welded steel mains require careful attention as the pipe being continuous carries a diminishing stress away from the bends as the ground resistance to sliding accumulates.

The possibility of ground movement affecting the stability of a pipeline may be significant. In areas where subsidence or earthquakes may be encountered then either short lengths of ductile iron pipes or all welded steel pipes appear to be the most suitable. For pipes laid in poor ground suitable compaction of the embedment may meet the design specification. Where very poor ground is encountered then it may be necessary to excavate a trench up to five times the pipe diameter. Suitable embedment material may then be imported to an adequate level of compaction without support from the original ground. In really unstable conditions pipes should be placed on piles, one for each pipe, situated immediately behind the socket. This enables movement in one pile to be taken up in the joints on either side.

3.5 Pipe joints

3.5.1 Rigid joints

Pipe joints are rigid, semi-rigid or flexible. Rigid flanged joints are popular for exposed pipelines; particularly for pipework in pumping stations and treatment plants where disconnection may be required to be easy and quick. Welded joints on steel mains are used particularly for large diameter pipes where the internal welds can be made efficiently. Solvent joints were used extensively when small diameter plastics pipes were first introduced but have proved unsuccessful.

3.5.2 Semi-rigid joints

Hot run lead caulked joints on grey iron mains were used widely fifty years ago before being replaced by the introduction of flexible joints. The big disadvantage was the use of highly skilled labour necessary to make effective water tight run lead joints. Lead has the mechanical property of being very plastic and under slow progressive ground movement it flows under stress and allows the joints to take up considerable deflection without leaking.

3.5.3 Flexible joints

Flexible joints of a patented proprietary nature were developed to overcome the problems of run lead. Early mechanical joints of the bolted or screwed type incorporated a gland or sealing ring and this lead to the present rubber ring seal push in type of joint which is flexible and easy to make. Basic requirements for making most push in type joints are cleanliness and the avoidance of grit that can affect the efficiency of the seal between the rubber ring and the pipe. Proper location of the component parts is important. Some proprietary joints can tolerate abuse more than others, but the speed and simplicity of making the modern push in type of joint has led to rapid and widespread acceptance.

3.6 Hydraulic friction in pipelines

Much has been written in recent years on the limitations of empirical formulae for the hydraulic friction in pipelines. Hazen-Williams is still used widely for various types of pipe and takes into account the roughness of the bore either actual or predicted. Modern pipes have a coefficient C ranging from 135 to 155 and these figures may be maintained provided that there is no attack or erosion of the inside of the pipes and no internal deposition. Build-up of even a thin layer of slime can lead to a rapid reduction of C value; a few millimetres of deposit can effectively reduce the calculated diameter of a pipe by several centimetres. Blair, Colebrook and Lamont besides others have produced modified formulae for smooth pipes. Judicious choice should be made of the most suitable formula and due allowance must be provided for entry losses, bends, tees, changes in diameter, fittings, valves, meters and exit losses before arriving at the proper hydraulic conditions.

3.7 Standards and codes of practice

National and international standards and codes of practice for pipe materials are numerous and are being revised continuously. U.K. standards are published by the British Standards Institution and many have been metricated in recent years. In the U.S. the American Waterworks Association publishes its own standards. Many national standards are now based on recommendations produced by the International Standards Organisation. Relevant codes and standards are set out in Appendix B.

3.8 Comparative statistics

With a wide range of materials available to pipeline designers the selection of the most suitable can be a lengthy and laborious task. To illustrate the variations in mechanical properties and facilities available in the U.K. Table No. 1 has been drawn up showing the normal range of comparative design statistics for various pipe materials.

3.9 Factors affecting pipelines subject to traffic loadings

Work in the U.K. has been undertaken by the Transport and Road Research Laboratory. Trott and Gaunt recently presented a report on studies made during the construction of a main road when it was found that the most severe loadings on pipes occurred during the construction period when heavy contractors' vehicles traversed the pipes before the road was completed. Subsequent loading by road traffic produced lower strains and deflections.

A decade ago, Page carried out tests on concrete pipes laid under a road to investigate the impact factor produced by several different types of lorry travelling over a severe surface irregularity consisting of a length of timber 45 mm thick by 250 mm wide. With vehicles travelling at speeds varying between 3½ and 55 km/h the impact factor increased with speed. However, the relationship between impact factor and vehicle speed was found to be independent of size of pipe, its depth below the road surface, the type of pipe, the pipe bedding and the material used to backfill the trench.

fittings in the U.K., it is suggested that the same parameters are relevant to pipe materials, linings, jointing compounds and lubricants used in making joints and which can come into contact with water being transported.

Methods cover test procedures for assessing the ability of materials when used in contact with water to produce taste, odour, colour, turbidity or toxicity in the water or to support microbiological growth. Acute toxicity is determined by seeding water, in which the material has been soaked, with monkey kidney cells, applying standard tissue culture techniques and examining for toxic effects. Organoleptic and physical assessments for taste, odour, colour and turbidity use the

TABLE No. 1

COMPARATIVE STATISTICS FOR VARIOUS PIPE MATERIALS

	Ductile Iron	Steel	Prestressed Concrete	Asbestos Cement	Glass Reinforced Plastics	Grey Iron	Polyvinyl Chloride
Tensile Strength (MN/m^2)	420	340 to 420	214 (Cyl)	22,5	150–500	18–40	45–60
Young's Modulus (GN/m^2)	165	207	30	23,5	10–25	100	4
Elongation	7–10%	According to grade	½%	Nil		< 1%	40% min
Impact (IZOD) Resistance (Joules)							4–4,5
Beam Strength (MN/m^2)	500	Depends on grade		24	15	95	92
Compressive Strength (MN/m^2)	300	Depends on grade	40	45	70	615	68
Design fact of safety	2,5	2–2,5	1,8–4	4 bursting 2,5 crushing		2,5	1,5 min
Max. Working Temp. (°C)					30		60
Linear Thermal Expansion (°C)	11×10^{-6}	11×10^{-6}	11×10^{-6}	$11{,}8 \times 10^{-6}$	27×10^{-6}	11×10^{-6}	$50\text{–}60 \times 10^{-6}$
Thermal Conductivity (Cals cm/sec/cm^2/°C)	13×10^{-2}	12×10^{-2}	24×10^{-4}		63×10^{-4}	12×10^{-2}	35×10^{-5}
Density (kg/m^3)	70×10^2	78×10^2	$26\text{–}28 \times 10^2$	22×10^2	25×10^2	70×10^2	18×10^2
Hazen Williams Flow Coefficient (C)	135/150 (lined)	152 (lined)	150	140/155	150	148	150
Water Absorption	Nil	Nil	1,1–2% by weight	Up to 20% by weight	0,1	Nil	0,1
Diameter range (mm)	80–1 200	60–2 140	400–3 000	50–900	300–3 000	80–700	12,5–600
Type of Joints	Mech, push-in, flange, couplings	Weld, push-in, couplings	Push-in, couplings	Push-in, collar, couplings	Push-in, couplings	Lead, push-in, couplings	Push-in, solvent weld
Type of Fittings (material)	Ductile	Steel	Steel C/L/C	Ductile Steel	Steel, Ductile	Grey Iron	PVC iron or steel
Pressure Ratings (working) (bar)	25/40	16/70	4/18	7,5/12,5	6/16	10/16	6/15

4 Suitability of materials used in contact with potable water

Reference is made to the problems of taste and toxicity with early plastic materials. BS 3505 requires that PVC pipes will not have any detrimental effect on the composition of water flowing through them. Carbonic acid is used to extract any metals or other toxic substances and determinations made for lead, dialkyl C_4 and higher homologues and other toxic substances. Advanced methods for testing the suitability of materials for use in contact with water used for domestic purposes have been produced recently. Although such tests were evolved specifically for the testing of materials used in water standard methods described in "Analysis of raw, potable and waste waters". Taste and odour tests are repeated using tap water with 1 mg/l of chlorine in order to detect chloroderivatives. An assessment for heavy metals should be within the limits specified by the World Health Organisation European Standards for Drinking Water. Microbiological growth tests are carried out by inoculating the soak water with a mixture of micro-organisms which are likely to include those capable of utilising as a source of nutrient a variety of natural and synthetic organic materials in an aquatic environment. Samples are examined quantitatively for coliform organisms, bacteria capable of growth at 37°C and 22°C Pseudomonas aeruginosa, fungi and yeasts. It is essential that these tests are carried out or supervised by qualified chemists and microbiologists who are familiar with the standard methods and media.

5 Life expectancy

In a recent survey water undertakings estimated the life expectancy of modern pipeline materials and these are shown summarised in Table No. 2.

TABLE No. 2

Pipe material	Expected life in years		
	Minimum	Maximum	Average
Grey iron	50	100	80
Prestressed concrete	50	100	73
Asbestos cement	50	100	65
Ductile iron	50	100	65
Steel	30	100	56
Plastics	50	67	*

* *statistically unreliable*

As may have been expected, a traditional material like grey iron had the longest expectation of life but was followed closely by cylinder and non-cylinder prestressed concrete pipes. Reservations may have been expressed about asbestos cement because it is likely to be laid in highly corrosive conditions. Ductile iron as a new material is treated with suspicion, but steel is also regarded with some doubts presumably because of possible failure of protective coverings to prevent corrosion. The statistics were unreliable for plastics materials but none gave a life expectancy greater than 67 years. It would appear from this table that the waterworks engineer's natural reluctance to accept new materials until absolutely convinced of longevity is well illustrated.

6 Trends in the use of different materials in the United Kingdom

Last year in the U.K. the Department of the Environment published a useful Report of a Working Party on Sewers and Water Mains that sets out views on trends in the choice of materials for water mains. Ductile iron is steadily replacing grey iron, particularly in the smaller sizes. This change has been accentuated by manufacturers' policy, nevertheless the use of ductile was generally favoured. Some reservations have been expressed regarding the life of ductile in comparison with grey iron due to the action of corrosion on the thinner pipe wall. Such doubts have been discounted by the research carried out by Collins, Fuller and Harrison and published by the British Cast Iron Research Association.

Steel pipes with bitumen linings and welded joints are predominantly used for large diameter trunk mains and it is thought that this practice would continue. Since ductile iron pipes are now being made in large diameters there might be a change from steel, although the heavier pipe could be a significant factor.

Prestressed concrete pipes are not at present being used extensively possibly due to the extra weight per metre of pipe. Asbestos cement pipes are being used in certain areas particularly where there is corrosive ground. In rural or mixed rural and urban areas either asbestos cement or plastics are used more frequently.

PVC pipes in small diameter sizes are used because of their corrosion resistance and ease of handling, particularly in rural areas. Push-in type joints are regularly used except when laying by means of the mole-ploughing technique when solvent weld joints are necessary.

7 Comparative statistics on pipe failures

7.1 General

Experience varies widely in comparing statistics for different pipe materials. This is not surprising in that ground conditions, laying methods, depth, traffic and temperatures vary from site to site. An interesting example is given in Table No. 3 which shows the rate of failure in street water mains in twelve selected Swedish towns from April 1974 to March 1975.

TABLE No. 3
FAILURES IN SELECTED SWEDISH TOWNS 1974–1975

Pipe material statistics					
Pipe material	Number of failures			Pipelines length in km	Failures per km
	Trunk mains	Dist. mains	Total		
PVC	49	55	104	302	0,344
Steel	23	32	55	488	0,113
Grey iron	193	164	357	3 585	0,100
Asbestos cement	1	1	2	46	0,043*
PEL/PEH	3	6	9	314	0,029
Ductile iron	2	3	5	333	0,015
Concrete	1	0	1	296	0,003
Others	2	7	9	—	—
Total	274	268	542	5 364	0,101

* *statistically unreliable*

Major causes of failures			
Cause	Grey cast iron	Steel	PVC
Corrosion	11%	81%	—
Uneven settlement	42%	2%	11%
Faulty material	—	—	37%
External influence	6%	2%	4%
Not specified	12%	—	34%
Not known	29%	15%	14%

In comparison Table No. 4 shows similar figures provided for two large cities in Poland from 1971 to 1974.

TABLE No. 4
FAILURES IN TWO LARGE POLISH CITIES
1971–1974

Pipe material statistics			
Pipe material	Failures per km/year		
	Small dia.	Medium dia.	Large dia.
Grey iron	0,2–0,32	0,37	0,45–0,86
Steel	0,09	0,03	
Asbestos cement	0,12–0,21		
Plastic	0,22		

7.2 Grey cast iron

Roberts and Regan have carried out a detailed investigation into the causes of fractures in grey cast iron water mains for the former Metropolitan Water Board, London, England over the period up to 1974. It was reported that the average fracture rate was 0,16/km/a for a total length of all sizes of approximately 15 000 km and it was found that the principal type of fracture was the transverse break, ranging from 90% in non-corrosive soil to 40% in a corrosive area. The rate of fractures decreased as the pipe size increased. Age was not thought to be a primary cause of failure, unless in a corrosive soil. In medium and large diameter pipes the main form of failure was longitudinal splitting due to ground pressure. It was suggested that mains under heavily trafficked major roads did not suffer unduly because highway authorities recognised the heavy-loading and had provided suitably strong road structures. Conversely, mains under the footpaths of major roads suffered, as did mains at road junctions. Mains seldom fractured under lightly trafficked roads, but suffered repeatedly where unusually heavy traffic turned on to a side road leading, for example, to a factory.

A study of meteorological data indicated that a spate of fractures followed a drop in air temperature. Figure 2 shows the relationship between average daily

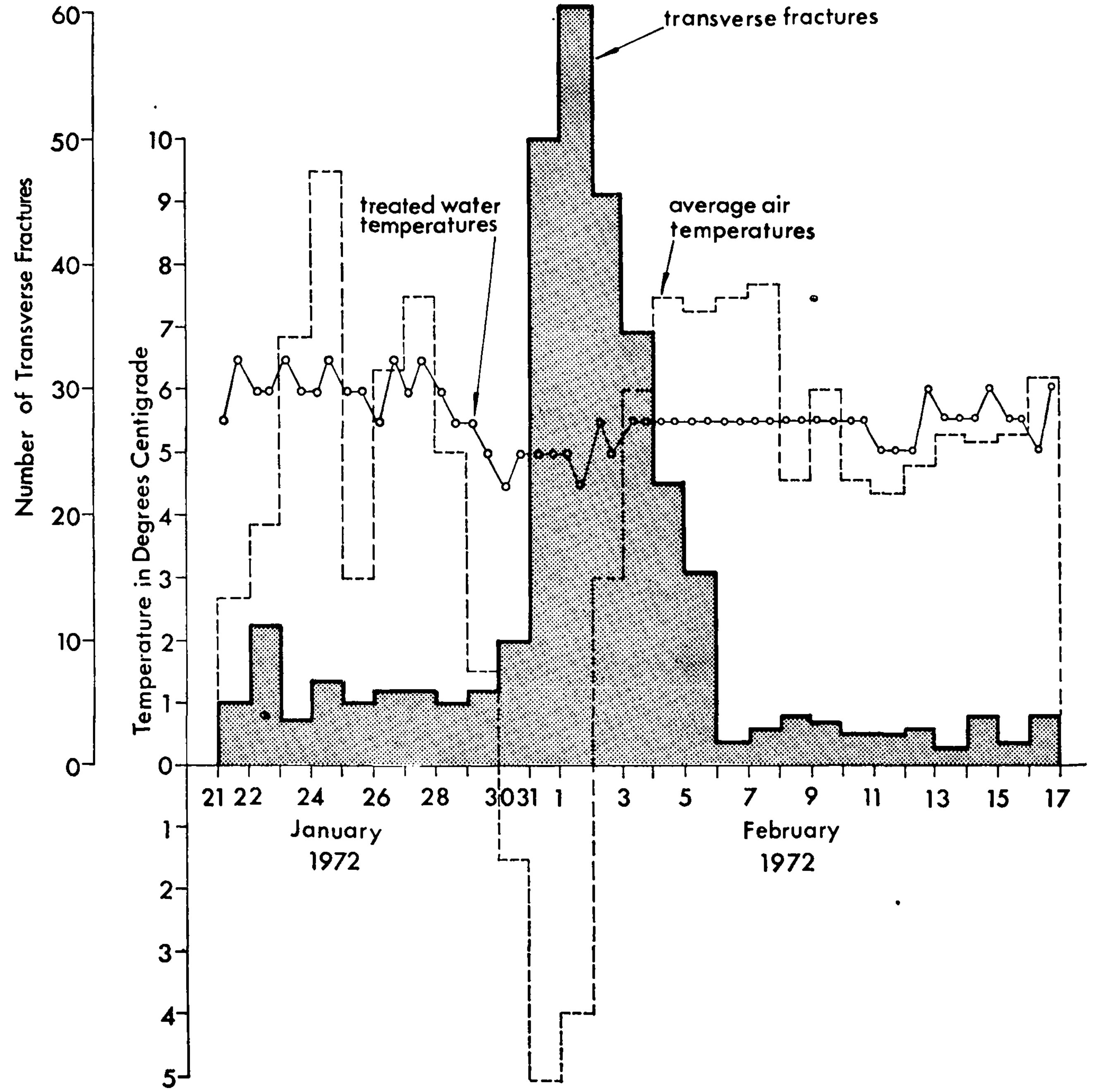

Figure 2—Relationship between Air Temperatures, Treated Water Temperatures and Transverse Fractures London 1972.

air temperatures at London airport, treated water temperatures and the total number of transverse fractures in the Board's area during January and February 1972. As water pipes in the United Kingdom are normally laid about 1 m deep it was not immediately obvious how air temperature could cause fractures of pipes and excite such a rapid response, particularly when the water is only marginally changed in temperature. It has been suggested that changes in air temperature had induced differential ground movement. One of the results of the survey was a recommendation that 100 mm ductile iron or 150 mm grey iron pipes should be the minimum sizes used provided there was adequate protection against corrosion.

7.3 Plastics

Plastics are now used widely for water pipes in many countries throughout the world. Most commonly used are unplasticised Poly Vinyl Chloride (uPVC), Polyethylene (PE), Polypropylene (PP) and Acrylonitrile-Butadiene-Styrene (ABS) with uPVC used in the greater quantity. Although it is appreciated that each thermoplastic material has different characteristics it would be impossible to deal with each separately in this paper and consequently these comments are directed primarily at PVC pipes. Expected design life is calculated to be in excess of 50 years. Impact strength decreases with lowering temperatures and tensile strength decreases with increasing temperatures. As a result careful handling of pipes is required in extremes of temperature. Exposure to sunlight tends to embrittle PVC and ultraviolet light will affect PE pipe. Flow characteristics are good with Hazen-Williams coefficient of 150 and no reduction of flow is expected with age unless slimes are formed on the bore of the pipe.

It has been acknowledged that there has been a continuous improvement of extrusion technology in the manufacture of plastics pipes as a result of the experience that has been built up over the years and failures that were rather more frequent than expected even with a new material have now reached reasonable proportions. The most dramatic type of failure in the past was the so-called spider line failure where as a result of a manufacturing defect pipes split longitudinally, sometimes several together even travelling through solvent joints. Large diameter pipes and integral joints gave trouble and for one reason or another solvent joints leaked, probably due to lack of care from inexperienced jointers.

Interesting figures have been given by a number of Swedish towns in Figure No. 3, illustrating how PVC failures varied with age. Uneven settlement showed up quickly but material faults took a number of years to appear. The relationship between failure of pipelines according to age is shown in Figure No. 4 comparing PVC, grey iron and steel for the same towns.

7.4 Prestressed concrete

Many hundreds of miles of prestressed concrete cylinder and non-cylinder pipes have been laid throughout the world and operate satisfactorily. Failures are few and most of the incidents in recent years have been well investigated and documented so that knowledge of potential problems has increased, leading to improved manufacturing and laying techniques. In Jordan, Regina and Karachi failures were associated with both general and pitting corrosion resulting from action by chloride ions, while at Karachi sulphate ions were also present.

In Australia 250 km of prestressed concrete pipelines have been laid since 1944 and five aqueducts have failed in 11 incidents since 1968, a frequency of 0,006 failures/km/year. The major problem appears to have been the high permeability of the cement mortar coupled with inadequate thickness of cover and the presence of voids next to the steel/concrete interface. In the case of the Geehi River pipeline acidic soil and groundwater aggravated the situation and the corrosion was shown in some cases to result in hydrogen embrittlement of the prestressing cables. Failures on the Morgan-Whyalla No. 2—36 inch (900 km) diameter aqueduct were due to normal wasting of the steel. Chlorides in both ground water and water conveyed of up to 350 mg/l with an average of

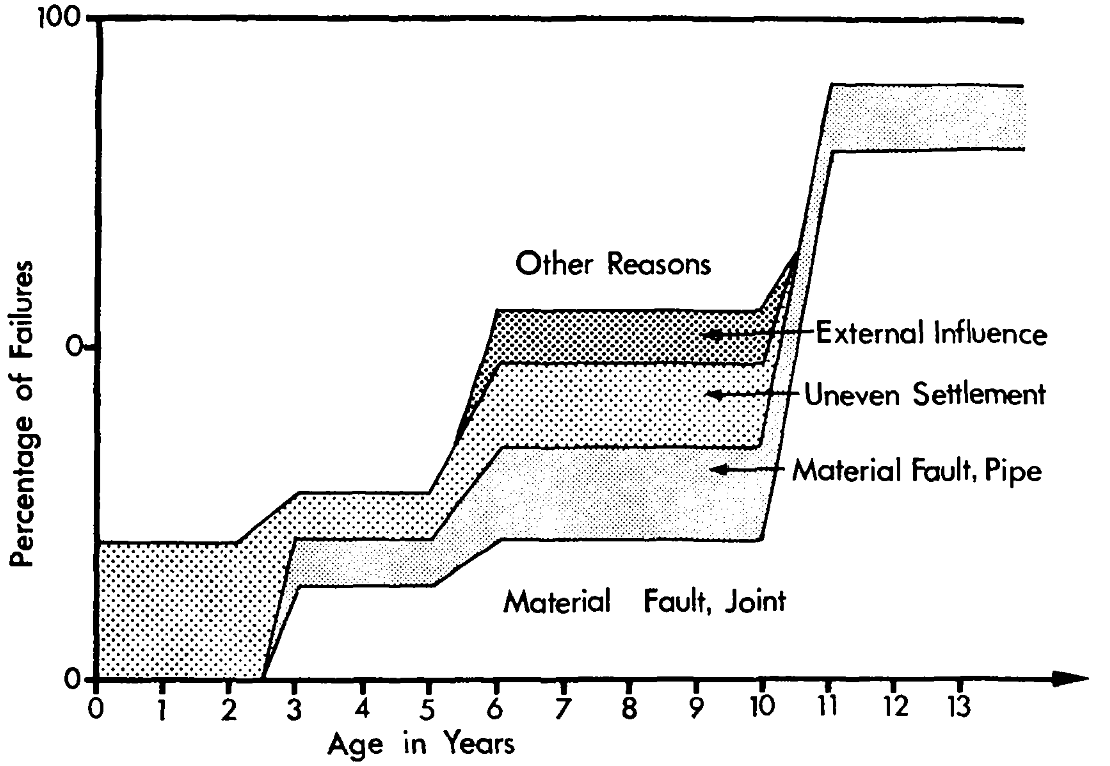

Figure 3—Fault/Age failure in PVC pipes.

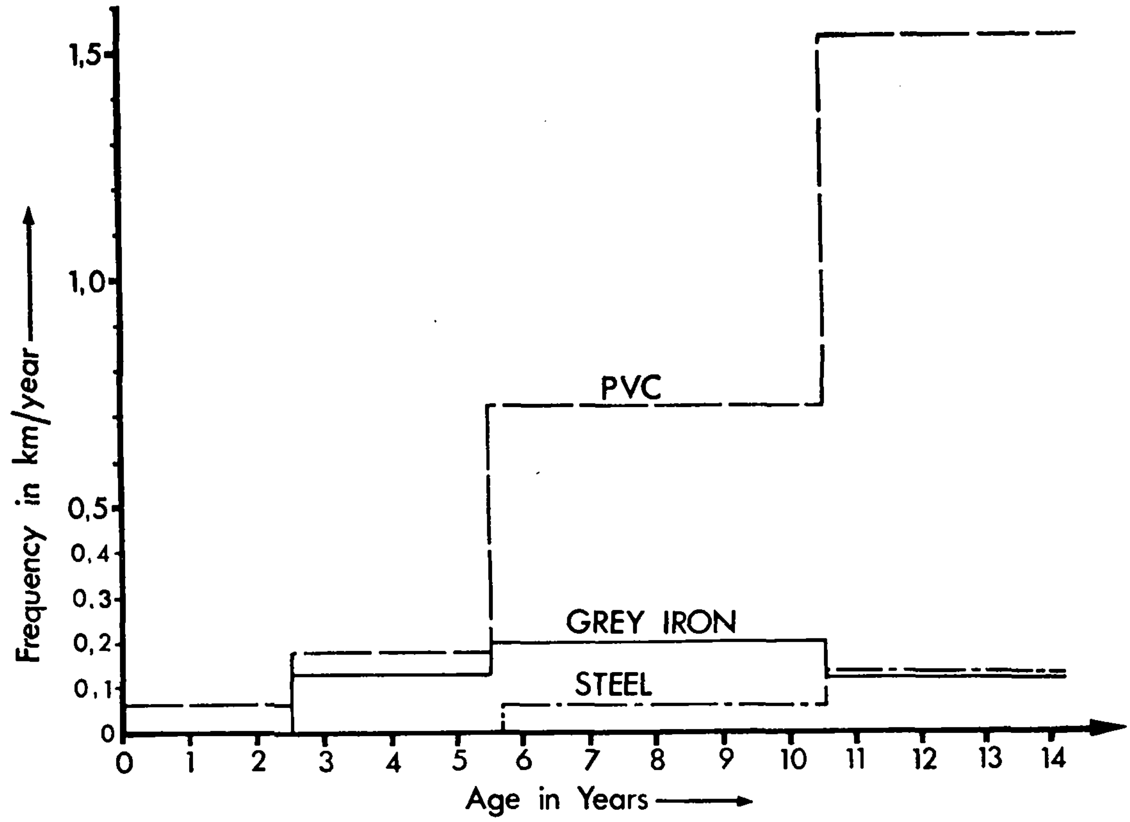

Figure 4—Failure in pipelines according to age.

135 mg/l were contributory factors. Complete replacement of 45 kilometres of Morgan-Whyalla and the repairs and lining of the Geehi pipeline is estimated to cost $ Aust. 15–16 000 000.

Dangers posed by the presence of aggressive ions in soil or groundwater are now fully appreciated and effective methods of cathodic protection or sealing the mortar surface are available, although such safeguards add to the cost of a pipeline. Epoxy tar coatings or similar forms of protection are now applied as standard practice in Australia when the soil conditions are known to be at all aggressive. In the Jordan Valley the mortar cover has been sealed with coal tar epoxy since 1963 and no failures have been reported since that date. Other alternatives worth considering are bitumen paint, coal tar enamel, plastic film wrapping or clay backfill. However, it must be recognised that the mechanics of how a soil can change from non-aggressive to aggressive does not appear to be fully understood and it would be wise to err on the side of caution.

7.5 Asbestos cement

Asbestos cement pipes are prone to beam breakage and care must be taken in preparing the trench. The bedding must be properly prepared and levelled. Hard high spots or stones must not be left projecting. Care must be taken not to drop pipes or to drop heavy objects onto a pipe laid in the trench. According to Anderson the most usual cause of failure is damage by impact sustained at some stage after manufacture, perhaps during off-loading or stringing out. It is recommended that the ends of each pipe are carefully inspected before laying. Correct positioning of the components of the joints are important. With push-in type joints the most common cause of failure is scuffing or tearing of the rubber seal. It has been said that stresses are set up in asbestos cement pipes as they absorb moisture after laying, but this rarely appears to cause failure.

It is important to protect cast iron detachable joints by running solid with bitumen if used in aggressive soils. Many asbestos cement pipelines suffered from this type of attack that did much to influence early users against asbestos cement as a pipe material. The construction and maturing of asbestos cement materials confers a built-in resistance to corrosive effects of soil. As a general guide asbestos cement pipes should be bitumen coated if magnesium sulphate is present greater than 2 000 mg/l as SO_3 in soil water or 0,8 % in soil.

Reports of 80 % of all breakages of asbestos cement pipes due to external factors such as traffic and adjacent excavations are significant. Other cases have been cited of failures due to rapid pressure surge.

7.6 Steel

Perfectly protected steel pipes are claimed to last for ever. This is no doubt true, but no protection can be said to be perfect. Most failures are due to corrosion leading to pinhole leakage or leakage at joints. Some failures have been dramatic. Three major circumferential breaks in a portion of the newly constructed Second Los Angeles Aqueduct have occurred since 1970, in each case due to brittle failure. Faults in design, specification and welding techniques contributed to the breaks.

Steel is used mainly for large diameter pipelines because for distribution systems in towns steel is prone to attack from stray currents and alternative materials have been found more satisfactory. Reports have been received of chemical and electro-chemical attack in aggressive soils due to failure of the other sheathing.

8 External protection of metal pipes

8.1 General

Some protection of iron pipes against possible corrosion in a neutral soil is afforded by dipping or spraying with coal tar or bituminous solution or enamel. Bitumen based sheathing usually reinforced with glass fibre predominates for the protection of steel pipes. For aggressive or very aggressive soils it is becoming more usual to employ inert materials such as asbestos cement, concrete or plastics rather than iron or steel, particularly in the smaller diameters. However, there are occasions when the properties of metal pipes are required in a mildly or sporadically corrosive route, in which case special precautions need to be taken. The cost of external protection must be taken into account when the decision is made to employ metal pipes.

8.2 Cathodic protection

A report in 1973 by the U.K. Institution of Water Engineers Research Panel No. 13 gave details of a comprehensive survey into the extent of cathodic protection. In the U.K. the number of sacrificial anode schemes had grown at a uniform rate for some time, while over the same period impressed current schemes had shown a much greater rate of increase. Difficulties had been experienced with electrical continuity and this highlighted the need for an adequate soil survey to determine the likely corrosive properties of the ground before deciding upon the type of joints to be used. Bonding across push-in type joints when laying a main is not particularly expensive, but the cost of excavation to bond after reinstatement is prohibitive, although for medium and large diameter pipes bonding may be carried out inside the pipe. There was a preference for sacrificial anodes to be used on medium diameter pipelines. It may be that the impressed current method is considered more reliable with a greater degree of control.

More than three quarters of the protected mains were steel, half used flexible joints and these were copper bonded generally using thermalite welding. Some mains with lead joints appeared to achieve satisfactory electrical continuity without bonding. Many ground beds for impressed current schemes used silicon iron as the anode material. For sacrificial anodes magnesium was generally used although zinc had been used in other countries. The spacing of anodes varied from 7 to 500 metres and the expectation of life was between 2 and 100 years with an average of 15 years. It has been calculated that cathodic protection added about 1 % to the capital cost of a pipeline. There appeared to be little difference in cost between the two systems; impressed current was cheaper marginally to install but running costs, although small, were significantly variable.

8.3 Polyethylene sleeving

There is now the considerable experience of many hundreds of kilometres of iron pipe that have been protected successfully by the use of loose polyethylene sleeving. It has been suggested that a saving of 50 % can be achieved over bitumen sheathing. As the wrapping takes place on site it has the advantage of not being subject to damage in transit. Minor damage does not impair the efficiency of protection but repairs are easy. Fixing may be by either PVC tape or string. Material specification standards exist in the U.S.A. and West Germany. In the U.K. the usual recommendation is for a natural polyethylene film with a single nominal thickness of 1 000 gauge i.e. 250×10^{-3} mm, a melt-flow index of 10 or less to BS 2782 and a density of 915 to 925 mg/ml. In France and the U.S.A. the thickness is specified at 200×10^{-3} mm, while in West Germany the limits are 200 to 250×10^{-3}. In Newcastle, England, there are pockets of highly aggressive soil capable of eating through iron pipes in less than two years. Since 1966 all iron pipes have been wrapped in polyethylene sheaths and to date no failure of any main so protected has been reported. The cost for a 200 mm pipe is £100 per kilometre run. It is interesting to note that in that area the water company uses blue polyethylene sleeving, while the gas board uses yellow as an aid to identification.

8.4 Zinc based coating

Metallised zinc plus a coal-tar varnish has been used on small diameter grey iron pipes in France since 1959 and for ductile since 1962. The protection is different from that given by the galvanic action of zinc in galvanised steel. The layer of zinc is believed to combine gradually with the ground waters in contact with the pipe to form insoluble mineral products of carbonate etc. which then being impermeable protect the pipe from further attack. This chemical reaction raises the pH locally slowing up the action of sulphate reducing bacteria. According to Brooks no failures have been reported over 15 years in a production of 70 000 km of pipes protected by zinc based coating.

9 Internal protection of metal pipes

Internal corrosion of metal pipes is less severe in terms of failure than external attack and is directly related to the quality of the water being carried. In the author's opinion the key to this problem is likely to be the adequate and proper control of the treatment given to the water before transmission whenever that is possible. Although internal corrosion may be less serious from the point of view of failure, it is more important as regards operational costs. Corrosion internally results in tuberculation formed from the products of corrosion and the additional roughness will lead to a reduced C value in the Hazen-Williams pipe friction formula. This means a reduced flow for the same headloss, or, if pumping head can be increased, then increased power consumption for the same flow. In distribution systems this will usually result in complaints of dirty or red water. If the normal protection of either hot coal tar dipping or bituminous paint is not satisfactory then it is usual to apply a centrifugal spun lining of dense cement. In some cases this will be a sulphate resisting cement. Cement linings must be cured under controlled conditions or by the application of a seal coat of bituminous material while still moist.

10 Cleaning pipelines

After laying, a pipeline to carry potable water will require cleaning. Provided that care and adequate supervision during construction has left the inside of the pipes reasonably free from foreign matter, then flushing followed by adequate sterilisation usually 50 mg/l for at least 24 hours will possibly give satisfactory bacteriological results after draining and refilling. More usual practice in the U.K. is to clean the main by passing a relatively soft grade swab made of expanded polyester foam throughout the length using the techniques developed by the former Water Research Association now the Water Research Centre, Medmenham.

Nowadays metal pipelines are protected against internal corrosion but many thousands of kilometres of cast iron pipe have been laid in the past without adequate protection. In time cleaning is required to restore the hydraulic flow rate and to obviate dirty water complaints. Debris and loose material may be removed by the swabbing techniques using hard grade pigs driven through the main by water pressure. Serious encrustation requires boring and flailing, drag scraping or hydraulic pressure cleaning. Recently in Birmingham, England a hydraulic jet blaster working at 20×10^{3} kN/m^{2} has been successfully used in small diameter mains. Up to 200 metres can be cleaned from a single excavation using a retro-jet nozzle which keeps the head concentric within the main and drives it forward. For larger sizes boring and scraping are more usual. Problems are primarily in negotiating awkward bends, valves, connections and the removal of debris. Scraped mains become encrusted again more rapidly than before and it is essential to provide a new lining or to accept a regular programme of scraping. As a result specialist contractors are usually employed to provide a complete scraping and relining service.

11 On site lining of pipes

Adequate cleaning and removal of debris is essential before lining. A number of patented processes are available including sprayed bitumen coatings and the electrophoretic deposition of bitumen. The widespread practice of applying cement mortar lining after cleaning varies with the diameter of the pipe. For small diameter pipes, particularly where there are connections, the cement is discharged centrifugally with a resultant dense but rough finish which is reputed to be less significant in terms of C value than would appear possible. With medium and large diameter pipes the cement is sprayed centrifugally and then finished to a smooth surface with rotary trowels. It is suggested that the thickness of the lining should be not less than 4 mm in small diameter pipes, 6 mm in medium, but 10 mm thickness for pipes above 450 mm diameter. It has proved difficult to obtain reliably comparable figures between the cost of relining and relaying water mains but a rough guide would be a saving of 50–70%.

In the U.S.A. and Africa the epoxy lining of steel pipelines on site have been reported as being completed successfully. The pipes are cleaned chemically leaving an etched surface which is then neutralised, phosphated, and coated at least twice with epoxy resin to a dry film thickness of at least 150 microns. An instance of damage to amide-cured epoxy linings has been reported where a certain amount of damage has been associated with making good field welds. Apparently it is difficult to avoid damage to thin coatings. A high standard of in situ cleaning is essential and may be the cause of certain failures at field welds.

12 Rubber joint seals

In many modern flexible joints the seal to prevent water leakage is formed by means of compressing a rubber ring. Leeflange reported in 1963 that in the Netherlands out of several hundred rings examined it had been found that over half had deteriorated on the surface in contact with potable water, but only a small percentage showed change on the soil side. Since that date wasting of rubbers from pipes have been reported from Australia, New Zealand and elsewhere. It seems likely that a change in rubber technology in the early 1960's made such rings more prone to attack by bacteria. However, the problem involves much more than biodeterioration; chemical attack and poor joint design may be contributory factors.

It has been suggested that the phenomenon may be more prevalent throughout the world than is generally believed by rubber and pipe manufacturers. The evidence so far suggests that the water supply industry should be on the alert and aware of the problem so that instances may be recorded which, together with current research investigations, will produce the evidence on which a decision may be based as to whether changes are necessary. With the present widespread use of rubber joint rings based on the premise that the life of the rubber will be compatible with the life of the pipe, it cannot be denied that the subject is not important, but how important will depend upon the result of the present investigations.

13 Glass reinforced plastic pipes

During the last few years reinforced plastics have begun to be used for large diameter pipelines. Advantages claimed are high strength to weight ratio, good resistance to corrosion, low resistance to flow and non-tainting of potable waters. There are two basic methods of manufacture, each using resin impregnated glass fibre filaments either wound in a controlled helix or placed in circumferential and longitudinal directions to give the required hoop and beam strength.

GRP pipe has the fundamental advantage over homogeneous materials in that the pipe can be made to suit exactly any particular duty. It is classed as a flexible conduit and care must be taken to ensure proper backfill and vertical deflection of the pipe which should not exceed 5% of the pipe diameter after proper reinstatement. Internal vacuum can introduce buckling and with shallow cover the possibility of vertical heaving. According to Greatorex and Chambers, GRP/RPM pipes are designed on the assumption that the internal pressure induces uniaxial hoop tensile stress which is taken solely by the hoop glass filaments.

In 1971 a pilot batch of twenty-five 750 mm diameter GRP/RPM pipes manufactured at Stanton and designed to operate at 12 bar internal pressure were laid experimentally at Bristol, England with different conditions of bedding or consolidation. Strain gauges were fitted and variations in vertical and horizontal dimensions measured. Some of the sidefill consisted of lumps of new wet clay that would never have been used under normal circumstances and at these points the pipe deflection was 14,7% compared with the more usual figure of up to 5% elsewhere. After completion of the tests the pipeline was connected into the normal distribution system and continues to perform satisfactorily.

Design to a stipulated wall section has tended to limit competitive pipe sizes normally to those over 600 mm diameter with current pressure class limitation of about 4 or 6 bar. Smooth clean internal bores indicate low hydraulic friction and Hazen-Williams coefficients of 145 to 155 are reported. It is claimed that there is no leaching of noxious substances or fostering of yeasts, fungi and bacterial growths that might impair the quality of transported water and the pipes are virtually inert to all except the most acidic soils. In common with most plastic materials GRP pipes are time dependant and long term tests have been undertaken to ensure that adequate strength will be maintained for a service life presently anticipated as being up to 50 years. Reports have been received of mixed success in operation of GRP in the U.S.A., U.K., West Germany and the Middle East.

Appendix A
Acknowledgements

The following water authorities, associations, research organisations and companies have provided a great deal of data and information and their assistance is acknowledged with thanks.

Anglian Water Authority, England
Antwerp Waterworks, Belgium
Atlanta Bureau of Water, Georgia, U.S.A.
Austrian National Committee IWSA, Vienna
Barcelona Water Company, Spain
Bristol Waterworks Company, England

British Steel Corporation, England
Brussels Water Company, Belgium
City University, London, England
Concrete Pipe Association, England
Construction Industry Research and Information Association, England
Copenhagen Water Supply, Denmark
Dallas Water Utilities, Texas, U.S.A.
DVGW, Gas and Waterworks Association, Germany
General Descaling Co. Ltd, England
Gentofte Water Supply, Denmark
Gothenburg Water and Sewage Works, Sweden
Groningen Waterworks, Netherlands
Humes Ltd, Melbourne, Australia
Hydraulic Research Station, England
Indianapolis Water Company, Indiana, U.S.A.
Milwaukee Water Works, Wisconsin, U.S.A.
Newcastle and Gateshead Water Company, England
North West Water Authority, England
Northumbrian Water Authority, England
Pipelines Industries Guild, England
Pitometer Associates, New York, U.S.A.
Rand Water Board, South Africa
Sentab Pressure Pipe Consortium, Sweden
Severn-Trent Water Authority, England
South West Water Authority, England
TAC Constructions Materials Ltd, England
Tate Pipe Lining Processes Ltd, England
Thames Water Authority, England
Transport & Road Research Laboratory, England
Warsaw Polytechnic, Poland
Warsaw Water Supply, Poland
Water Research Centre, England
Welsh National Water Development Authority, Wales
Yorkshire Water Authority, England

Appendix B

Standards and codes of practice

International Standards

ISO/R 13	Cast iron pipes for pressure main lines
ISO/R 160	Asbestos cement pressure pipes
ISO/R 559	Steel pipes for gas, water and sewage
ISO/R 1165	Plastic pipes for transport of fluids—uPVC pipes
ISO/R 2531	Ductile iron pipes for pressure pipelines
ISO/R 2785	Guide to the selection of asbestos cement pipes subject to external loads with or without internal pressure

United Kingdom

BS 486	Asbestos cement pressure pipes
BS 3505	uPVC pipe for cold water services
BS 3601	Steel pipes and tubes for pressure purposes
BS 4622	Grey iron pipes and fittings
BS 4625	Prestressed concrete pressure pipes
BS 4772	Ductile iron pipes and fittings
CP 310	Water supply
CP 312	Plastics pipework (thermoplastics material)
CP 2010	Part 1 Installations of pipelines in land
	Part 2 Design and construction of steel pipelines
	Part 3 Design and construction of iron pipelines
	Part 4 Design and construction of asbestos cement pipelines
	Part 5 Design and construction of prestressed concrete pipelines

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Résumé

Ce rapport a été rédigé grâce à l'aide d'un grand nombre de services d'eau, sociétés et personnes. Il couvre les tuyaux et conduites allant de 100 à 1 000 mm de diamètre interne.

Etude des conduites

Les conduites en amiante-ciment, la fonte grise et le béton précontraint peuvent être considérées comme des cylindres rigides dont l'aptitude à supporter une charge extérieure est en relation directe avec la rigidité annulaire de la structure, en tenant compte du matériau de remblaiement, de sa compaction et de la forme de la tranchée. Un choix personnel de normes pour le matériau excavé et choisi est donné.

Les tuyaux en fonte ductile, acier et plastique ont une faible rigidité inhérente et s'affaissent sous les charges verticales. L'emploi de ces matériaux flexibles signifie que l'on doit prendre en considération l'effet de support du sol. Les charges externes exigent l'analyse de la déflexion, de la courbure, de la compression et du flambage. Il est nécessaire de vérifier les corrosions d'efforts, les forces cycliques et les propriétés d'impact.

La détermination des conditions physiques et chimiques du sol est importante. Le développement des joints flexibles est retracé depuis les joints rigides en passant par les joints semi-rigides. Référence est faite aux choix disponibles dans la sélection des formules pour déterminer la perte de charge hydraulique des conduites.

Expériences d'emploi des divers materiaux

On a récemment exprimé des inquitudes pour les problèmes de goût et de toxicité pouvant résulter de l'emploi de certains matériaux. Il existe maintenant des tests pour déterminer l'aptitude à donner à l'eau goût, odeur, couleur, turbidité, ou toxicité ou à favoriser un développement microbien.

Une enquête récente a montré que la fonte grise avait la plus longue espérance de vie, suivie par le béton précontraint. Des réserves ont été exprimées sur l'amiante ciment et l'acier posés en sols corrosifs, tandis que la fonte ductile et les plastiques, étant des matériaux relativement nouveaux, étaient regardés avec une certaine suspicion.

En Grande-Bretagne, la fonte ductile remplace la fonte grise, particulièrement dans les petites tailles. Pour les conduites principales de grande taille. on utilise des tuyaux d'acier revêtus de bitume, à joints soudés. On utilise des tuyaux en amiante-ciment en sols agressifs malgré les doutes sur leur espérance de vie. Le PVC est populaire en régions rurales.

L'expérience mondiale varie quand on compare les différents matériaux de conduites, ce qui n'est pas surprenant car les conditions du sol, les modes de pose, la profondeur, le trafic et les températures varient d'un site à l'autre.

Dans un rapport de Londres sur les conduites en fonte grise, le principal type de fracture était la rupture transversale. On ne pense pas que l'âge soit une cause primordiale de rupture. Dans les grands diamètres, le nombre de fractures diminuait et les fentes logitudinales devenaient prévalentes. Il semble qu'il y ait une relation entre la température de l'air et les fractures, peut-être en raison de mouvements différentiels du sol.

Il est difficile de dire des généralités sur les conduites en plastique en raison des caractéristiques des divers matériaux. L'expérience passée a été décevante en ce que les ruptures ont été plus fréquentes que prévues, même avec un matériau neuf. L'amélioration des techniques d'extrusion s'est réalisée avec le temps et les conduites en plastique sont maintenant largement utilisées en de nombreux pays.

Les ruptures avec le béton précontraint ont été relativement rares et la plupart des incidents ont été complètement examinés et ont fait l'objet de rapports. Le danger posé par la présence d'ions agressifs dans le sol est maintenant bien apprécié, encore que le mécanisme de la transformation de non-agressif à agressif puisse ne pas être complètement élucidé.

Un pose généralement les conduites en amiante-ciment dans les sols corrosifs. C'est un matériau cassant, dont la manipulation doit être faite avec soin, mais s'il est convenablement posé et non soumis à des influences extérieures, il semble très satisfaisant.

L'acier durera pour toujours s'il est parfaitement protégé, mais c'est un idéal. En pratique des ruptures mineures sont apparues en raison de défauts dans le revêtement extérieur.

Un protège les tuyaux métalliques extérieurement en les trempant dans le goudront ou le bitume, ou en pulvérisant ces produits, en les revêtant, en les plaçant dans un manchon de polythène, en pulvérisant du zinc métallique, ou en installant des anodes réactives ou par soutirage de courant. La protection intérieure, quand on l'utilise, est généralement faite de ciment centrifugé. Le nettoyage des conduites anciennes se fait par cylindres de plastique ou par grattage. Il est nécessaire, après grattage, de refaire le revêtement intérieur pour éviter la réincrustation.

Des préoccupations ont été exprimées sur la tenue des joints en caoutchouc et des recherches sont nécessaires à ce sujet.

L'introduction de plastiques renforcés à la fibre de verre est mentionnée, mais l'expérience actuelle est limitée.

Pressure control in distribution systems

by Dr. Chr. Laske

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1 Introduction

The following paper is merely a contribution to an ambitious subject. Experiences are drawn from operational aspects of the water supply to the large city of Stuttgart which has differences in elevation of more than 300 m. In normal circumstances, 90% of the city's water supply is imported in two long-distance supply lines, whose delivery points are on the highest peripheral locations of the supply area, with the result that a good deal of the water has to be reduced step by step to the pressure of the respective supply zone. The city's own waterworks, which are in lower-lying locations, serve to cover peak consumption, the supply of several large-scale users, and as a reserve supply. This water has to be pumped to the higher-lying consumption points.

In treating this subject, both technical and economic aspects of the supply pressure, as well as legal ones are considered.

2 Supply pressure

The overriding principle of any water supply is to guarantee the drinking water requirements at every location within the supply area and to ensure that it is at all times of sufficient quantity, best possible quality and adequate supply pressure (4).

By supply pressure is meant the static head p/γ for the consumer x with the geodetic height z of the highest consumption point. To transport the required amount of fluid to the consumer, the resistance caused by fluid friction has to be overcome, i.e. hydraulic energy is lost, which is denoted as loss of head. The energy balance is represented by the well-known equation

$$p/\gamma + z + h_v = H$$

(Velocity head disregarded)

h = Difference in height between two points
z = Height related to sea level
p = Normal force exerted by water on the surface unit
γ = Specific gravity
h_v = Pipe friction loss

A water supply consists of a collection of elemental systems; for treatment, boosting, pressure reduction, transport, storage, measuring and control. These are joined to supply the required amount of water at the necessary pressure.

The meaning of pressure control in this respect is the influence on and control of all elements which guarantee the supply: irrespective of the height of the consumption point.

The necessary supply pressure is not, however, accurately defined in Germany. It should also be taken into account that a pressure once set cannot be kept at the desired level without control.

There are, however, points of reference for the pressure limits:

1. Disregarding negative pressures (vacuums), which should not appear in any drinking water supply, because of the danger of germs and other extraneous materials penetrating into the pipe network, the lower pressure range is defined by the "minimum flow pressure" for the highest outlet point of a building. The minimum flow pressure amounts to 0,1 bar (tap)–2,5 bar (gas operated flow heater).
2. The highest permissible operating pressure is defined as follows in a German standard (DIN 1988): "Pipe and ancillaries should be designed for at least 10 bar internal pressure, insofar as higher operating pressures do not require larger dimensions."

Exceptions to this are boilers, which are frequently only designed for an internal pressure of 6 bar and which then mostly require fittings to reduce a higher pressure to 6 bar.

Whilst an operating pressure in the supply network of up to 10 bar is completely permissible, economic reasons and noise protection dictate against the supply of water at a pressure in excess of 6 bar. Generally, pressures between 2 and 8 bar are the rule in the supply network.

3 Pressure control

The following considerations are features of pressure control:

1. Pressure control is the requirement for *technical operation.*
2. Pressure control enables an easy to view, *economic operation.*
3. Pressure control in the sense of the *reduction of excess supply pressures* has advantages for:
   —the life of pipelines and connections,
   —the protection of consumer devices and installations,
   —the reduction of losses from leakages.
   and water is consumed more sparingly.
4. Pressure control is evidence of the correct and regular operation of the water supply, e.g. also from a legal point of view.

3.1 Pressure control in the technical operation

The design basis for much water supply equipment is the fluctuating demand for water. The flow or delivery rate results from this. The flow is significantly determined by consumption. Consumption, on the other hand, shows marked alterations, despite statistical superimposition of all individual consumptions, especially when comparing shorter intervals of time. To be able to cover peak consumption at all times, it is, however, not necessary or economic to construct the system to the absolute peak. The interval of time to be used for design is dependent on the type of system and topographic or other conditions [18]. The capacity of the various parts of the system is hence better exploited and economy increased. Occasional slight, short-term pressure reductions are quite permissible.

The ratio of flow and pressure drop, using the annual duration curve for the peak year 1964 as an example, as well as the temporary pressure fluctuations, which are always present in the network, are presented in Figs. 1 and 2.

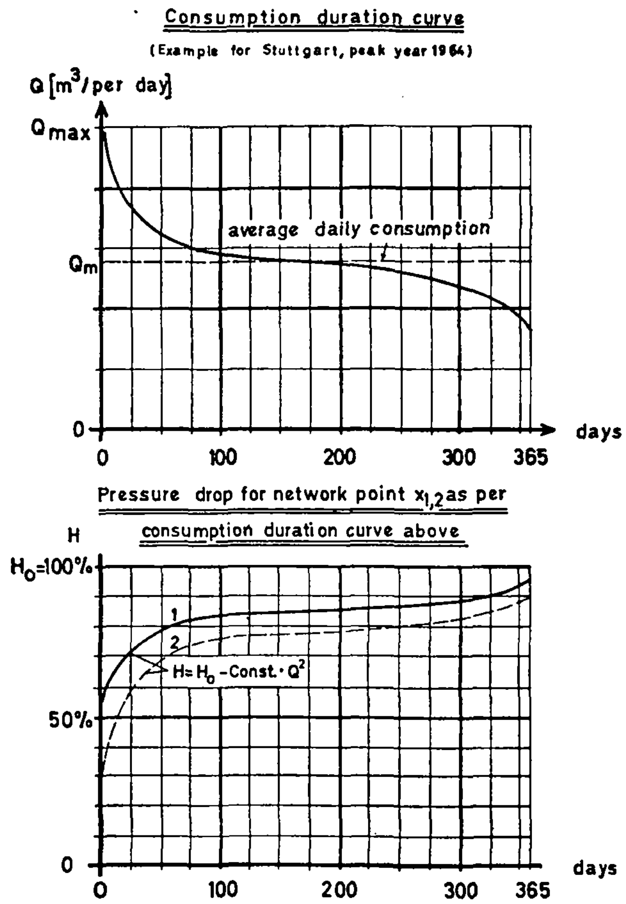


Figure 1

Large pressure fluctuations can have the following causes:

—Start up or shutdown of connection, treating and delivery systems
—Filling and emptying of lines or flushing
—Vibration of control or cut-off devices (e.g. float valves)
—Pulsations in the pump pressure lines
—Incorrect layout of systems' parts
—Fracturing of drive or control elements
—Consumption fluctuations
—Drawing off water for fire-fighting
—Pipe fractures
—Accumulations of air in lines (high points)
—Power failure

3.2 Pressure changing systems

All equipment, with which or via which a specified pressure is set or maintained, is pressure changing equipment, as for example

—Pumps and turbines
—Control valves, such as cylindrical piston valves, pressure relief valves
—Other fittings and
—Tanks

Problems with pressure reducing and booster systems, for operating conditions of larger lines and in the network respectively are to be touched upon in the following.

3.2.1 Pressure relief in feed lines

These are defined as lines with an internal diameter of approx. 600 mm and more, which feed into a tank. With this procedure, a considerable energy potential has very often to be reduced.

Both turbines and special control elements are used to transform pressure energy into velocity energy. An economic assessment should decide which solution is preferable in individual cases. The cylindrical piston valve to a special design is discussed as an example of a control element.

3.2.1.1 Cylindrical piston valves

Fittings which are to be used as a control element should meet the following requirements:

(a) Good control behaviour, i.e. the opening and closing procedures in the fittings are tuned to the pipeline, the respective flow rate and the opening or closing time such that water hammer only occurs within permissible limits. The shortest closing time can only be reached with constant lag for a specified water hammer increase, i.e. by steady velocity change in the pipeline. This condition should only be produced in the control element itself. In the case of supply lines with greater pipe friction losses use is made of an adjustment with a non-linear control unit opening function, i.e. a time or path dependent function.

(b) continuous operation over a broad opening range (stroke ratio s/s_0) without cavitation and with the least possible vibration.

(c) Trickle-tight seal.

While the conditions of (a) and (c) can be met most satisfactorily with a cylindrical piston valve (also called cylindrical piston gate) with a normal plunger, and (a) furthermore with a stepped law of closure, (b) presents problems if a relatively low local counter pressure is present with a tank of max. 4–5 m water level. The evaporation pressure of the liquid is partly attained as a result of the velocity head. Bursting of the vapour bubbles is accompanied by considerable noise generation. Pulsating of the interference leads to impermissible vibrations in the various parts of the system. Material destruction is the consequence.

These phenomena, which can considerably hinder the operation, can be reduced under certain conditions using a fitting provided with a flow-promoting shut-off device, and which for instance has orifice plates [14].

At very high pressure energies orifice plate elements are also used independently of a fitting, thus becoming non-controllable energy converters. Several elements such as these can be connected in series to form so-called cascade throttles.

With the cylindrical piston valve with orifice plates a cylinder with a certain number of concentrically arranged bores is mounted on the axially displaceable plunger (see Fig. 3).

In the closed position the piston sits on the sealing ledge. When opening, the cylinder first opens up, exposing in each case a discharge section corresponding to the sum of the opened orifice sections.

The liquid flowing from the upstream pressure side passes through the bores in a large number of thin jets, which disperse from outer to inner into the cylinder. The jets, which hit each other concentrically at great velocity, are decelerated. In doing so, part of the velocity energy is converted into thermal energy, deformation energy and sonic energy.

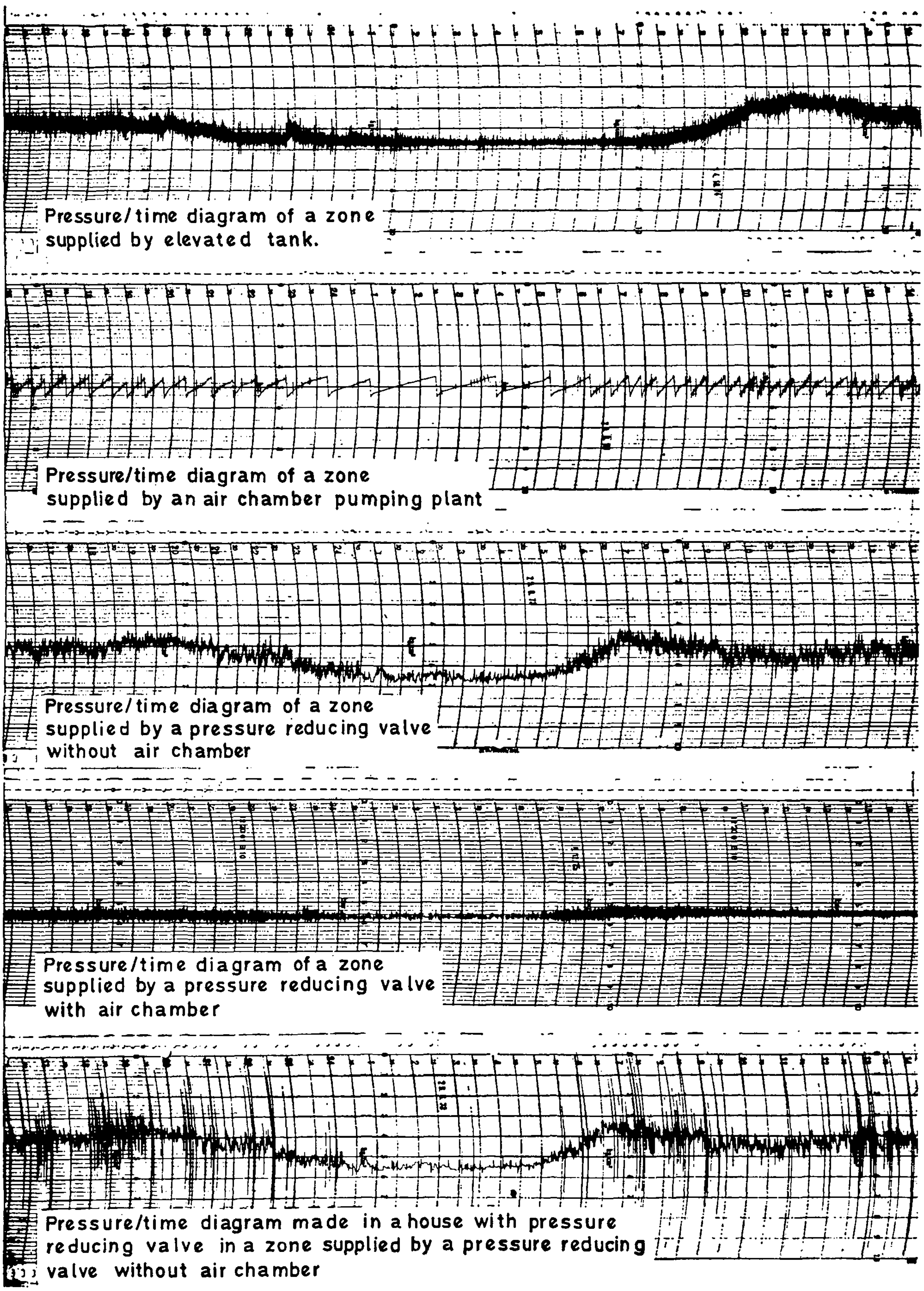

Pressure/time diagram of a zone supplied by elevated tank.
Pressure/time diagram of a zone supplied by an air chamber pumping plant
Pressure/time diagram of a zone supplied by a pressure reducing valve without air chamber
Pressure/time diagram of a zone supplied by a pressure reducing valve with air chamber
Pressure/time diagram made in a house with pressure reducing valve in a zone supplied by a pressure reducing valve without air chamber

Figure 2

Cylindrical piston valve

Cylindrical piston valve with orifice plate

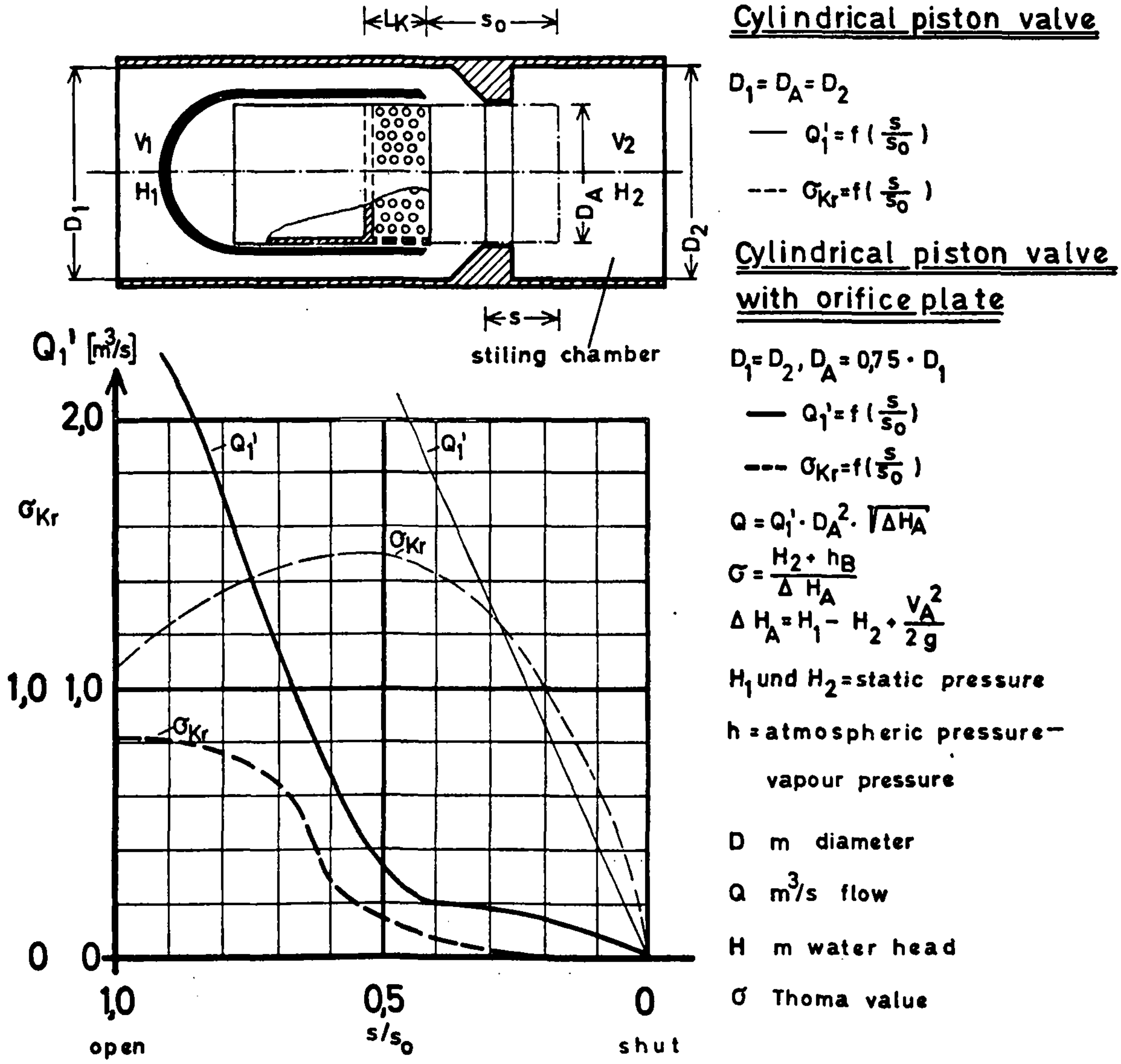


Figure 3

If the chosen gate is large enough, the whole operating quantity can be passed through the orifice plate cylinder. Otherwise, the cylinder has to be opened far enough for the normal cylindrical cross-section of the valve to be opened via the last part of the flow path.

Which size of valve is to be preferred in the planning should be based on the hydraulic system data and is not least a question of costs.

Should a permanent operation of the valve be necessary or possible also in the operating ranges with partial opening, the question of whether cavitation might in fact occur in the operating range should be clarified. In this respect the characteristic value σ (Thoma value) should be considered. The continuous operating range is defined by the cavitation factor σ.

The σ value for the system is defined by:

$$\sigma_{system} = \frac{H_2 + h_B}{H_1 - H_2 + \frac{C_A^2}{2g}} \geqslant \sigma_{perm.}$$

The condition that no cavitation occurs at the point of least pressure is as follows:

$$H_2 \geqslant \sigma_{perm.} \times \Delta H_A - h_B$$

$$\Delta H_A = H_1 - H_2 + \frac{C_A^2}{2g}$$

H_1, H_2 = Static pressure upstream of valve or end of stilling basin, mWH

C, C_A = Velocity, end of orifice plate cylinder, m/s

σ = Thoma value

h_B = Air pressure minus evaporation pressure, mWH

g = Acceleration due to gravity m/s²

In Fig. 3, the flow ratio Q_1' depending on the stroke ratio s/s_0 for $\Delta H_A = 1$ m, $D_A = 1$ m is drawn both for a valve in normal design and for one with orifice plate cylinder. The most favourable path of the limit curve $\sigma_{critical}$ can be seen for the cylindrical piston valve with orifice plate.

"Goldberg" plant (Diagramatic view Fig 4)

The Goldberg tank with only 490 m^3 content, serving as pressure controller and corresponding tank to a larger tank six kilometres away, supplies a large suburb of a town with approx. 125 000 inhabitants. A cylindrical piston valve (nominal dia. 300 mm) with orifice plate operates as a reducing valve in the inlet to the tank, as described above. A second valve serves as a stand-by or operates alternately. The max. flow rate for the control fittings was specified as $Q_{max} = 0{,}834$ m^3/s with a static pressure difference of $H_{geo} = 100$ m and a max. pressure loss in the 2 km long feed line (nominal dia. 800/650 mm) of $h_v = 8{,}0$ m water head. The outlet goes into an ante-chamber with a water level of 4 m. The latter is connected to the main chamber by an overflow. The valve is controlled according to the water level in the main chamber, which should neither empty nor overflow. The closing time is fixed at 50 secs, with a max. permissible water hammer of $\Delta H_A = \pm 15$ m water head.

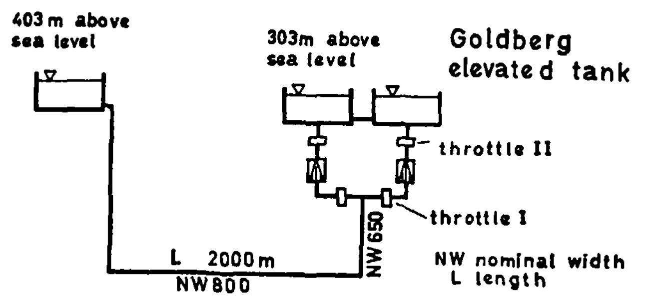


Figure 4

With a static counter pressure of only 4 m water head, relatively large ΔH_A values result, with the result that with correspondingly small σ values for the system, the σ value falls below $\sigma_{critical}$ at a certain partial opening range s/s_0. As the pressure reducer must operate cavitation-free in all operating ranges, it has been constructed as a two-stage design with upstream and downstream throttles. It has been shown that after start-up the system's σ value should be improved, e.g. by installation of a fixed orifice plate.

3.2.1.2 Turbines

Turbines can also operate as pressure reducers. The reduction of the pressure energy potential can in this case be used economically. Turbines, however, require more investment and higher operating costs. Moreoever, their control range is smaller.

The real task for turbines in a water supply system is pressure reduction. The electrical energy produced is in this sense a "waste product". Even so a waterworks' turbine cannot be operated with other than normal economy.

When specifying the size and number of turbines to be used, the starting point is the development of the water requirement over a certain period of time. The average consumptions $Q_{m\,1,2}$ ($Q_{m\,2}$ after 20 years) and the summer peaks resulting from them $Q_{max\,1,2}$ are considered (where $Q_{m\,1}$ and $Q_{max\,1}$ are current mean and maximum consumptions, and suffix 2 refers to a time 20 years later).

The result of an economic comparison derived therefrom is that a turbine, despite less use and lower efficiency, should be designed as per $Q_{max\,2}$ (see Fig. 5, and for profit and cost comparison curve Fig. 8).

A turbine fitted as a reserve unit does not operate economically. The reserve unit should therefore be designed as a control fitting. In Fig. 6, units with equal capacity are compared in each case ($Q_{max\,2}$ from Fig. 5).

Remote monitoring and control of turbines requires additional investment, the capital costs of which in fact correspond to the saving on personnel costs involved. Turbines should therefore be used as far as possible in systems in which operating personnel are used already (see Fig. 7 in comparison to Fig. 6),

Control fittings as a reserve unit cannot be dispensed with under any circumstances, as shutdown times for repairs cannot be disregarded. In a great number of applications, control fittings will remain the only suitable unit.

Profitability comparison between turbine systems with differing design

Turbines designed for	$Q_{m\,1}$ 417 l/s	$Q_{m\,2}$ 624 l/s	$Q_{max\,1}$ 742 l/s	$Q_{max\,2}$ 1 125 l/s
Investment costs per turbine system	DM 500 000	DM 600 000	DM 650 000	DM 700 000
Interest plus 12% depreciation per year	60 000	72 000	78 000	84 000
Operating costs/year	18 000	18 000	20 000	20 000
Maintenance costs/year	2 500	2 500	3 000	3 000
Total costs/year	80 500	92 500	101 000	107 000
Credit from power sales	209 320	256 492	268 132	283 758
Profit/year	128 820	163 992	167 132	176 758

Invoicing of electricity costs for day tariff = 0,09 DM/kwh
for night tariff = 0,05 DM/kwh

Night tariff applies between 21.00 and 06.00

Fig. 5

Profitability comparisons between turbine systems and pressure reducing systems

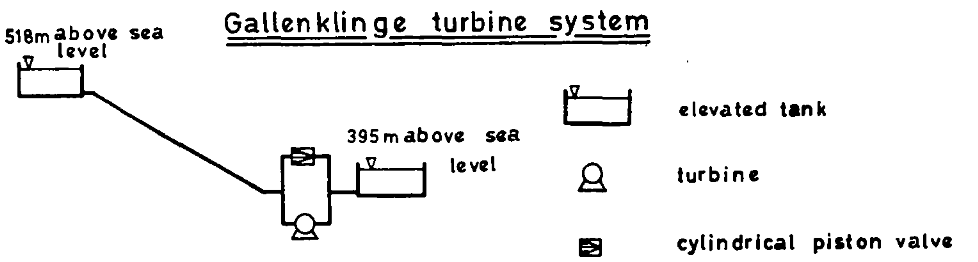
	Waterworks operation			Power station operation
	2 turbines	1 turbine+ 1 pressure reducer	2 pressure reducers	1 turbine
Investment costs	DM 1 400 000	DM 700 000	DM 250 000	DM 650 000
Interest+12% depreciation	168 000	84 000	30 000	78 000
Operating costs/year	40 000	25 000	15 000	20 000
Maintenance costs/year	6 000	4 000	2 000	3 000
Total costs/year	214 000	113 000	47 000	101 000
Credit from power sales	160 000	160 000	—	415 000
Residual expenditures	54 000	—	47 000	—
Profit/year	—	47 000	—	314 000

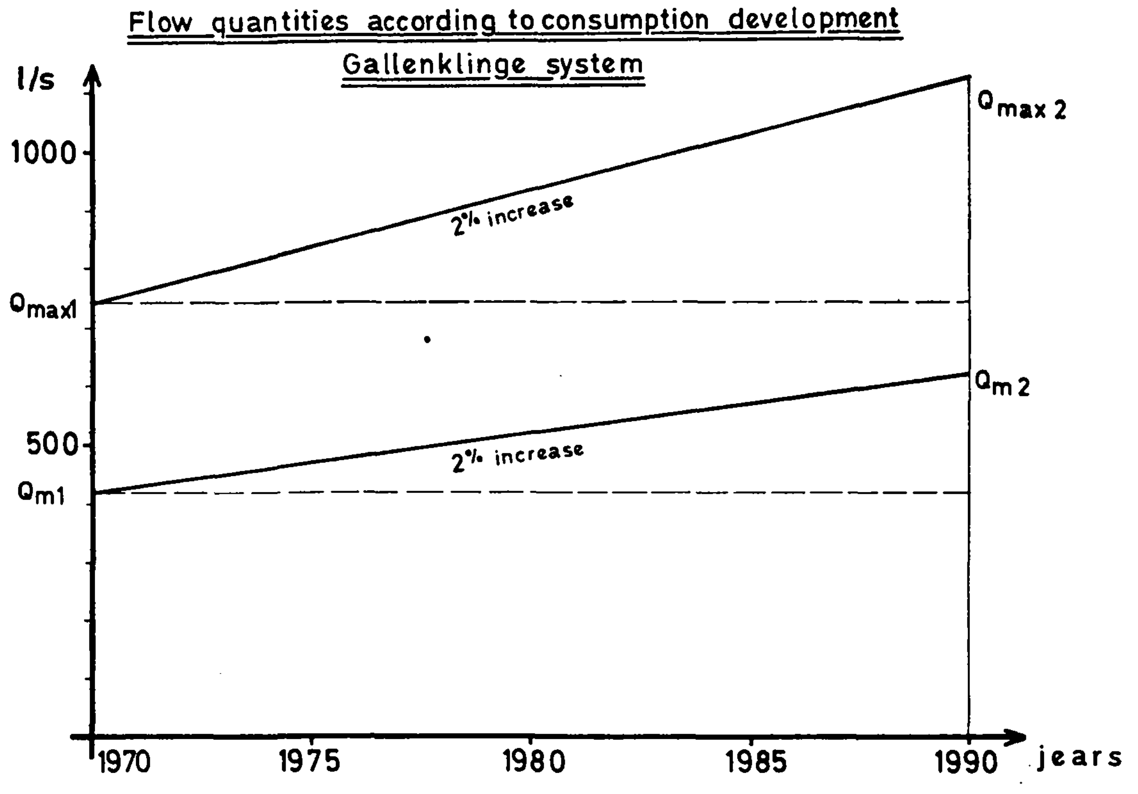
Fig. 6

Profitability comparison between turbine and pressure reducing systems with remote control through a central control station

	Waterworks operation			Power station operation
	2 turbines	1 turbine+ 1 pressure reducer	2 pressure reducers	1 turbine
Investment costs/year	DM 1 500 000	DM 820 000	DM 290 000	DM 700 000
Interest+12% depreciation/year	180 000	98 400	34 000	84 000
Operating costs/year	20 000	12 500	5 000	20 000
Maintenance costs/year	10 000	7 000	4 000	10 000
Total costs/year	210 000	117 900	43 000	114 000

Fig. 7


Gallenklinge turbine system
518m above sea level
395m above sea level
elevated tank
turbine
cylindrical piston valve


Flow quantities according to consumption development
Gallenklinge system
l/s
1000
500
Qmax1
Qm1
Qmax 2
Qm 2
2% increase
2% increase
1970
1975
1980
1985
1990
jears

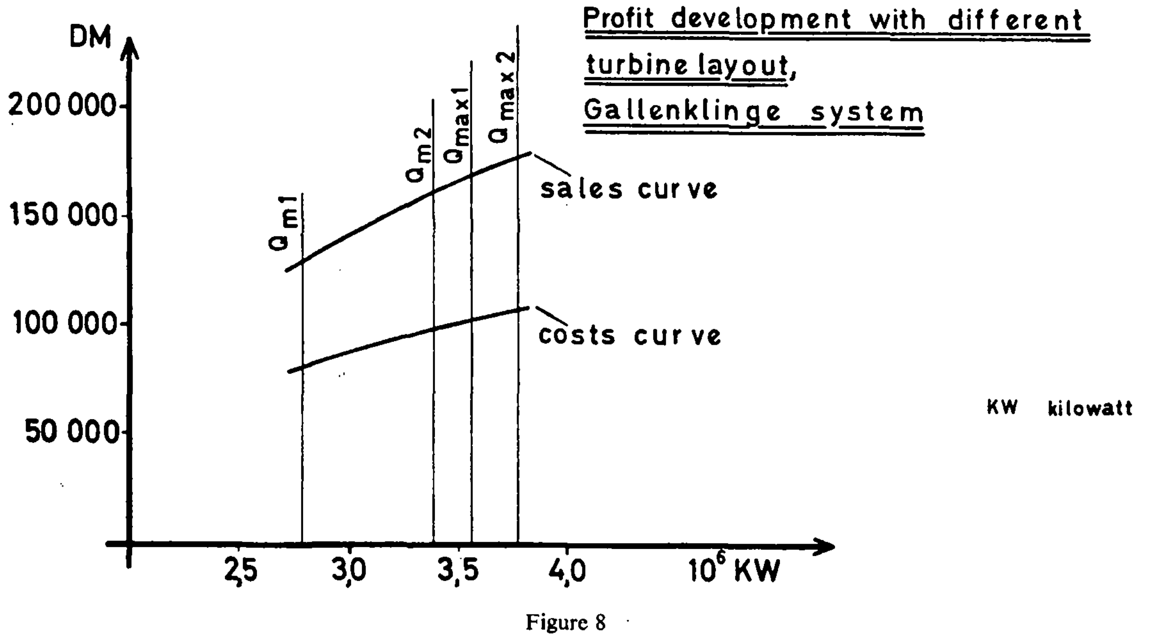

Profit development with different turbine layout, Gallenklinge system
DM
200 000
150 000
100 000
50 000
Qm1
Qm2
Qmax1
Qmax2
sales curve
costs curve
2,5
3,0
3,5
4,0
10⁶ KW
KW kilowatt

Figure 8

3.2.1.3 Pressure reducing valves in networks

It is proposed to deal only with pressure reducing valves smaller than those described in 3.2.1.1 which control automatically, i.e. without extraneous auxiliary energy. The auxiliary energy is removed from the controlled system. The control always takes place steady proportionally or proportionally integrally according to the selected downstream pressure.

Pressure gradient and flow rate are the criteria for selection of appropriate size.

The first uncontrolled pressure reducing valves were built as early as at the turn of the century. Controlled valves have been well-known in Germany for 25 years.

The most important demands on pressure reducing valves are [10, 11]:

1. The selected pressure should remain as constant as possible with fluctuating consumption and changing upstream pressures.
2. Tight seal at zero consumption and even over a long operating period (the pressure should not be transmitted).
3. Operational safety.
4. Inexpensive maintenance.

Pressure reducing valves are used particularly downstream of long-distance supply system branches and in municipal networks with large differences in elevation, either for the supply of pressure zones, insofar as the water is available on a higher level, or for the reduction of pressure in individual buildings as a house pressure reducer.

Whilst pressure reducers have their limitations, they also have the following advantages:

—They are relatively cheap and require no costly structures.

—They are self-controlling and can be re-adjusted within certain pressure limits.

—Under certain pre-conditions, they offer the possibility of a small pressure division, where there are large differences in elevation within the supply area.

—They allow pressure compensation to overloaded or distant network components, which are supplied via water tanks.

—They protect elements of the system against excess pressure and as such also serve to cut down on noise.

At present the pressure reducing valve does not yet offer the operational safety of a tank, which is exposed to atmospheric pressure and whose storage effect is a pre-requisite for a continuous supply. The pressure reducing valve is an essential aid for the applications considered above, under the following conditions:

—Feeding in against closed network only wherever short-term transmission of the upstream pressure does not give grounds for any serious damage (otherwise additional safeguarding by a safety valve, insofar as the latter can still draw off the water up to a certain upstream pressure, in case the valve should fail),

—Use of pressure control tanks with long feed lines,

—Regular maintenance,

—Shutdown times are not too long,

—No overdimensioning.

The pressure reducing valve with external, auxiliary energy has even better control characteristics and can be controlled according to the pressure profile of a distant network point. The operational safety of the pressure reducing valve with external energy is not, however, increased in comparison with that without external energy.

The possibility of failure of the pressure reducing valve must be considered and corresponding precautions taken. A disturbance in this case means more than just shutdown. It means possible impermissible pressure rise in the pipe network and household installations. If this is in excess of the stress limit of the pipelines and consumer devices corresponding damage will occur.

3.2.2.1 Pumps with long pressure lines

With the operation of pumps in long pressure lines, particular attention must be paid to the non-steady flow conditions. These include start up and shutdown of the pumps, and also control of them by means of changing the pump speed or by actuation of a control valve. If the speed decelerates rapidly, and as the delivery head of the pumps decreases as the square of the speed of rotation, an oscillation is caused in the water column. With longer lines, the full, direct, negative water hammer can take effect, as the discharge within the reflection time of the pressure waves returns to zero [7]. The effects of the velocity change are obviously different for individual systems, as also are the measures which should be taken to keep the pressure change within permissible limits. In any case, the potential damage from underdimensioning should be avoided just as much as the excessive system costs due to overdimensioning.

The permissible pressure change in the pipeline is derived from two conditions:

Tear-off condition: the pressure line should not drop below the height of the pipeline at any point.

Max. pressure condition: max. pressure head to be expected should not exceed the permissible operating pressure (rated pressure) of the line.

An assessment of whether there is a danger of water hammer can be derived from the formula $K_2 = (L \times c)/\sqrt{H}$ (length of pipeline L and delivery head H in m, flow velocity c in m/s), which for $K_2 > 70-100$ necessitates the installation of water hammer safeguards [12].

In addition to relief valves, energy accumulators are also very frequently used as water hammer safeguards. Their function is the control of the ratio of the kinetic/translational energy of the water in the line, to the kinetic/rotary energy of the pumping unit.

Energy accumulators are:

a flywheel with the flywheel mass of the pumping unit for the storage of rotary energy;

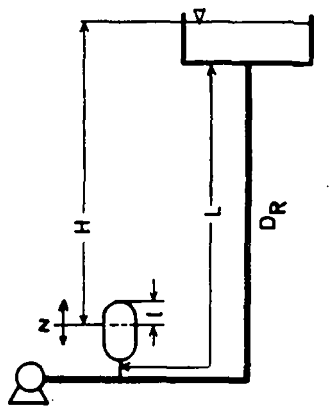
an air chamber for the storage of hydraulic energy.

Flywheels

Flywheels are suitable for horizontal pumps with higher speeds at average or large delivery heads and pipe lengths as far as possible not in excess of 3 000 m.

The determination of the pressure drop at pump run-down can be worked out using electronic computers. In practice, however, the graphic method of Schnyder–Bergeron [15, 16] is still proving very useful. The speed change during the pump rundown process can be determined using an iteration method or more quickly still with the aid of a ω–K diagram [12]. The approximate speed of rotation after the first reflection period is found using the values for the pipeline length, the velocity of the pressure transmission and the torque and flywheel effect of the pump. In the c–H diagram, the point of intersection of the water hammer straight lines with the

Diagram of a water supply system with pressure control tank



H the static pressure head in m/wh

L length of line in m

D_R diameter of pipeline in m

l height of air space in air chamber in stationary condition in m

z level deviation

Figure 9

corresponding c–H line for the pump obtained from the relationship $\omega/\omega_l = c/c_l = \sqrt{(H/H_l)}$, gives the pressure drop after the first reflection period.

Air chambers

The air chamber is used with great success as a damping element in reducing stations with long supply lines and also as a control element for zonal pumping plants. Its main application is water hammer protection for pump pressure lines, Fig. 9.

There are several design criteria for the air chamber in pump pressure lines. In addition to the breaking and rated pressure conditions, there is a third condition, namely that the volume of the air chamber should be large enough to ensure that no air can escape into the pipeline when it is released.

It has become common practice to select the volume of the air chamber to within 1–2% of the content of the pump pressure line to the tank, as an initial approximation. However, in the majority of cases, a more accurate calculation of the volume of the air chamber should be made. However, to date no success has been achieved in combining all the phenomena of the oscillation process into one single formula. The step-by-step calculation of the Schnyder–Bergeron graphic method gives good results. The water which flows out from the chamber after pump failure results in a reduction of the volume of water and in a change in the pressure-volume state of the gas cushion in the chamber. This change of state, assumed for several volumes within the time unit, results in part of a curve in the c–H diagram, which at the intersection with the water hammer straight line, gives the pressure drop after the first reflection period (if this was chosen as a time unit). The other points are developed therefrom and, for reasons of simplicity, the change of state can be calculated as an isotherm, although its actual trace represents a variable polytropic curve.

Ludwig and Stack [13] solved the oscillation equation for the isothermic change of state by the incorporation of pipe friction, introducing the oscillation of the water level z in the air chamber as a variable. However, the elasticity of the water and pipeline (so-called "Inelastic Theory" as for example with Boerendans [3] and Evangelisti [6]) is not taken into account. This method gives good results except with small chamber volumes.

Starting from the differential equation for the downward directed motion

$$\frac{d^2z}{dt^2} - m\left(\frac{dz}{dt}\right)^2 + n\left\{p_0\left(\frac{1}{l-z} - 1\right) + z\right\} = 0$$

$$m = \frac{F_w \times \lambda}{2F_r \times D_r} - \frac{F_r}{2F_w \times L}$$

$$n = \frac{g \times F_r}{L \times F_w}$$

L = Length of line
z = Water level oscillation, air chamber
$F_{r,w}$ = Cross-section pipe, air chamber
l = Height of air space in chamber
λ = Characteristic value of friction loss

the solution equations are developed, which implicitly contain the extreme values of z, which can be easily derived by trial and error. Nevertheless, for numerical evaluation, values for the respective integral logarithms should be taken from mathematical tables.

The analogue computer also lends itself to the solution and evaluation of the above-mentioned differential equation. This has the advantage that the effect of parameter change can be rapidly recognised, so that different operating conditions can also be simulated. The results appear as curves on the plotter [17].

Non-return valves

Sudden closing of the valves when the pump fails is hazardous. It occurs if the time in which the velocity of the water column drops to zero (T_R) is shorter than the valve shutting time (T_S). The longer the pipeline, the greater the velocity, the smaller the static pressure and the greater the pump run-down time, the larger T_R will be.

Air chambers shorten the effective length of the pipe to the distance between pump and chamber. Water hammer is therefore the rule here, even weights on the valve lever are not particularly effective. The installation of non-return valves with a flap edge seat on one side should be disposed with. In contrast, nozzle-type non-return valves with spring pre-tension have proven successful. Attention should be given in every case to stable pump characteristics.

3.2.2.2 Pumps in the network

The size and design of a pumping plant are determined by flow rate and height and the type of layout of the pumps with regard to the suction and pressure end.

Four types of layout can be identified [8]:

1. Tank—pump—tank T - P - T
2. Network—pump—tank N - P - T
3. Tank—pump—network T - P - N
4. Network—pump—network N - P - N

The particular situation in a supply area is always decisive for the type of layout. Irrespective of whether the system is necessitated by the development of a built-up area or by its subsequent installation due to development (overloading of the network), the following factors are normally the basis for the layout:

—Topographic position of the respective procurement and storage point, as well as that of the supply area;

—Length of supply lines;

—Size and condition of pipe networks;

—Structure of consumption.

1. $T_1 - P - {}^{N_1}T_2{}^{N_2}$

With this layout it should be made clear whether the tank T_2 is operated as a flow tank, whether the feed line to the tank is only a pump pressure line and whether the network N_2 is fed separately from the tank; or whether T_2 is at the other end of the network as a corresponding tank or whether the pump pressure line is also the supply line. In the latter case, the delivery head will change according to the network consumption requirement; pumps with a steep characteristic should be used.

If the costs of a flow tank and a corresponding tank are compared, the latter is usually cheaper. The take-off line N_2, which is separately laid, is not necessary.

As the network is fed from two sides, the tank take-off line does not need to be designed for peak consumption. A further advantage of the corresponding tank system is the possibility of subsequent correction of the supply pressure by changing of the pumps. With the flow tank, the hypothetical energy curve is finally fixed, while this is super-imposed for the corresponding tank with the sum of all head losses from the network to the tank. The flow tank is simpler to operate, the geodetic percentage of the delivery head is constant and there is less danger of water stagnation during storage.

2. $N - P - T$

In this case, the network pressure is not always sufficient to supply the tank. By corresponding dimensioning of the tank, the pump can be completely adjusted to the network conditions. Pump or supply line failure does not lead to a short term failure of the water supply. The water supply for fire-fighting is in reserve.

3. $T - P - N$

The type of layout with direct delivery into the network is encountered, if

—Sufficient height is not available for the tank on flat ground, or

—Two supply zones are super-imposed in height and the tank for the lower lying zone simultaneously serves as the drawing off tank for the pumping plant delivery into the upper zone.

In the first case, speed controlled pumps are very suitable for the supply of larger parts of the network. Control of the flow is carried out according to water requirements, where in each case only the required delivery head is produced. The speed control can be carried out with gears or by control of the drive motor, which is normally more effective. The basic delivery flow can be met by pumps of a fixed speed. The most frequent form of zonal pumping plant is the air chamber pumping plant, which operates more economically with pressure controlled pumps up to a delivery rate of Q = 100 l/s, than a quantity controlled pumping plant.

The advantage of direct discharge into the network is its ability to be adapted to consumption by the switching on and off of corresponding pumping units. With less consumption and correspondingly less frictional head losses, pumps with less delivery head can be used.

4. $N_1 - P - N_2$

With this layout there must be sufficient capacity present in the network section N_1 to draw off the required amount of water by operation of the pumps, without impermissible pressure drop.

The feeding-in into larger networks is also carried out via speed controlled pumps according to requirements.

"Pipe pumps" have proved themselves for pressure boosting of individual network sections. In normal operation, as pipeline installations, these are bypassed if the pump is shut down, and are only switched on when required.

3.2.2.3 Booster systems on private properties (BS)

The normal operational pressure supply from the waterworks only extends to the highest situated consumer according to local conditions (compare with section 4). Multi-storey blocks can only be supplied with the normal supply pressure up to a height according to local conditions, while the floors above this height have to be supplied via a booster system which belongs to the building itself. Booster systems cause severe localised stress on the network. The following is demanded in a DVGW guideline [5]:

Booster systems should be so laid out, designed, operated and maintained that the continuous operational safety of the water supply is guaranteed and neither the public water supply, nor other consumption systems are interfered with. A subsequent change in the drinking water quality should be excluded.

Interference to the supply network is caused by:

—the possibility of backflow,

—cases where impermissibly large water hammer is produced at switch on and switch off, due either to too large dimensioning or too large a consumption, or if, due to the operation of the system, water is drawn off from neighbouring consumers.

If the network cannot provide a sufficient supply for the booster system an auxiliary storage tank should be used.

The direct connection of the booster system is only allowed if the velocity change in the connecting line, caused by the switching on or off of each pump, does not exceed $\Delta c = 0{,}15$ m/s or if an air chamber is installed on the suction side, or if this is guaranteed by other measures.

Some manufacturers of booster systems describe their product as being water hammer free. This means systems which produce smaller water hammer, insofar as their switching actions occur with smaller volumetric flow or with a time lag.

Many booster systems have pressure controlled switching with a large air chamber installed downstream as a control element. At switch on, full output from the pump is produced, regardless of whether there is consumption or not. If the pumps' dimensions are too large, as is mostly the case, this has its full effect on the network. The demand curve is the result of average peak values, which in practice are not met in every case. On

the basis of constant stress curves [9] for houses with flushing cisterns or flush valves, it can be shown that only the latter bring larger and short-term peak draw-off of water. Pumps for air chamber systems in booster systems should be closely attuned to the anticipated consumption structure and dimensioned accordingly. It is also better to have a stand-by in the chamber for this contingency. This is equally true of booster systems for department stores, office blocks, hotels and hospitals.

With important buildings, the installation of a further reserve pump should be considered to cover occasional, but rare consumption peaks.

Booster systems which have only a small diaphragm tank as a control element on the pressure side or even none at all, discharge only the amount drawn off by the consumer on each occasion. The system only switches off at the minimum consumption, i.e. at a small delivery flow. Water hammer at switch off is therefore normally within permissible limits. If there is a power failure, this system can also produce an impermissibly large water hammer, Fig. 10.

Although a system without an air chamber is better for preventing impermissibly large water hammers in normal operation, it will not displace the booster system with air chamber, because, despite lower investment cost, it cannot attain the profitability of a system whose pumps always operate at optimum efficiency.

4 Supply pressure and profitability

Whilst it is capital costs which significantly mount up with tanks, it is operating costs, in particular energy costs, which mount up in the case of pumping plants.

With current electricity costs (e.g. Stuttgart, 1975: basic charge 12,9 DM per KW and month, operating price 9,7 pfennigs per KWh) and an average pro capita consumption of 240 l/day, every pressure rise of approx. 1 bar provided by pumps results in total costs pro capita and per year of 53,6 pfennigs. All precautions for reduction of energy consumption should therefore be considered at the system planning stage, e.g.:

—Position and arrangement of the pumping plants in the network; number, graduation, control and degree of efficiency of the pumps.

—Least possible distance between the pumping plant and the focal point of consumption. This results in smaller investment costs and, as a consequence of lower pipe friction losses, smaller energy costs.

—Maintaining static pressure head at the absolute minimum necessary to guarantee the minimum flow pressure to the highest consumer in the area.

If consumers are supplied with pumping pressure by a single point to different locational heights, then the

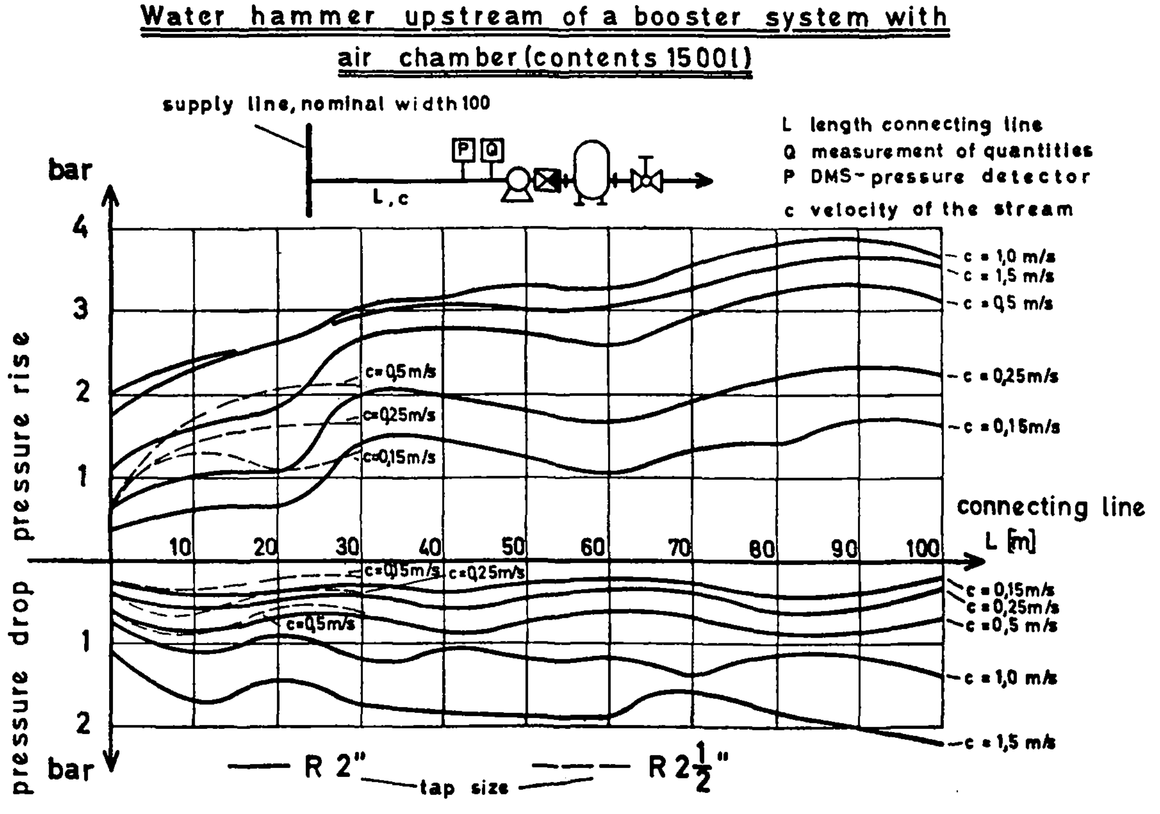


Figure 10

3.3 Control and monitoring

There is available a number of automatic control procedures for the maintenance of a given pressure and other important measuring parameters. Alternatively, control will be undertaken wholly or partly by a central station. Both systems can be manually controlled and require uninterrupted monitoring by the central station. Central, manual control can only be used with very small systems or with a limited number of switch commands. With larger systems, the evaluation of data and control can be carried out via a process computer.

same delivery head z_i should be used for them even though this pressure head would only be necessary for the highest consumer. As this is normally unprofitable, the supply area is broken down into supply zones (mostly zones with graduated heights) [1].

The zoning can be caried within the specified permissible pressure limits; a break down into arbitrary numbers of sections of smaller pressure differences can, however, only be continued until the amount of investment caused by the increase in the number of operational devices is counterbalanced by the advantages and savings resulting from a smaller supply pressure. That limit is of

Waterflow -out from tap valves experimentally determined at different pressures

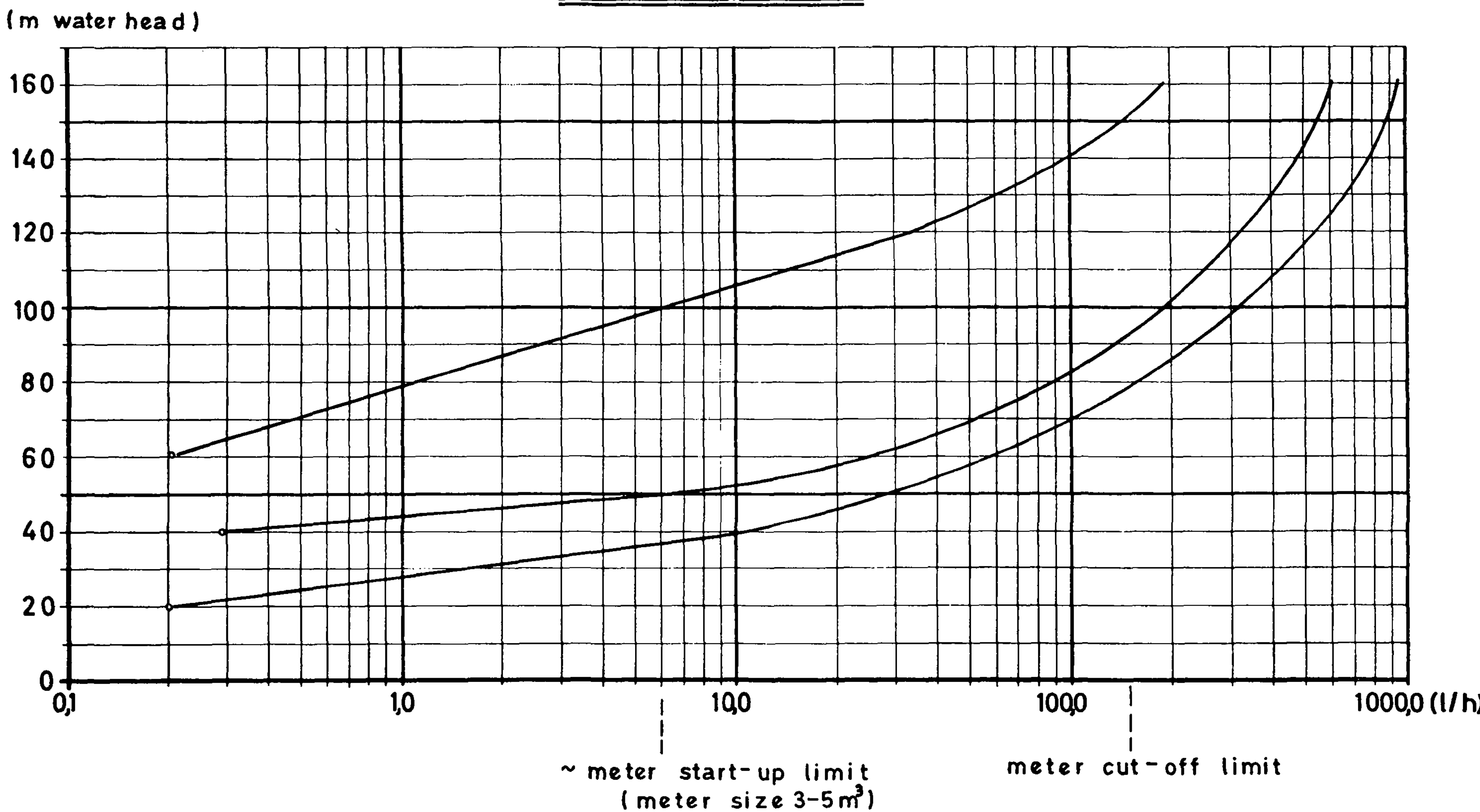


Figure 11

course dependent on local conditions and therefore varies. However, too many zones make operational surveillance more difficult.

4.1 Pressure zones

Supply to the individual zones can be in parallel from a central point or in series from one zone to its neighbouring zone.

A separate pressure line to every individual zone is necessary with parallel connection. This system should only be selected if transporting distances and differences in elevation are not too large. For larger transporting distances and differences in elevation, a series connection is more economical. But here, however, operating stations are necessary in the respective transition.

If the supply pressure is not to exceed 60 m water head (with a minimum pressure for the highest consumer of 20 m water head), the differences in land level may be approx. 40 m. The larger is the permissible max. supply pressure for the lowest-lying consumer the less the number of zones required. Too high pressures cause increased stress on fittings, pipe materials and seals and therefore an increased amount of damage. An increase in consumption and leakage losses can also be proved, as shown in the following.

5 Supply pressure and water consumption

From tests carried out it has been shown that there is a connection between pressure rise and consumption quantity or the quantity delivered into the network. There is no doubt that water losses (trickle losses) and leakage losses (pipe connections, cracks, hair line fractures, pitting) are involved in the increase in consumption.

5.1 Trickle test

An ordinary tap R ½" with normal seal was used. The fitting was connected to a testing system and so adjusted to several set-points (20 m water head, 40 m water head, . . .) that a drop formed every three seconds, following which the pressure was increased in each case.

The following can be read off and deduced from Fig. 11:

With a normal supply pressure, a valve may lose one drop (= 0,3 l/h) every three seconds. If the supply pressure increases during the night over a period of 12 h by 10 m water head, the trickle loss of the same valve correspondingly increases to 6,0 l/h. This leakage loss is too small to be recorded by the water meter and is therefore not paid for.

For a supply area containing 80 000 water meters and assuming one trickle point allocated per water meter, the result is that 2,1 million m^3/a of consumption is not metered (= 30% of the difference between the water supplied and that paid for by consumers).

The above reasoning is somewhat hypothetical because it is not known how many actual trickle points there are. Also only one trickle point is allocated to each property. However, in practice slow losses in flushing cisterns, tap valves etc. can be observed everywhere so that the figure estimated is probably not too far out. The following example bears this out.

5.2 Verification in a block of flats

A Woltmann water head meter (50 mm) was bypassed in an apartment block of 56 flats (built in 1963) and the nightly consumption measured with a 7 m^3 meter. The actual consumption times and the leakage losses can be clearly read off from the diagram (Fig. 12). From the gradient of the straight lines, an overall consumption in four hours of 480 l results with leakage losses of 230 l (= 48% or 57 l/h leakage loss) corresponding to 1 l/housing unit. This value appears extraordinarily large, but confirms the result of 5.1.

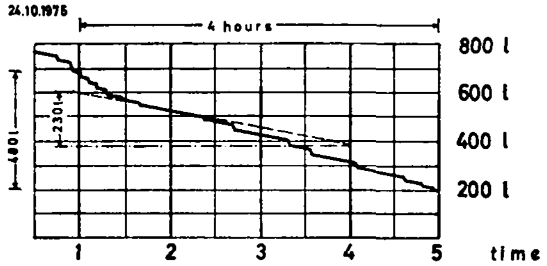


Figure 12

5.3 Zonal consumption with various pressures

In a zone supplied by pressure reducing valves (2 860 inhabitants), the consumption was measured starting from normal pressure and then at smaller pressures reduced in each case by 5 m, 10 m water head and so on. The curve in Fig. 13 resulted from the basic values and clearly shows a fall off in consumption dependent on the reduced pressure.

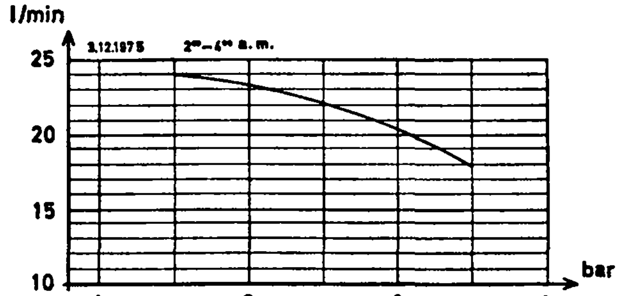


Figure 13

5.4 Zone with air chamber pumping plant

A pressure quantity curve was deduced from the characteristic circuit diagram of the air chamber pumping plant and this is shown in Fig. 14.

The consumption peaks shown during the first 20% of the tap-off phase (nightly consumption) can be reproduced from other graphs. No explanation for the steep initial slope of the curves (increased consumption could be found other than the increased pressure level.

Unfortunately, the example in 5.1 is not sufficient to be able to prove the relationship between the leakage loss and the increased consumption at increasing pressure; the influence of the pressure level on consumption can, however, be shown.

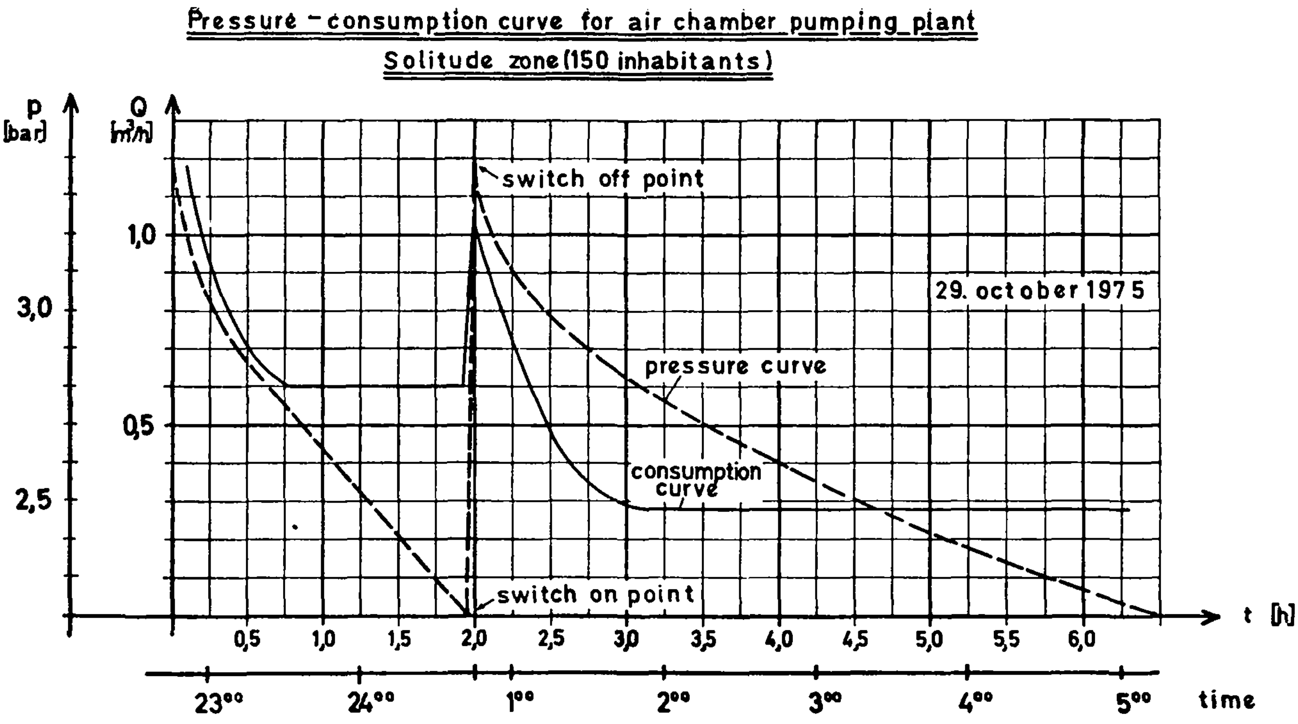


Figure 14

6 Legal aspects of pressure control

In Germany there is liability for risks* in the field of electricity and gas; in the field of water supply, there is liability for intentional acts†. The judiciary in the Federal Republic of Germany has, however, demanded a duty of care of the public supply companies such that there are not too many differences between this and absolute liability in the final analysis.

In the field of electricity and gas, the General Supply Conditions are standardised by the General Liability Declaration of 1942. These regulations have the character of statutory orders and conclusively control the contractual relations between public supply companies and consumers in the Federal Republic of Germany.

No such General Liability Declaration has been made in the field of water. With the General Supply Conditions in this case, these are actual General Operating Conditions, which control the rights and duties of the waterworks and the consumer in civil law. In accordance with this the water supply companies have reserved the right to make pressure changes. It is regarded as an accessory, contractual consideration that the public supply company will inform the consumer about any intended pressure change. The consumers, however, have no claim upon the maintenance of a particular pressure.

If damage occurs due to a change in pressure, it is for the judiciary to decide in each case whether the damage occurred as a result of typical operational dangers for water supply companies (e.g. failure of technical equipment) and therefore whether it is a risk to be borne by the consumer.

To sum up, there are no adequate criteria given by the legislature in the Federal Republic of Germany for the pressure field.

* Liability for risks means responsibility for a certain material or operational danger for social reasons, where a culpable behaviour of the party liable for damages does not have to be given.

† Liability for intentional acts means responsibility for culpable behaviour.

7 Conclusion

In this paper, an attempt has been made to derive the correlation between Pressure Control and technical operation and to look at these from an economic point of view as well. In each case, examples of operating conditions for energy changing systems, both with long lines and in networks have been examined and in addition water hammer problems have been discussed.

The necessity of reducing excess supply pressures as a requirement for the reduction of the energy used in networks supplied with pumping pressure has been discussed, as has the requirement for the reduction of operational malfunctions and network losses.

In the final analysis, Pressure Control and evidence of its preservation by uninterrupted recording of operations is important for protecting the public supply company against unjustified claims for compensation.

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Résumé

En traitant ce sujet, les aspects techniques, économiques et légaux ont été considérés.

Le principe essentiel de toute distribution d'eau est de garantir les besoins en eau potable en tous points de la zone desservie, en quantité suffisante, de la meilleure qualité possible et sous une pression d'alimentation adéquate à tout moment.

Ce que l'on entend par pression d'alimentation est la pression statique présente dans le réseau: elle doit être au moins suffisante pour garantir le minimum de pression de débit aux points de soutirage les plus éloignés et les plus élevés dans la région desservie. La pression maximale admissible dans la réseau est fixée à 10 bars. D'une façon générale, les pressions normales dans le réseau sont de 2 à 8 bars.

Ce que l'on entend par contrôle de la pression est l'influence sur et le contrôle de tous les paramètres, en vue de garantir une pression d'alimentation adéquate en tous temps, quelle que soit l'altitude du point de consommation.

Pour la contrôle de la pression, il faut dériver les relations suivantes: le contrôle de la pression est une exigence pour l'exploitation technique; le contrôle de la pression permet une exploitation claire, facile à suivre, économique. Le contrôle de la pression, entendu comme réduction des pressions d'alimentation excessives, est avantageux pour:

—la vie des conduites et des branchements,

—la protection des appareils et installations du consommateur,

—la réduction des pertes causées par les fuites possibles.

Le contrôle de la pression est nécessaire comme preuve de l'exploitation régulière de la distribution d'eau, par exemple du point de vue légal.

On discute de l'équipement de modification de la pression en technique d'exploitation, par exemple:

pompes et réducteurs de pression, rattachés chaque fois à l'exploitation d'une longue conduite et dans le réseau.

Description est donnée d'une vanne à piston cylindrique avec plaque d'orifice comme exemple de réducteur de pression, dispositif de contrôle à l'extrémité d'une longue conduite avant déversement dans le réservoir.

Les problèmes spéciaux et mesures pour éviter la cavitation en exploitation continue dans les régions où il y a des ouvertures partielles sont discutées pour un cas donné.

Pour l'emploi des turbines en vue de réduire l'énergie potentielle hydraulique, une comparaison économique est donnée à titre d'exemple, avec les accessoires de contrôle comme les réducteurs de pression.

Bien que les turbines exigent un investissement supérieur aux réducteurs de pression, elles peuvent être utilisées avec bénéfice. Mais cependant on ne peut pas se dispenser d'un réducteur de pression comme unité de réserve.

Les utilisations possibles des vannes réductrices de pression dans le réseau sont mentionnées. Les conditions pour leur bon usage sont énumérées.

Référence est faite aux changements admissibles de pression dans la conduite lors des manoeuvres sur les pompes pour les longues conduites ou aux mesures à prendre pour étouffer les coups de bélier.

Des valeur empiriques des domaines d'application sont énumérées pour les volants et les réservoirs à air qui sont les accumulateurs d'énergie les plus utilisés.

Référence est faite à quatre types de schéma d'insertion des pompes dans le réseau, et les conditions d'exploitation qui en résultent sont décrites. Une attention particulière doit être apportée aux pompes de surpression installées dans les propriétés privées, car elles peuvent amener des désordres dans le réseau de distribution.

Si la pression d'alimentation est produite au moyen d'une énergie de pompage, la pression peut être rettachée directement au coût de l'énergie. Le rendement de l'exploitation est augmenté si l'on évite les pressions excessives. Les régions où il y a d'importantes différences d'altitude sont divisées en zones d'altitude graduée, une différence étant faite entre les connections en série et parallèles.

Lorsque la pression d'alimentation augmente, on observe une augmentation de la consommation dont une part est attribuable aux pertes par les fuites.

Finalement, le contrôle de la pression et la conservation de sa preuve par un enregistrement général de toutes les manoeuvres est important pour protéger le service de distribution d'eau contre les plaintes en dédommagement injustifiées.

Subject 3

Advances in the protection of distribution systems against backflow

by Ir. W. C. Wijntjes

Gemeentelijk Waterbedrijf Groningen

Introduction

At the I.W.S.A. Congress of Barcelona in 1966, G. J. Angele gave a survey of some historical accidents leading to the contamination of drinking water supplied by the water company (Ref. 1). In his lecture he made (along with a number of technical possibilities to prevent the pollution of the central drinking water system, proceeding from the drinking water installation as a result of particular, unpermitted connections or situations) a number of remarks on the responsibilities of the authorities and persons involved in the water supply of a water company. Some of Mr. Angele's important theses were in this connection: "Pure drinking water is a vital component of life. It is a tremendous undertaking to maintain a potable water supply through the vastly complicated public and private pipelines of a nation today. The duty to investigate the water supply and to ascertain possible sources of pollution rests on the water company together with the further duty of taking such positive action as is necessary for the protection of its customers!"

Now that ten years have elapsed since the Congress of Barcelona, the discussion should be opened as to which way developments in the possible pollution of drinking water installations and grids of water companies have been evolving, and how a better safeguarding (in spite of the connection of an increasing number of appliances to the drinking-water installations) can ensure that the drinking water that is supplied to the consumer remains drinking water.

Risks in drinking-water installations

An obvious question to pose is that of what possibility is there that eventually, under particular circumstances, a drinking water installation may be polluted, and what dangers for the consumers may arise from it.

Our English colleagues inquired closely into this difficult problem in the years 1970/71. The inquiry was made by the Committee on Backsiphonage of the Department of the Environment and was delivered as Technical Paper TP 82 of the Water Research Association (Now Water Research Centre at Reading) (Ref. 2).

Although the report refers to English circumstances, some conclusions are not unimportant for the problem posed:

(a) 61 % of domestic properties are at risk in terms of the requirements. If temporary connections are included as a risk this figure rises to approximately 85 %.

(b) The majority of risks (about 95 %) relate to cisterns and taps.

(c) The actual probability of occurrence of backsiphonage from domestic properties is a function of the risk as investigated herein, the occurrence of low or zero pressure and other factors such as taps actually being flooded or hosepipes being immersed in some contaminant during the occurrence of low or zero pressure.

The probability of backsiphonage occurring is anything other than very low.

It should be observed that in this investigation only household connections were involved. The results of similar inquiries connect naturally on a large scale with the question, if, and to what degree the construction or the modifications of drinking water installations are followed by an inspection. Moreover we shall have to accept on the grounds of practical experience of those who have something to do with the inspection of drinking water installations of factories, offices and other household installations, that countless examples can be given where the situation in the installation contains a serious threat to the health of a number of people. In fact it is for that reason that since 1968 in a number of countries ample attention has been paid to these problems, in which case in those countries increasing attention has been given to techniques of installation.

Pollution of the drinking-water

The pollution of drinking water from consumers' installations is a problem not easy to approach. The problem has been in existence since the origin of central drinking water supply and is, in fact, indissoluble from central supply, in particular where the drinking water installation in the case of direct connection is connected directly to the mains.

It means equally, based both on economic and practical considerations and experience that the quality of the drinking water from a draw-off point is sometimes inferior to that which it should be.

We have to make it our object to take such measures and precautions that the chance of pollution of the water supplied will be extremely small. In fact the basis of the problems is laid down by the necessity that for the transport of water, "pressure" is required.

In many devices connected to a drinking water installation, water is used as a solvent for other substances, e.g. the washing-machine.

In these appliances contact with other substances takes place, and it is possible that the drinking water in the consumer's installation and also in the grid of the water company becomes polluted with this matter.

And now, what are the dangers that may threaten, and what measures may be taken to prevent possible pollution?

When studying the question of which dangers may threaten, two elements have to be distinguished.

(a) What technical conditions have to be fulfilled so that the foreign matter pollutes the drinking water.

(b) If the drinking water has been polluted with such foreign matter, what may be the consequences for public health.

In the Netherlands a distinction was made in 1967 (Ref. 3) between so-called "press-cross" connections and "suck-cross" connections.

By a "press-cross" connection is understood a connection between the drinking water installation in

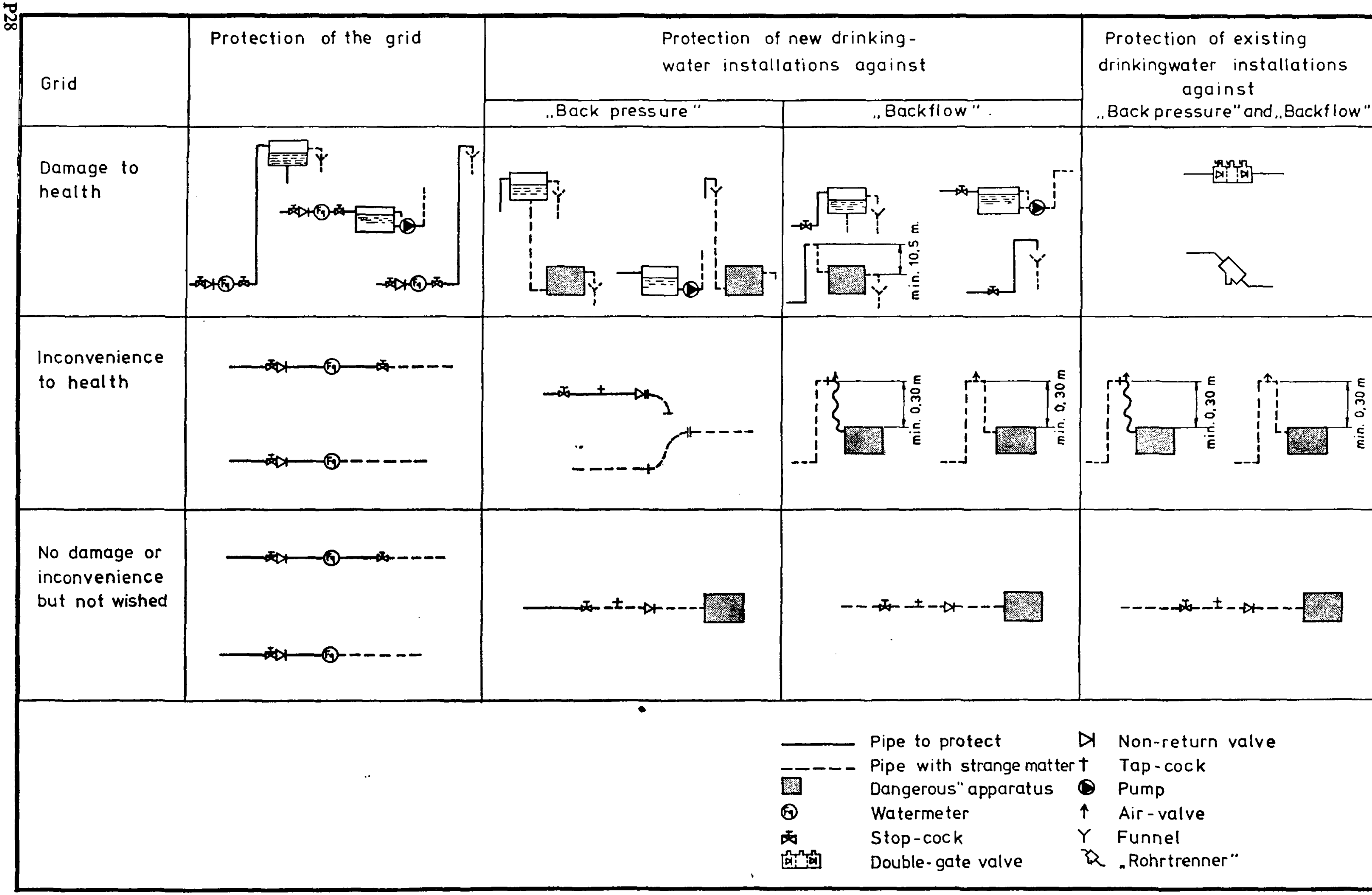


Figure 1—The classification table.

which a foreign substance is present under a higher pressure than that of the atmosphere.

The "suck-cross" connection is a connection between the drinking water installation and an apparatus or a pipe system in which a foreign substance occurs under a pressure not higher than that of one atmosphere.

The difference indicates practically, that in case of the press-cross connection only one condition is sufficient before contamination of the drinking water appears (e.g. the opening of a stop cock or where a non-return valve becomes inactive), and for the suck-cross connection there are at least two conditions (e.g. at a certain moment a suction must occur, and at the same time the connected apparatus must be in operation).

Fortunately it is evident, at least in Dutch practice, that press-cross connections may occur sporadically, but that suck-cross connections may occur in many places. This leads to the important conclusion that undertakings that have at their disposal an evenly balanced and well dimensioned grid, and can see to it that drinking water installations, too, meet similar demands, will have the least trouble with the consequences of a suck-cross connection. Because in practice, however, there are numerous situations in which they can occur, the necessary vigilance should be exercised even for those undertakings. But where there is the chance that pollution will appear, the possible consequences of the pollution will be important. In several countries attention has been paid to this aspect, and there has been a division into classes of hazard.

In the U.S.A. (Ref. 4) two categories with respect to potential or actual cross connections are distinguished:

Category 1. A physical or toxic hazard which could be dangerous to health.

Category 2. A non-health hazard which would cause aesthetic problems or have a detrimental effect on the quality of the water in the system.

In England (Ref. 5) and in the Netherlands (Ref. 3) they strive for a more extensive classification into three categories:

Class 1: Lasting damage to health appears (e.g. cyanide of potassium).

Class 2: Temporary trouble for public health appears (e.g. paratyphoid fever).

Class 3: The connection is undesirable (e.g. coffee).

Apart from the above, this categorical division does not give any practical guidance as to when and under what circumstances what demands have to be made. Types of protection that have to be allowed are still lacking in every class and furthermore all possible substances that may pollute the water, eventually with their concentrations, should be subdivided in the same way.

Means of prevention

Amongst the various types of safeguards an important difference can be made. There are a number of ways of preventing a foreign substance from flowing back into the drinking water installation as a result of a "backpressure" or a "suction".

An example of this is the so-called interruption. Provided that it is well executed, **backflow** is out of the question. Technical safeguards are liable to wear, age or corrosion and there are consequently no so-called absolute safeguards. In the course of time, dependent on construction, water quality and situation it is to be expected that they will lose their functional vitality and consequently not exercise their protective task any longer. The backflow preventer (non-return valve) is a good example of these.

From practical experience it is to be expected, however, that for this category of safeguard a difference can still be made between the quality of the safeguard as a construction and the time that the latter will last. A certain order would, so to speak, be drawn up.

With a view to the classification of dangers, it is clear that for the highest class of danger nothing less than absolute safeguarding will suffice, whereas in the remaining classes (referring to the English and Netherlands principal classification) safety devices will be sufficient.

In case of this classification the difference between "backpressure" and "backflow" is an important factor. The high loop (H = 10,5 m.) can be applied in the second case, but not, however, in the first case. Furthermore we have to observe that, on principle, a single safeguard will have to be sufficient.

The starting point has to be that the quality of the safety devices applied provide excellent and proper control.

The combination of safety devices or the formation of series of them leads to extra cost, and to higher pressure losses, but scarcely to higher safety.

In fig. 1 an essential survey of the possible safeguards in the various categories is given.

In some situations the "double gate valve" with a spacer between is acceptable. This shows by means of an exhaust pipe if both "valves", which for the rest must be sealed by the water company, are closed. Naturally the use of this type of safeguard depends upon the degree of danger (fig. 2).

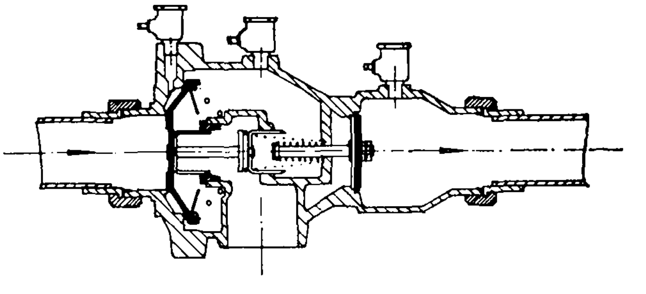


Figure 2—Double gate valve.

For the time being this type of safeguard has been found equivalent to a non-return valve in England. As a result of its signalling function it **does** give a higher protection, however, and application of this "device" might be considered in a number of cases to be on the line between categories 1 and 2 of fig. 1. In addition, a similar apparatus might be used in existing installations where reconstruction or adaptation of the existing installation would induce additional difficulties. In Germany (Ref. 6) also, an apparatus has been developed that might fit in with the whole. It is the so-called "Rohrtrenner" ("Pipe Separator"). Fig. 3 gives an example of the construction.

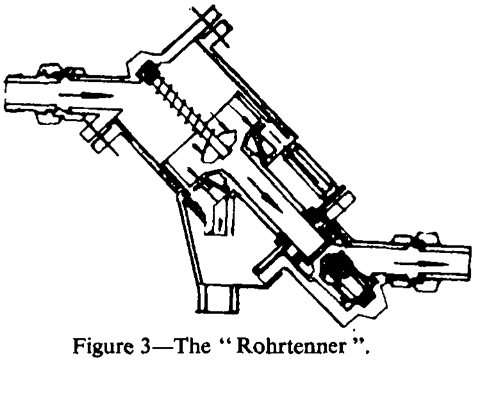


Figure 3—The "Rohrtenner".

Safeguarding of the grid of the water company

Theoretically speaking it would be sufficient with regard to the safeguarding of the quality of drinking water to introduce protective devices at those "points of use" that possibly would present danger for water quality.

From experience of the daily practice of the inspection department of a water company it appears, however, that it is impossible to obtain a 100% safe and protected situation in drinking water installations. This is in spite of the fact that in some countries only approved specialists may work on drinking water installations, and of a good inspection of the installation. Particularly in those categories of consumers where a so-called technical service is extant (hospitals, factories etc.), the possibilities of wrong situations are particularly great. This has been the reason that, particularly in the Netherlands, not only are special tapcocks and/or apparatus protected but an extra safeguard is provided at the point of the supply by the water company: namely near the main cock and/or the water meter.

Here, too, the class of hazard to which the premises must be assigned is weighed, and then the right protection is chosen. This means that to hospitals, sewage purification plants, large factory buildings, the water is only supplied via a reservoir as an interruptor.

In England (Ref. 5) they advise the separation of industrial water, or that part of the installation in which no drinking water in the sense of drinking water has to be tapped, from the pure drinking water installation.

In America (Ref. 4) the dangers of cross connections in dual industrial and potable water supply systems are emphasised.

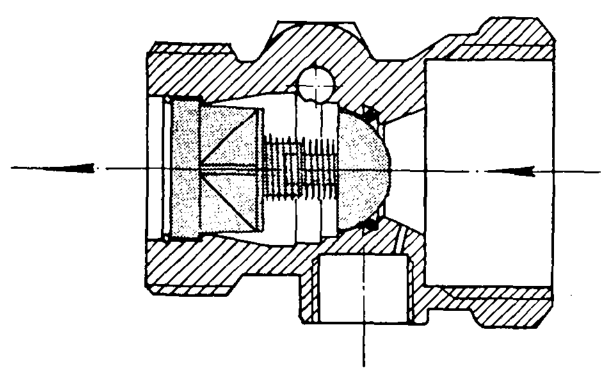
In the classes of hazards 2 and 3 the situation has been developing in a favourable sense in the Netherlands since 1967.

Conventional return valves generally close against a high difference of back pressure. Certainly they do not do so against a low pressure, which must be considered of real interest. Therefore types of so called hygienic non-return valves have been developed which also close in cases of a low difference of counterpressure or, in other words close when the direction of the current is still positive. For that reason the sealing is obtained by means of spring-loaded valves (Ref. 7).

In order to come up to these requirements the testing requirements contain not only a necessary flow rate with 10 m.w.h. headloss, but also a minimum flow rate with a headloss of 0,5 m.w.h. Moreover the non-return valve must be closed at 0,03 m.w.h. on the out-flow side. The construction has been designed in such a way that the closing-body is subjected to very little wear. In fig. 4 a similar construction is shown in more detail.

Since these non-return valves have been developed in the Netherlands, some millions of them have been installed, notably in service connections (already more than 50% of all connections have been safeguarded in this way). So far experience with them has been excellent. Furthermore, the building-in of the body of a non-return valve to the outlet of the water meter was developed in the Netherlands (see fig. 5).

Hydraulic experiments have shown that installing such valves has no influence on the measuring qualities of the water meter. Of course the advantages of this construction are particularly interesting. Not only are there practically no installation costs for fitting the non-return valves, but in addition the non-return valve can be preserved in a simple way; along with the water meter the non-return valve is developed also. In Germany, too, (Ref. 8) a trial has been carried out with similar non-return valves in a large town. As expected (Ref. 7) we may suppose that by introducing these non-return valves the risk of backflow into the mains system of a water company has been reduced by a factor of 20.



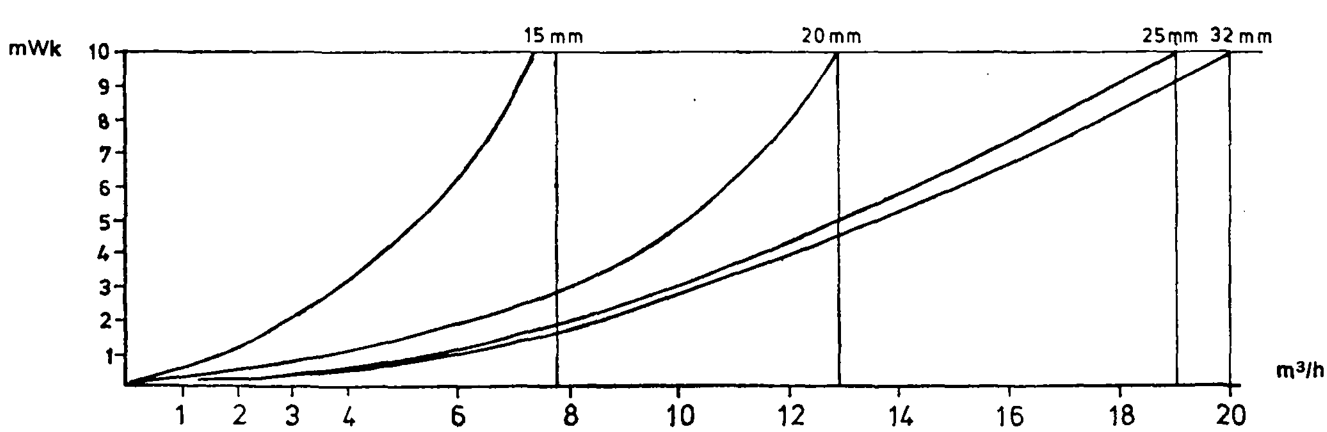
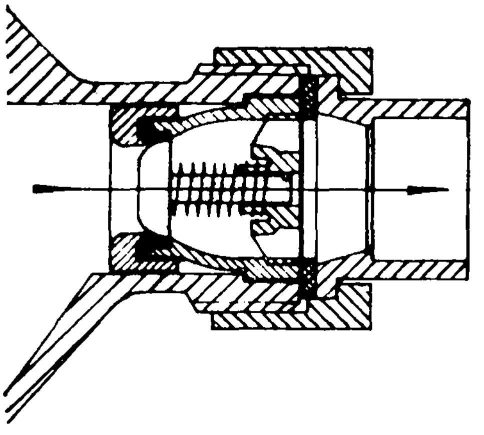


Figure 4—The hygienic non-return valve.



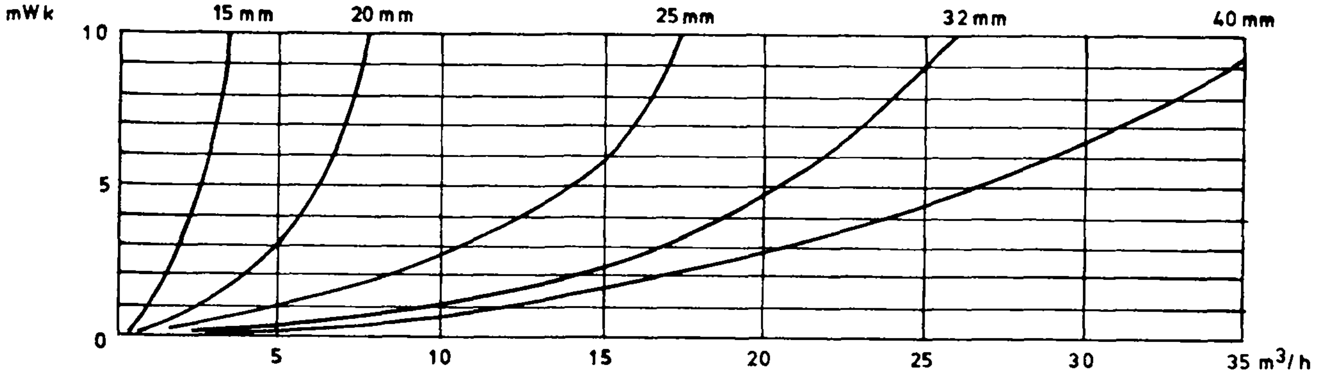


Figure 5—The combination of hygienic non-return valve and water meter.

The introduction of similar non-return valves is leading consequently to a much safer quality of the drinking water that is distributed. It must be emphasised that Netherlands water companies believe that when taking non-return valves into use, they did not take over the responsibility of a possible backflow of water. That remains the responsibility of the user of the installation. The non-return valve only serves to provide a higher level of protection.

The solution as advocated in America (Ref. 1), where the consumer has to place the non-return valve, has not been adopted in the Netherlands for fear that the maintenance of these valves would not come up to the demands made.

Naturally, special experience has been gathered with every type of connection when introducing the system of the non-return valve.

Loading effect

In practice it was observed that after the introduction of non-return valves in the supply pipe to premises in the Netherlands certain complications could arise, even if it concerned only a few cases. Among others a phenomenon may be observed that is known as the "supercharging effect".

By loading effect we understand the step-by-step or gradual course of a pressure increase in a closed system downstream of a closing element, such as a non-return valve. This pressure increase may arise from the following thermal (a–e) and hydraulic (f and g) causes:

(a) transfer of heat from the neighbourhood of the pipe system;

(b) transfer of heat from the waste gas of a bypass in a gas-heated hot water appliance, which contains cold water;

(c) transfer in a water heater of the accumulated heat of the water in the tap spiral;

(d) normal heating of the water in boilers;

(e) continual burning of water heaters caused by faults in the heater itself;

(f) pressure variations in the pipe system on the upstream side of the non-return valve;

(g) pressure impulses in the pipe system on the downstream side of the non-return valve (e.g. from the shutting of cocks).

The principal pressure increases have been measured by the Dutch K.I.W.A.-laboratory in the situations c and e. Pressures of ca. 35 bar. appeared there. Under Dutch circumstances lower pressures appear normally in the drinking water installations upstream of the float of the cistern as a relief-valve. An important conclusion of the working-group which accompanied the Dutch inquiry was that pressure increases in drinking water installations with a small capacity especially if they have originated from the functioning of water heaters, may have disastrous consequences.

It was suggested to increase the spontaneous ignition pressure of water heaters by application of a thinnen upward pressing pin to at least 35 bar. The fitting of relief valves must principally take place close to the apparatus. The principal problem that may arise when placing the non-return valves is the situation where the present non-return valve near the water heater did not work any more. In that case, and in addition, the relief valve on the water heater has probably not been working for a long time, and consequently may have stuck to the water heater.

An explosion may be the result if a non-return valve has been fixed into the inlet pipe by the water company.

Internal safeguarding of the appliances

With the possible kinds of safeguard(s) of the preceding paragraph it is possible to construct safely a drinking-water installation. On the whole it must be stated, however, that the safeguards that have been judged necessary are **minimum** safeguards.

The arm of the water world must in any case strive to take such measures as are necessary to obtain absolutely safe installations, without any risk. This means that we must make it our objective that only such pieces of apparatus are connected that have been provided with an integral interruption, i.e. internally safe appliance.

Tested apparatus

In case of efficient testing by inspection it is necessary that we can observe easily whether or not the apparatus connected has been constructed with an integral device to interrupt supply. For that purpose it is particularly useful, if types approved, whether by a nationally or generally acknowledged institute or acknowledged organisation on the basis of testing requirements, can bear the certificate of "internally safe". Also the safeguard to be applied in non-internally safeguarded apparatus, together with the eventual placing of non-return valves in the supply-pipes to the premises, need to be inspected for preference by one central entity.

International aspects of washing machines

A good example of this development may be noted in West Germany. The "Deutsche Verein von Gas- und Wasserfachmännern" (D.V.G.W.) published Working-Paper W 503 in 1966 in which regulations were given for so-called internally safeguarded pieces of apparatus. Machines with the "D.V.G.W. Prüfzeichen" have come up to these requirements (Refs. 10, 11).

In these appliances care has been taken that the shut down according to the class of hazard 1 has been built into the apparatus itself so that extra safeguarding can be omitted, and direct connection to the drinking water system is permitted. Among others, washing-machines, dish-washers, urinals and slop pails are included in these regulations. Each year a survey is published by the D.V.G.W. of the appliances (manufacturers and types) that come up to the demands made. In the Netherlands and in Belgium (Ref. 12) there are a number of water companies which occupy themselves with the framing of regulations for so-called internally foolproof appliances. One development should be applauded particularly; that special appliances, among which are those named above, should only be allowed to be fitted if they meet the requirements of internal safety. In view of the fact that such appliances as washing-machines and dish-washers are put on the international market, an internationalisation of the regulations is urgently required. In this connection one good development is the drawing-up by the Common Market of similar directives for washing-machines and dish-washers, which in the year 1976 will be introduced officially as law in the countries which have joined the Common Market. It would be far better if these rules were handled at a still higher international level. Fortunately many makers of washing-machines and dish-washers will, when designing new machines, take the claims of the water companies into account.

Responsibilities

In connection with the possible contamination of drinking water from drinking water installation(s), an interesting point is the responsibilities of the various parties involved in the construction of such an installation. In the various countries this may involve several parties. In principle, at any draw-off point in the drinking water installation, it must be possible to draw drinking water which meets the demands that have to be made of it. The total distribution system, except in those countries where they work with an interruption (e.g. England), is seen technically as an uninterrupted whole. In addition there are two owners: the water company as owner of the main pipe system and the consumer as owner of the drinking water installation. Therefore, there are two responsibilities: that one of the water company and the one of the house-owner. On the part of the water company there is the responsibility for the water that is on the point of being supplied, both in a qualitative and a quantitative sense. In addition the water company is responsible for the qualitative aspects with regard to the water supplied.

As to the quantitative aspects, there are generally fewer prescriptions in use. In this connection we have to refer to the regulations for installations in Barcelona (Spain) (Ref. 9). There we can find what *minimum* quantities have to be supplied by several draw-off points. At the same time a classification is given of the types of dwellings on hand of the present drinking water installation, while in relation to it, the maximum withdrawal at their disposal is laid down. Generally no obligations of this kind are accepted by the water company. In various existing regulations it is stated that sufficient drinking water, being harmless to public health has to be supplied. In England the following phrase is added: "sufficient for the domestic purposes of all owners and occupiers of premises who are entitled to demand a supply for those purposes." In the "Landesgesetzblatt für Wien" of 23rd May 1960 (Ref. 14), it has been pointed out in para. 3 that a claim on a fixed quantity of drinking water or a special pressure cannot be made. In technical regulations for drinking water installations, if existing, the diameter of the main supplying it is chosen by adding a number of so-called consumption units, by which, if pressure permits, a fixed quantity of drinking water is supplied (Switzerland) (Ref. 13).

The problems of "backflow" have, as mentioned before, both qualitative and quantitative aspects. The first care is that a drinking water installation consists of a pipe system of sufficient diameter and that for special apparatus to be connected, the necessary allowances must be made. In several countries there exist directives which are declared binding by the water company concerned when entering into an agreement with the customer. In this connection a greater necessity for the good execution of drinking water installations has been ascertained. In Israel, for instance, new regulations are being prepared; in Switzerland they want to give the existing recommendations the force of law. In Belgium they are working with national directives. In most countries no clear demand, however, is made for the quality of water as it must be supplied at the draw-off point in the drinking water installation. The formula is often "wholesome" water. Further demands can sometimes be made by the responsible minister. This was the case in Spain 2 years ago, when in a situation that cholera was diagnosed, the water at the taps had to contain at least 0,5 mg Cl_2/l.

For the rest, responsibility for the quality of drinking water in drinking water installations has been laid in all situations at the owner c.q. the consumer. One often confines oneself to the owner, who, in this case, by way of the tenancy agreement, can hold the consumer responsible. In Vienna, in addition to a fine, imprisonment may be imposed in a similar situation (Landesgesetzblatt für Wien par. 28, 23 May 1960) (Ref. 14).

Supervision of drinking water installations

Having good regulations for the construction of drinking water installations together with having these installations executed by a well-educated and specially trained staff does not prove sufficient guarantee that every installation has been constructed in the desired manner. On the grounds of past experience, an inspection of the construction, maintenance and modification of the drinking water installation is indispensable.

In various countries the company itself exercises supervision of the installations that are connected to its mains system. This does not mean that with this supervision, the responsibility of the house-owner c.q. the consumer is taken over by the water company! In view of the know-how the water company has of the

FIGURE 6. Categories of premises to be converted, and safeguards to be adopted in connections to mains networks.

Category	Description of the premises	Manner of safeguarding	Minimal interference of w. company		Remarks
			Before and after construction	During use	
A	Dwellings.	Aim at placing a non-return valve at the end of the service pipe, installation and maintenance by the water company.	Advice and testing of the drinking water installation.	No periodic inspection of dr. water installation. Periodic change of non-return valve. Connection of dangerous apparatus must have prior approval. Normally no periodic test of non-return valve at end of service pipe.	
B	Small undertakings and institutions which, as to the danger and the chance of backflow into the mains network, can be put on a level with dwellings.	Non-return valve to be placed at the end of service pipe; installation and maintenance by the water company.	Advice and testing of dr. water installation.	Random spot check of installation. See further under category A.	
C	Undertakings and institutions where they do not work with dangerous matter.	See under category B.	See under category B.	Periodic inspection (once in 1 or 3 years) of installation and check of safeguards. Periodic change every 3 or 5 years of non-return valve at the end of service pipe. Every modification of the installation must be notified.	
D	Undertakings or institutions where matter is present which in case of backflow into the grid would cause danger to health of consumers.	Non-return valve to be placed at the end of the service pipe. Installation and maintenance by the company. Break (via a reservoir) near the dangerous object, or break (as near as possible to the end of the service pipe) of the whole installation.	Advice and testing of the installation.	Yearly inspection of the whole installation (both upstream and downstream of the break). Every change to the installation must be notified.	If BEFORE the break, hygienically inadmissible situations are found, modification of such situations can be forced by the w. company (in the last extremity the supply can be turned off). If AFTER the break hygienically inadmissible situations are found, advice will be given by the w. company. If the advice of the w. company is not followed, the Labour Inspectorate (e.g. with factories and the like) or the competent Health Officer (e.g. in hospitals, etc.) must be informed.
E	Undertaking or institution where inspection of the drinking water installation or part of it by the water company is in fact impractical, e.g. extraordinary complexes of which it is to be expected that frequent changes will be introduced without forewarning to the water company.	Non-return valve at the end of the service pipe, installation and maintenance by the water company. Break (via a reservoir) before the unverifiable part of the break (as nearly as possible at the end of the service pipe of the whole installation).	Advice and testing of the installation in co-operation with the technical service of the undertaking concerned.	Yearly inspection of the directly connected part of the installation. Every change of the installation before the break has to be notified in advance.	To the undertaking in question and eventually to the Labour inspector or the Health Inspection, a communication must be made that the installation has not been inspected by the water company. In addition the water company must be willing to give advice on special problems.

quality and the quantity of the water supplied by it, this supervision is considered to be of social importance. In most countries the state supervision of public health is in any case—with or without the inspection by the water company—equipped to intervene in cases of unpermitted situations. The water company can and will co-operate in similar situations by turning off the supply to the premises in question.

Besides making inspections we have to find an answer to the question in what measure, and at what periods the inspection of a drinking water installation has to take place. Here, too, a relationship has to be found with the possibility of dangers to public health. About this last aspect a growing concern exists owing to:

(a) the many new appliances placed upon the market some of which are not altogether undangerous from the point of hygiene;
(b) increasing do-it-yourself activities by non-experts that we have to accept;
(c) internal installation activities by the technical services of large industries and institutions, e.g. factories, laboratories and hospitals.

In the Netherlands detailed advice was given in 1975 by the Union of Proprietors of Water companies to their members with regard to this matter. This advice is set down in fig. 6.

In this advice the premises connected are subdivided into some 5 categories, depending on the nature of the possible danger from backflow into the mains system. For each category the means of safeguarding the mains network together with the measure of the inspection of the drinking water installation has been indicated. In the case of a higher class of hazard the premises have to be visited more often.

In practice one has observed that, for example, yearly supervision of a "dangerous" connection is of real interest. A continual flow of information to those concerned about dangerous situations is an absolute necessity with this work. A good relationship has developed with the technical services of those undertakings or industries which formerly carried out incorrect installations. For the sake of completeness one should observe that the advice given in fig. 6 has to be seen as minimum conditions in the Netherlands.

Recapitulation

In the last decade an increasing interest has been observed in the construction and maintenance of drinking water installations.

For the construction of good, and for sanitary reasons, safe water installations one has to comply with the following conditions:

(a) to have available good instructions for the construction and maintenance of drinking water installations, which refer both to the quantity and the quality of the water that has to be supplied;
(b) the construction of the installations has to be carried out by skilled staff;
(c) drinking water installations have to be connected in such a way that the chance that they will influence adversely the quality of the water to be supplied is reduced to a minimum;
(d) the construction of domestic appliances should be such as to be fitted by preference with an "internal" safety device;
(e) the supervision of the construction and the modification of drinking water installations must take place in a measure and with a frequency directly related to the possible dangers that may appear in the installation;
(f) the safeguards c.q. protected appliances should be tested preferably by a central inspection service.

The demands judged necessary for it should preferably be organised on international lines.

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Résumé

Depuis le congrès de Barcelone, où M. G. J. Angeli Sr. P.E. a traité le sujet "Protection des réseaux de distribution d'eau contre la pollution par retour d'eau", environ 10 ans se sont écoulés. On a constaté pendant cette période un intérêt croissant pour les installations d'eau potable.

Les spécialistes anglais qui ont étudié les dangers qui menacent la qualité de l'eau potable par les appareils d'utilisation sont parvenus à des conclusions intéressantes.

Les dangers éventuels qui peuvent menacer non seulement l'eau potable dans l'installation, mais aussi directement le réseau de distribution lui-même sont subdivisés en trois classes de danger, indépendamment des conséquences possibles qui peuvent résulter des joints défectueux.

Un aperçu a été donné des protections possibles qui peuvent être appliquées pour les différentes classes de dangers. Le rapport traite plus amplement les expériences aux Pays-Bas avec les nouveaux types de clapets de non retour hygiéniques qui peuvent être utilisés en combinaison avec le compteur d'eau. Il recommande de munir les appareils qui doivent être reliés aux installations d'eau potable d'une protection interne, dont il signale spécialement les développements favorables en Allemagne.

Il plaide pour une solution internationale de ce problème. La responsabilité de la qualité de l'eau qui est livrée au robinet, doit être divisée entre le service d'eau (jusqu'au point de livraison) et le propriétaire c'est-à-dire le consommateur (installation intérieure).

Malgré de bonnes prescriptions pour la construction et l'entretien des installations d'eau potable et malgré la bonne formation du personnel qui doit être chargé de ce travail, une instance d'inspection est nécessaire. D'après un exemple, on indique plus amplement sur quel rythme le contrôle doit avoir lieu.

En outre la liaison entre les responsables de cette inspection et la direction des services d'entretien des établissements industriels est importante.

The planning, organisation and control of an integrated and fully comprehensive public relations department in a water authority

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1 Introduction

The Water Act, which received the Royal Assent in July 1973, revolutionised the water industry in England and Wales. The Act set up nine regional water authorities and a Welsh National Water Development Authority to take responsibility for all aspects of the management of the water cycle. The new authorities' boundaries were broadly based on river catchments but because of historical organisation of sewerage through local authorities and the way pipes run and water flows, each regional water authority tended to find itself with a slightly different geographical boundary, on the margin, for each of its various functions. Generally speaking this was not an operational problem but was a problem in communication to the general public and was an internal problem to Public Relations and information services particularly in the early days of re-organisation since so often it was in these marginal areas that problems affecting to the individual consumer tended to arise.

1.1 The historical position

To understand the problems faced in all aspects of the operations of the authority and particularly in the early days of the public relations departments, a brief outline of the previous situation is necessary.

1.2 Water supply

Prior to the 1973 Water Act, public water supplies were provided by water companies (set up usually by Act of Parliament with statutory limits on charges and stock dividends), by Joint Water Boards which were consortia of local authorities and others, individual local authorities who formed a significant proportion, and some Bulk Supply Boards. These latter were responsible for the collection, impounding, treatment and bulk distribution of water to individual constituent authorities. Unlike water companies, Joint Boards and local authorities they were not involved in direct distribution to consumers. Under the 1973 Act water companies and a number of bulk supply boards retained independence of operation although subject to certain overall constraints from the water authorities in whose areas they operated.

1.3 Sewerage and sewage treatment

Responsibility for sewerage and sewage treatment, before the 1973 Act came into operation on 1 April 1974, rested with 1366 local authorities and 27 joint sewerage boards. Between them they provided main drainage for about 94% of the population of England and Wales. It was the fragmentation of this part of the water cycle, the limited resources of smaller undertakings coupled with greater awareness of the amenity aspects of watercourses and the increased need for treated sewage effluent to meet water supply demands which were potent forces behind the 1973 Act. But whilst the Act made Regional Water Authorities responsible for sewage treatment the operation and maintenance of the local sewerage system was to be carried out by district councils acting as agents of the water authorities.

1.4 Land drainage and flood prevention

Land drainage legislation dates from the reign of Henry VIII. The 1973 Act transferred land drainage as well as the responsibility for rivers and water resource management, abstraction licensing and pollution control of the former 28 river authorities to the new water authorities. However, Internal Drainage Boards, set up under the 1930 Land Drainage Act for districts capable of deriving special benefit from land drainage, remained as anomalies, with the right to levy independent rates, subject to a water authority watching brief.

1.5 Fisheries and recreational use of water

The former river authorities had a responsibility for the protection and development of fisheries. This—together with an increased emphasis, in fact a statutory responsibility, to develop water based recreation—was transferred to the new water authorities. In some cases water authorities had responsibility for navigation on their water courses. But a further anomaly remained in that the British Waterways Board continued to function as a completely separate entity, responsible for its extensive network of canals, an increasing recreational facility, and also for navigation on many principal watercourses. Water Authorities were to control only the quality of effluents discharged to British Waterways waters. (The Government Consultative paper on the re-organisation of the water industry at present under discussion, proposes an integration of British Waterways Board functions with the rest of the industry.)

1.6 Finance

To set the scene finally, some indication should be given of the change in the methods of financing the water industry brought about by the 1973 Water Act and particularly their impact on the general public. The Act required water authorities to break even taking one year with another. Minor grants for rural sewerage schemes and land drainage were still to be available but would make up probably less than one per cent of the total revenue. Other revenue had to be raised from consumer charges, and capital only initially from the National Loans Fund at interest rates for a 25 year period considerably above those available to local authorities raising their own funds on the open market.

1.7 Billing

Water companies continued to bill their customers direct, as did certain sections of water authorities where direct billing for water supply had been the practice of the predecessor authority. But for the most part local authorities act as collection agents for water authorities for both water supply and sewerage charges (more properly general services charges since an element—less than three per cent in general—was included for amenity, pollution control and other water authority functions). Consumers were charged for these items on their rates bill. Although collection agents were reimbursed for this service, it was in the first year virtually impossible and is currently difficult to persuade them to include adequate explanatory information on water authorities' charges with their own rate demands. This is, to some extent, due to inflexibility in computer format rate demands.

Apart from economies which seem likely from direct control of billing, and the improved consumer contact and information it will engender, the public relations benefits are obvious and regional water authorities are moving towards direct billing phased over varying periods of up to five years.

1.8 Planning the new organisation

The foregoing paragraphs have attempted to outline the variety of functions taken over by the new water authorities and the multiplicity of predecessor organisations formerly responsible for those functions. In addition some of the anomalies retained under the 1973 legislation have been indicated as a pointer to some of the problems of communication which the water authorities took over.

Against this background it did not need great foresight for the Ogden Committee (a committee set up by the Secretary of State to consider and advise on forms of management structure in the new water authorities) to forecast that there would "need to be a considerable degree of effort put into the public relations operation in the very early stages." Quite properly the Ogden committee also suggested that public relations was a function that Chief Executives might well wish to have under their direct supervision because it "is a function which is not only sensitive and often requiring very speedy action across all departments but which equally provides the Chief Executive with a broad picture of the impact of the authority on the public." In spite of this recognition, however, it was 1 April 1974—the day on which the new authorities came into being—before the first regional water authority Public Relations Officer (by whatever name) was in post. 1 April 1974 was also the date on which the re-organisation of local government in England and Wales was made effective. The contrast between the local government re-organisation and the water industry re-organisation was that the former had two years to get under way; the latter a mere eight months. It was not surprising, therefore, that water authority chairmen and their management teams were principally concerned with operational arrangements to maintain the ongoing state of the vital public services of wholesome water supply and sewage treatment. All credit must be given that within three months from 1 April 1974 all water authorities had a PRO in post. Nevertheless it would not be proper in a paper of this nature if it is to be of use to other members of the water industry faced with major or minor re-organisation to omit a reference to the importance of appointing a PRO and setting in hand a public information campaign well in advance of the actual date of implementation of that re-organisation. In so far as the multifunctional water authority is unique to England and Wales, it must be recognised that it is essential that the "Planning" part of the title of this paper should refer to activities which should ideally take place before the official take-over date of the water authority.

For the reasons previously outlined this did not happen. Hindsight is a malleable quality: but an informed interpretation of the experience of others should not be lightly discarded. For this reason the advance measures which should be undertaken to prepare the public for re-organisation are detailed.

1.9 Preliminary public relations activities

The preliminary public relations described here are the minimum that can be undertaken by a qualified public relations officer with secretarial and clerical assistance. The primary requirement before a sensible message can be relayed to the public is good information and statistics. Unfortunately in the re-organisation of the water industry in England and Wales and probably in any other re-organisation requiring integration of a large number of discrete organisations such information will be at a premium. Six months or more after re-organisation water authorities in England and Wales were still discovering rural sewage works for which they were reponsible but of which they had no record.

On the other hand, all aspects of the water industry present good visual material and copy which can be made appealing to the man in the street via the media. The local daily press (morning and evening) are always ready to take information in advance which will affect their readers in the future. Weekly press are always ready to take features. Local radio appreciate contact with local voices and TV stations will take good visual subjects. This type of campaign, provided internal information and co-operation is available, can give good coverage without significant costs. Experience in the Severn-Trent area has shown that appropriately presented press releases will be used practically verbatim by local weeklies and well used by dailies. These involve marginal costs other than postage and administration.

Advertising has been used as a means of acquainting the public with the changes in the organisation of the water industry. Except where there is a statutory requirement to advertise such matters as reservoir inspections under public notices (which by and large the general public does not read) this form of publicity is questionable when used by a monopoly supplier. Experience in England has shown that it can arouse serious criticism from consumers for unwise spending of public money. As always there is a chance that the vociferous minority attract more notice than their numbers warrant but the fact remains that good public information which is properly presented, not obviously angled, and of interest to their readers, will usually find space in local papers and on local radio free of charge.

Information sheets with basic data and key addresses sent to organisations and individuals (including press representatives), particularly if they can be supported by maps, can be produced fairly cheaply internally and if circulated sufficiently in advance of the event can not only provide secondary sources of accurate information but may also avoid the need to answer one-off inquiries from people such as MPs and others influential in public life.

These are the minimum steps which need to be taken as a preliminary to informing the public. In the course of taking these steps valuable information about the nature of the authority's publics, and the responsiveness and penetration of the media will be gained. This will enable sensible decisions to be taken about the type of Public Relations Organisation required to meet the needs of a water authority in its particular area.

In this context it is relevant to note that the PRO himself needs time to familiarize himself with the organisation and technology of the water industry—unless the water authority is able to appoint a public relations officer who is also familiar with the industry. In the recent re-organisation only the National Water Council had this good fortune, all regional PROs coming from outside the industry—though, in a number of cases with considerable experience of public authorities. Whilst it is always better to appoint a professional with public relations skills (who will rapidly acquire the necessary technical and other knowledge of the industry) than to appoint a man experienced in the industry without the professional PR skills this is a further argument for early appointment.

2 Setting up the organisation

2.1 Purpose of public relations

Before an attempt can be made to set up a public relations department, it is necessary to consider the purpose of such a department in a water authority. The Institute of Public Relations defines public relations as "the deliberate planned and sustained effort to establish and maintain mutual understanding between an organisation and its public." In a public monopoly such as a water authority, this understanding is essential to obtaining the "licence to operate" through public acceptance and consent.

2.2 Public relations in local authorities

The National and Local Government Officers' Association has been campaigning since 1932 for local authorities to undertake positive public relations policies. Since 1948 councils have had statutory powers to spend money from the rates on public relations, but until recently few have established public relations departments usually under the coy title of "information" and significantly the activities of these departments, where they do exist, are among the first to be axed in the current economy drive.

It is not perhaps public relations as such which is resisted so strongly but rather organised, or professional public relations. This resistance springs from the conviction that public relations needs neither men nor money but is a natural thing based on the divine right of councillors to know the needs of their constituents and their ability to supply them and the press with the appropriate information at what in the judgement of the elected representatives is the appropriate time. Some chief officers resent the introduction of a public relations expert who, they fear, might undermine their authority, or by stimulating criticism as well as interest add to their difficulties.

There is a large element of local authority thinking in the new water authorities. Many of their staff were formerly local authority employees and the majority of the members of each authority are local councillors deputed to represent one or more local authority. Thus it is not surprising that in spite of the high priorities put on public relations by the Ogden committee it is true to say that Public Relations Officers in most water authorities have had to fight to obtain meagre resources and with one or two exceptions, whilst they may report to the highest level officer, usually the Chief Executive, are not normally part of the seminal stage of deliberations which will affect subsequent policy. Lip service has been paid to the concept of the importance of public relations but real acceptance in many cases is still far to seek.

2.3 Analysis of water authority publics

Before the public relations strategy of a water authority can be planned, and therefore the organisation to carry it out, it is necessary to examine the various publics with which a water authority is confronted.

There is first and obviously the general public in their roles as—

- consumers
- ratepayers
- fishermen, sailors, walkers, naturalists, etc.
- and the people who live next door to a smelly sewage works.

There are water authority workers at all levels in their roles as—

- employees and members of a team
- public relations officers for the water authority.

They are the people who have most contact and probably much of the more influential contact with other publics of the authority. This role is crucial as there is no more damning publicity than bad publicity that comes from the man inside, whatever his level or the true extent of his knowledge and understanding.

Members of the authority function in addition to their role as representatives of the public as policy makers of the authority and also PROs for the authority.

Local authority members and officers are an important public. Operationally their activities as agents for sewerage work provide an interface and the water authority is concerned with most aspects of their planning. In addition local authorities control a number of information outlets from the back of the rate demand note to library noticeboards, schools, colleges of further education and careers advisory services.

Industry forms another public in its roles as—

- buyers and abstractors of water—who need our help to economise dischargers of effluent—over whom we exercise a policing function
- suppliers of goods and services.

The agricultural and horticultural publics similarly have roles as—

- consumers or abstractors of water
- dischargers of effluent
- recipients of land drainage work.

This list is not exhaustive but it indicates the wide range of roles of the people a water authority needs to inform and influence at various times.

2.4 Allocating priorities and determining means

The importance of various public in their different roles varies from time to time and geographically. Local farmers and naturalists may be of prime importance in a particular place whilst land drainage work is being carried out. In this instance resources will need to be focussed particularly on a relatively small number of individuals or pressure groups. This need to concentrate on a small area applies over all the water authority's activities and publics from time to time but there are certain basic functions which must have a continuing and consistent standard of service and the level of resources to meet this basic demand will form the base line for the total public relations resource requirement.

2.5 Press relations

Morning, evening, weekly, local, national newspapers, radio and television are the most vital tools in the public relations officer's kit. Used properly they can give the widest coverage of all publics at the lowest cost. Whilst many of the techniques of public relations can be "bought in" e.g. design, photography, some writing, to have effective relations with these media, the public relations department must be staffed to enable all enquiries emanating from them to be dealt with directly, either by PR staff or by controlled reference to the appropriate officer of the authority. A relationship with representatives of the media based on prompt response to their queries can only be fruitful in the long run. The new water authorities at their inception in a time of rapid inflation compelled to raise their charges significantly took a hammering from a press not fully conversant with the implications of the 1973 Act and also through the newly re-organised local authorities, with similar problems, diverting criticism away from themselves. It is probably fair to say now that it is largely through the ability and determination of water authority PROs to respond to the barrage of press calls which hit them from the moment each newly appointed PRO walked into his office that there has now developed a considerably greater understanding of the problems of water authorities.

Responding to press queries is the defensive part of press relations—but if this part is inadequately performed the rest can be a waste of time. The positive approach is through the production of press releases, liaison with editors, feature writers, producers on ideas for features and programmes, and however overwhelming the workload time has to be made for this side of the work—even if most of it has to be done in the evenings.

One of the water authority PRO's first tasks must be to arrange meetings between local press representatives and their local authority managers or contacts.

Effective press relations require certain minimum staff and hardware. Good press releases are timely and newsworthy; it follows that the public relations department must have access, with a high priority on use when required, to some form of duplicator. The speed and sophistication of the machinery required will depend on the number of papers, etc. within the area—as will the amount of clerical assistance required to get the release away. The numbers of newspapers, etc. regularly sent press releases by water authorities range from 50 to over 200.

Effective use of radio and television depends on having a core of members and officers who are competent to appear on these media. Experience has shown that the most senior or competent man at his job is not necessarily the best man to present the authority's point of view on radio and, even more particularly, television. Equally the man actually responsible for doing the job always carries more conviction with the public even though his performance may be less polished than that of the acknowledged paid professional spokesman—the PRO.

An early priority must be the setting up of familiarization exercises to give those likely to be called upon experience and training in the techniques of the media.

Criticism of the performances of senior people is an essential part of this exercise and one reason why outside services of experienced broadcasters may be more suitable than trying to complete the exercise in house. But the preparation of suitable briefs for both professional interviewer and water authority interviewee can take up considerable—of necessity internal—resources.

2.6 Employee relations

The importance of the contribution that good employee relations can make to the public image of a water authority and equally the contribution that professional public relations skills can make to good employee relations was early recognised by water authorities by the priority that was put on the production of a house journal. With one exception the responsibility for producing this was given to the Public Relations Officer, and the format chosen was a version of a tabloid newspaper, with the exception of one of the smallest authorities which made imaginative use of good in-house printing facilities to produce a document whose format would be as acceptable to the majority of its readers—the manual staff who account for about two-thirds of water authority staff—as would a tabloid.

The extent to which employee newspapers were produced entirely or partly in-house varied. In Severn-Trent we considered that the spin-off in overall PR terms amply justified keeping as much as possible in-house other than printing; in terms of closer contact with other employees in geographically remote locations, plus the fact that a good journalist or PR man whilst covering one story should inevitably pick up two or more other stories either for use in the house journal or to form the subject of local paper features demonstrating positive facets of the authority's activities or "human interest" stories showing that the authority is not remote but made up of individuals.

There is no doubt that contacts made in the course of covering stories for "Stream"—the Severn Trent Water Authority's newspaper—have strengthened links between headquarters and divisional working levels and facilitated the development of two way traffic in public relations.

This is particularly important in a large water authority (Severn-Trent employs over 11 000 people and covers an area over 8000 square miles) where people working on remote installations may find it difficult to identify with the large organisation. The editorial policy of "Stream" has been to give information about pensions, conditions of service and other essential matters in as readable a form as possible. There has also been a policy of reporting the policy making activities of the authority to keep our "field PROs" well informed and a bias towards articles on new works and technical and other developments in the Authority. This is not to say that sport and lighter interests have been neglected. Because of the serious nature of "Stream" content and the need for reporting informed by a close understanding of the policies, workings, and technology of the industry the paper is entirely produced in-house except for printing.

Other authorities with different objectives have used a range of external services to produce their newspapers from sub-contracting parts such as layout and printing and some feature writing to—in the case of the largest water authority—Thames—sub-contracting completely, writing, layout and production (subject to an editorial panel).

Objectives and resources will determine the methods used but if the house journal is to function as a respected organ of employee communication editorial freedom should be preserved from managerial exhortations as much as domination of space by particular interests or groups. Contributions which do not meet the standard of the publication should not be used merely because they are unsolicited.

2.7 Consumer relations

The Ogden committee put the responsibility for consumer relations squarely on the operating divisions, and there is no doubt that the manner and attitude of an individual telephonist, a water inspector, or counter clerk probably has a greater impact on the consumer than any amount of publicity material. The function of the public relations specialist is to inculcate an awareness of the public relations aspects of all the authority's functions at management level and encourage the spread of this awareness through the ranks by lectures, communications courses and awareness exercises. However no amount of communications courses or public relations effort will hide for long an operation which is not basically efficient. Public relations is not a cosmetic technique and the function of the PRO in a situation where the authority image is deteriorating through inefficiency is to draw the attention of this to management at the highest level.

Prior to re-organisation sewerage and water attracted less public notice than the more controversial local authority concerns such as housing. It must be recognised that it is easier for small local organisations to maintain good relationships with their customers on a day to day basis than it is for large regional bodies—even though many of the employees are the same people. The need for a conscious and continuing effort to promote an image is not so apparent.

The new water authorities are under close scrutiny. Their charges appear high to their consumers compared with local authority rates because the water authority charges are almost totally unsupported by central government funds whilst two-thirds of local authority expenditure comes from central funds.

The establishment of a corporate identity through the design of logos house style and livery was undertaken early on by the public relations officers. Employee competitions to design logos were tried in some authorities but experience has shown that the quickest and, in the long run, the cheapest means is to employ a good professional design consultant. The house style functions to identify the authority internally as well as externally and can have a positive effect on employee morale by giving them pride in their vehicles and uniforms. From this point of view, once designed the new style should be introduced as quickly as possible. But with a public eager to spot "unnecessary expenditure" caution has to be exercised and the compromise of introducing new colours and vehicle livery only when items are due for replacement or re-painting as part of normal maintenance must be well publicised.

The positive counter to public concern over their water authority charges is to seek every opportunity to demonstrate where their money is being spent to improve or replace services. This can be done through the press and also by large display boards on the site of works. There is, however, an internal public relations task to convince managers that this will benefit the image of the authority rather than provide a focus for increased complaints about traffic delays, mud on the road and other disruptions associated with construction work. In fact, of course, if complaints are justified they will find their way to the Divisional Manager anyway.

The power of the face to face contact with employees to influence consumers' views of a water authority has been mentioned. The planned extension of this is the meeting with such organisations as ratepayers associations, rotary clubs, women's institutes, and organised visits, and open days at the authority's works. The function of the public relations department in this respect is largely to provide back up material in the form of fact sheets and explanatory leaflets, and visual aids. A tremendous work load of addressing meetings, often initially extremely hostile over charges particularly, has been undertaken by Authority Members, and officers throughout England and Wales and rarely has a meeting ended without improved understanding on both sides. No public relations department could have scratched the surface of this task and nor should it have attempted to do so. For most of the subjects called for as topics for such meetings the local member or officer, or the technical specialist, carries far more conviction than the public relations officers.

Face to face contact is also made at exhibitions. The policy on participation in exhibitions needs to be very carefully thought out. One off static exhibitions on a particular site with a shell scheme are very expensive—recent figures suggest a cost of £3–£5 per useful contact made. This amount of money would buy a lot of alternative PR. Even if all design and fitting is contracted out they can still be costly in manning resources. Water authorities with large areas and scattered population and works may find that a mobile exhibition gives them a presence to their consumers that compensates for the remoteness of divisional or headquarters offices, and several authorities have chosen to do this. Again the initial outlay is high and provision must be made for refitting and refurbishing at least once a year, but particularly where agricultural shows are an important feature of local life such a unit may be an essential part of presenting the authority to its publics. Much cheaper than either of the other two types of exhibition is a small collapsible exhibition stand which can be packed into a car and used to back up contacts with various publics on a small local scale. Such stands can be used for careers conventions or to back up topic work at schools; in public libraries to explain about new water authority developments or to give background information. Over time a library of interchangeable panels can be built up allowing exhibits on various topics to be put together very rapidly to meet a local need.

An important aspect of a water authority's relations with its consumers is its reaction to emergencies. The development of completely multi-functional divisions in Severn-Trent has enabled information centres to be set up in each division manned round the clock 365 days a year and giving one telephone number for the public to call with the assurance that their problem will be dealt with and they will not be passed from department to department. A lecture on the public relations aspects of their job forms part of the training for all information centre staff.

Whatever the immediate crises or pressures no public relations programme can afford to ignore the consumers of the future. Educational material in the form of fact sheets on various topics can be produced cheaply and in quantity; sets of slides, films, tape slide programmes and small exhibitions can be loaned and wall charts are always an acceptable aid. More than preparing informed consumers for the future an educational programme can also get a message to present customers through their interest in the activities of their children at school.

3 Staffing and budgets

It is false economy for even the smallest water authority to attempt to cover the public relations function with a solitary public relations officer, however gifted and diligent. As the other activities of a water authority continue through 24 hours of each day of the year so does public relations, and whatever the policy with regard to making home telephone numbers of managers available to the press they should always be able to raise a member of the public relations staff. Professional public relations support is particularly important at times of change and re-organisation when the pattern for press and public reaction to the authority may be set for some time to come.

Wonders can be achieved with the assistance of a keen secretary but that cannot be guaranteed. The assistant to the PRO like the PRO himself should be in post in the planning stages of the new organisation; this is particularly important if the re-organisation is moving towards, for example, catchment areas as a basis which do not line up with previous administrative or other areas, or where there are anomalies in the re-organisation of the nature demonstrated in the first part of this paper. The public relations effort needed to put over a message increases geometrically with the complexity of the message.

This level of staffing plus in most cases secretarial or clerical assistance is the level at which the smaller authorities are now operating. It is not excessive in relation to the size and range of functions of even the smallest authority. On the other hand the public relations department should not increase out of proportion to other water authority headquarters departments. This is a base level to cope with the day to day situation and implies, of necessity, that much of the work involved in the production of print, visual aids etc. must be done by outside agencies.

In England and Wales whilst the re-organisation of the industry produced autonomous bodies the co-ordinating activities of the PR staff of the National Water Council and the willingness of individual public relations officers to make their design work, their publication and other public relations media with high initiation and design costs, available to their colleagues to amend for use in other water authorities has enabled a high standard of productions to be achieved over a wider area than if each authority had paddled its own canoe. We have not been too proud to use each other's ideas, to copy and adapt. The extent to which this facility may be available within a country—or through the auspices of the IWSA—will obviously have an impact on the resources needed to provide an adequate public relations service.

On the re-organisation of the industry it was recognised that some of the new authorities would be large enough to support a specialist library, technical information service. This has been implemented with most notable success in the National Water Council itself as part of the Information Services division. Their Bulletin is a valuable tool and essential reading for managers at all levels in all authorities. The success of the Bulletin may account for the limited impact of the information service in those authorities where it has been implemented other than as part of the public relations department. With modern techniques of tele-facsimile transmission available it is sensible to concentrate those areas of information requiring a high level of technical, legal or other defined specialisms in a central library technical information point. The short term local, rapid response aspects of information services, and the staff and resources to cover these are better organised as part of the PR activities. Public relations professional staff are used to a rapid response rate and extra resources to cover this function enables greater overall flexibility particularly in smaller authorities.

Budgets of public relations departments fall into two parts—on going expenses, chiefly salaries, telephone and other administrative costs over which little control is possible and changes can be forecast, and the budgets for specialist activities. In the early stages of a new or re-organised water authority it is advisable to be cautious. A certain number of new projects can be programmed and allocated for each year. These may be new publications, updates of existing publications, films or other media costly to initiate. With advice and guidance on inflation levels from the Finance Department, it is relatively straight-forward to control these aspects.

Policy decisions which will relate to the circumstances of individual authorities will dictate the extent to which flood, drought, or frost precaution publicity is part of the normal PR programme or should be budgeted for as a contingency sum, adequate to cover maximum publicity in the event but otherwise to be returned intact.

4 The future task

There is not much likelihood of large funds being available in the foreseeable future for public relations. The three basic areas of press, employee and consumer relations will need to be covered. In the field of press relations local managers and their staff are becoming much more conscious of the importance and value of good local press relations and many have achieved significant successes in placing local press stories and giving talks and interviews on local radio. Except for policy or authority wide matters the Regional Public Relations Departments may become less involved in local press contacts than in the past, particularly in the larger authorities. In the smaller authorities the PRO tends to be more a figure in local life and the frequency of contacts is likely to continue at the same level.

Employee relations will continue to demand a high level of attention. New legislation affecting people at work will probably call for greater public relations participation in the production of posters, guides and other aids to communication.

Consumer relations will require even greater efforts. Fundamental to success in this field must be a greater formalised PR involvement in the earliest stages of policy making and at the highest level. Assistance must be given in increasing the public relations content of management courses to bring about a situation where every manager is aware that all the activities of the authority have a public relations aspect. Their public relations specialist must either fit him by training to exploit this to the utmost benefit of the authority or in appropriate cases provide the necessary practical assistance. Improved awareness of, or facility in public relations techniques by all staff must benefit the water authority. But there will continue to be a place in the water authority for the public relations department manned by professional staff for many years to come.

The role of statistical data in Public Relations

by Dipl.-Volkswirt Harald Gundermann

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Good arguments are not sufficient

A considerable task of public relations is to afford to the public an insight into the daily work of water supply enterprises and to disclose their plans, successes and difficulties. For this reason more and more information is necessary in verbal and statistical form, to be published after selection and coordination. The increased demand for information by the public is generally met with an equivalent offer of information by water supply enterprises.

Internal and external relations cannot be kept up with empty promises. This is the bitter experience of enterprises that do not follow systematically planned public relations. As everybody knows the selected information includes text, figures and pictures.

Public utilities have good and clear arguments that can be convincingly proved and made understandable to the public. The first principle is: Public relations cannot be done "at a guess". From the very beginning of the planning phase it has to be borne in mind that verbal information and statistical data must form an inseparable unit. Statistical data serve to prove an argument right, to show a tendency or to illustrate economic or technical problems. In this way, publicity on the basis of data and facts will convey an overall view of the enterprise of the trade concerned.

In the Federal Republic of Germany generally an open approach is apparent, so that nearly all enterprises present more figures, data, facts and plans to the public about their work than would be strictly necessary.

German water companies also now like to offer more than the prescribed minimum of information. Occasionally, there is still a reserve towards publicity—even if subliminal—but it is increasingly discarded. Annual reports are being complemented by additional surveys, e.g. statistics in brief and circulars. Nowadays there are no more objections to putting at the disposal of a wider public extensive surveys or summaries, which will afford an insight into methods, forthcoming decisions, and the total efficiency of the enterprise. Today the public is more than ever interested in the proceedings of the general economy and its correlations as well as in the events of a single enterprise.

The publication of numerically proved information about the activities of an enterprise helps to allay the lingering public suspicion that the company wants or even has to hide something. In other words: Enterprises which are not forthcoming will very often be underestimated in their importance. The public is inclined to associate the importance and reputation of a water supply enterprise with the size of its undertakings and the quality of the drinking water.

Statement of the responsibility of the water industry

Under the prevailing conditions public utilities can help the political economists and experts to obtain a sufficiently differentiated picture of their own commercial situation or the general economic trend by publishing their successes or even failures in words and figures. Actually, clear statements and conclusions from all public utilities are not only being expected within the company but throughout the entire water industry. This responsibility towards the public cannot be evaded. How else than by real figures and clear descriptions can the public be properly informed that e.g. water supply enterprises situated in overcrowded regions, with commitments to industry, experienced considerable decreases in demand in 1975, exclusively due to the general economic situation. This shows also how closely economic conditions and the industrial water demand are connected. This obviously difficult problem, however, has to be explained to the public and that not only by a certain approach but also by a deliberate and concentrated effort to inform.

The aim is for the water supply enterprise to present the many different aspects of its own importance within the industry, the region and the community.

Supply enterprises publishing important information are regarded as progressive. The public expects clear explanations of concrete problems, e.g. the necessity of formation of reserves for the preservation of capital. People do not believe any more that even modern enterprises have no difficulties in certain fields.

The open and even self-critical publication contributes more to the reputation of an enterprise than a general rose-coloured picture.

Balanced ratio between text and figures

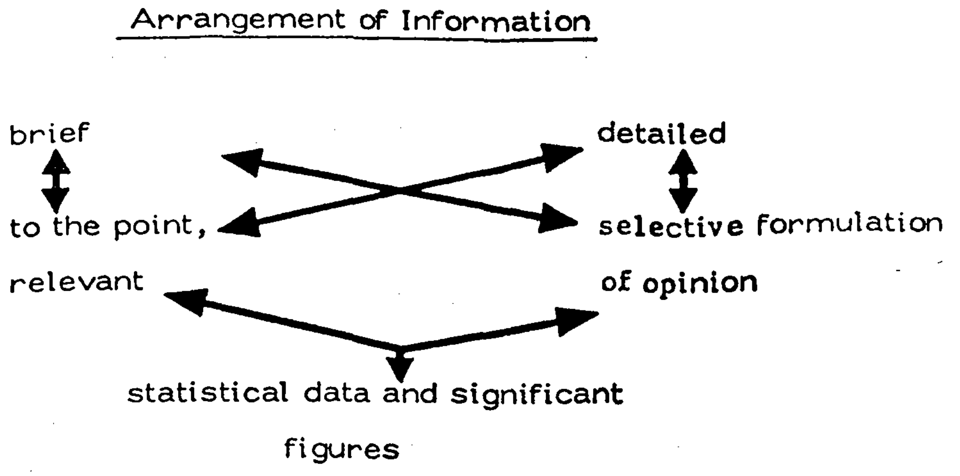
The standard of efficiency of a water supply enterprise should be expressed generally by carefully composed texts which are informative and easily comprehensible. The texts should be brief, to the point and confined to essentials. In order to break up the appearance of the text and as a proof for the arguments, statistical data or graphical representations, charts and pictures should be placed in such a way that a pleasant variety will be obtained and the reader's interest maintained.

It is known today that information without accurate figures is like a piece of apple pie without cream. As everybody knows, nothing is less appetizing for the public than a dry mouthful of platitudes.

Before carrying out a systematic and planned P.R. campaign one recurrent question arises for the team: "How can I pass on to the public as intelligibly and as informatively as possible particular information about a project, a special achievement or even an increase in prices?"

One fundamental rule has to be applied: As much information as possible, as much statistical data and pictures as necessary. As everybody knows, there are many possibilities from which one has to be chosen for the purpose of publication:

It is irresponsible to suppose that one could sell to the public such a many faceted undertaking as water supply exclusively by global statements and expect the P.R. officer to pass on to the public bare information in the form of assertions.



Occasionally it can be observed that water supply enterprises expect fair reports from the mass media in which they themselves have a lack of confidence. In general the courage of realistic self-presentation by selective information based on a background of real figures will however be rewarded.

The aim will not be reached if the public is overfed with statistical or technical data, e.g. on the occasion of the opening of a new waterworks, a pipe-line or a special installation. In this case the danger arises that the public simply will not understand the information. If a "grave yard of figures" consisting of statements, developments of trends or similar facts is offered to the public, one will certainly lose the chance of winning their understanding and confidence. Only a sensible combination of statistical data and verbal statements will bring the looked-for success.

There is no recipe and no scientific method for obtaining this, but delicacy of feeling, intuition and last but not least the "eye" for the essential are important.

On specific occasions the P.R. officer has the responsible task of preparing a publication after co-ordination with the technical department. A conflict of interest between P.R. officers and technicians, often referred to as "empire building" by one department or the other or unnecessary duplication of work should not be allowed for the sake of the project as a whole.

Publicity and policy-making

Nowadays it can justifiably be said that P.R. productions, well balanced as to text and figures as well as graphics and pictures, represent an important medium of policy-making.

In this connection internal communications (human relations) is as important as external ones. The staff-members of the enterprise have as much right to be informed as business-associates and share-holders and the general public. It is essential to report social benefits, the development and the organization of the social system in the company and the endeavours of the management to maintain or create new employment. The staff-members should be informed from time to time about the improvements in conditions by convincing time-comparisons and relative values.

An international committee has been confronted with the important question "how" and "how much" in public relations. This committee was concerned with the creation of comparable international water statistics and the corresponding evaluation of them by considering specific facts given by countries participating.

I should like to take the opportunity of presenting to you the first results of the work of that committee in which water-economical organizations of the following European countries cooperated: Belgium, Germany, Denmark, France, Great-Britain, Italy, Luxembourg, Netherlands, Austria, Sweden, Switzerland and Spain. The task was to find out and show the existing standards of water supply in 12 European countries in spite of different circumstances and conditions. This undertaking was not easy, and right from the beginning the committee was aware of the considerable difficulties which have, however, been overcome in the interest of the subject. You have certainly learned from your own experience that it is already problematic to compare the water supply of one region with another and draw the same conclusions for all of them.

The result of this work has been published in the brochure "International water statistics 1968–1972". At a first evaluation of this work it can be stated that the committee has succeeded in creating with small financial means a harmonious picture of water supply in the afore-mentioned countries by verbal texts, pictures, tables and diagrams. This work represents a valuable survey of the development of water supply in the last 5 years.

In this case the original idea was realized to show the development of single countries under a unifying theme by text and pictures and to illustrate this by technical data. The authors of the texts had the opportunity to point out the special problems of their own countries. The verbal text are followed by clear statistical data, comprising not only the extent of supply, sources and quality of water, but also water supply according to groups of customers, average consumption, consumer prices and peak consumption per hour, etc.

This was certainly a difficult but nevertheless successful beginning, and it can be assumed that this work will be continued, completed and improved. Obviously, this task was very challenging and in retrospect it can be stated that it was worth carrying out such an undertaking. At the same time it was proved that verbal presentations and the critical valuation of performances, successes and difficulties as well as the publishing of further plans are definitely of great value.

If Public Relations—within enterprises, regions and associations—are headlined by the following saying of Abraham Lincoln, success will not fail to appear: "Public opinion is all, without it there will never be a success in the long run, but with it, success will not fail to appear."

Public Relations Campaign by the public water supply service for improving the metering of detergents

by Dr. Klaus Zwintzscher

(Federal Association of the German Gas and Water Supply Service)

In 1975 the Federal Association of the German Gas and Water Supply Service launched a PR campaign to help ensure that only so much detergent was used as was essential for a clean wash. In the interests of protecting water and preventing its eutrophy it is urgently necessary that fewer phosphates should be washed into water courses. Phosphates, however, are contained in most detergents in order to make the water less hard. This problem was tackled in 1975 by the law on the tolerability by the environment of detergents and cleaning products. In addition to regulations on the publication of the substances contained in detergents and cleaning products and the possibility of banning or limiting the use of certain detergents and cleaning products, the law also contains

—the obligation for the detergent industry to print on its packets of detergent directions for metering the quantity of detergent used, graded according to the degree of hardness of the water, and

—the obligation for the water companies to publish in an appropriate way the degree of hardness of the water in each supply area.

The purpose of the PR campaign prepared by the Federal Association was to publish the information required by law so that if possible every customer is reached. To this end a working party of the technical committee "PR-Wasser" (PR-Water) of the Federal Association made the following proposals:

1 The sticker "Accurately metered—Money saved"

The campaign's most important task was to inform the housewife of the hardness of the water in each supply area, so that she has the information in front of her when metering the amount of detergent to be used, and this information is always at hand. These requirements are perfectly met by a sticker which gives the necessary information and can be affixed to the washing machine next to the opening for pouring in the detergent. The slogan printed on the sticker places less emphasis on the beneficial effects on the environment of an accurate metering of the detergent, and more on the financial saving effected. Thus the slogan "Accurately metered—Money saved" fundamentally opposes the view that no harm is done if half a cupful more detergent is used than is necessary. At the same time the sticker was so designed that it could also display a slogan commenting favourably on the water company. Each company could choose this slogan for itself.

2 Poster

In order to inform the public of the campaign in addition to the sending of stickers, a poster was designed which shows a large picture of the sticker and, underneath, reflects the slogan "Accurately metered—Money saved". In individual cases some appropriate references to the company concerned were to be added to the poster.

3 Press information

As it is advisable to inform the local press when conducting such a campaign in each supply area, the "PR Water" committee prepared press notices to which details about the supply area in question had to be added. Two press notices were devoted specially to the campaign. Another notice was also sent as a precaution, in order to help companies which supply hard water and are obliged to explain to the population the advantages and disadvantages of hard water. The advantage of this press service offered by the Association is that it provides a fairly uniform message from the public water supply service to the people.

4 Conduct of the campaign

The campaign was not conducted by the Federal Association of the German Gas and Water Supply Service directly, but by the individual supply companies under their own authority. The Association made available to the companies not only the materials but also instructions for the circulation of the stickers and the use of the posters. The costs of the campaign were borne by each individual water company.

The campaign was published within the Association by two news bulletins with appropriate appendices. The Association's headquarters also informed the competent ministries in the Federal *Länder*, which in their turn informed the authorities under them about the campaign by ministerial news-sheets or in other ways.

The water companies expressed keen interest in the campaign, and over 8 million stickers have already been distributed to householders. The campaign was also welcomed by the ministries and consumers' associations, which regard this campaign as an altruistic measure serving the interests of customers as well as those of protection of the environment.

Bonn, 22nd April 1976

Dr. Zw/Rü—BGW—

Projet d'installations de dessalement de grande capacité par compression mécanique de vapeur

par J. Franquin

SIDEM

1 Divers procédés de dessalement

1.1 Généralités

De nombreux procédés peuvent être utilisés pour le dessalement des eaux de mer et des eaux saumâtres. Ils sont basés sur des principes qui se répartissent en trois catégories:

—Procédés agissant sur des liaisons chimiques (échanges d'ions, formation d'hydration, extraction par solvants)

—Procédés utilisant des membranes (electrodialyse, osmose inverse)

—Procédés utilisant un changement de phase (congélation ou distillation)

Les deux premières catégories de procédés font l'objet d'études de principe, leurs développements ont été jusqu'ici très limités. On ne peut, dans un avenir proche, prévoir leur utilisation pratique que dans des cas très particuliers (eau de faible salinité, très faible production...) La distillation a au contraire, et depuis de nombreuses années, débouché sur des réalisations pratiques très nombreuses. C'est le seul procédé utilisé pour des installations de grande capacité.

1.2 Dessalement par distillation

La chaleur de vaporisation d'un kilogramme d'eau est d'environ 550 Kcal. Le coût de cette énergie rendrait économiquement inacceptable le procédé de distillation dans les cas où l'on ne dispose pas de source de chaleur bon marché (eau de refroidissement d'une machine thermique par exemple) si l'on ne pouvait récupérer la chaleur de condensation de la vapeur produite. Il est possible d'imaginer une très grande variété de schémas thermiques pour mettre en oeuvre ce principe.

1.2.1 *Postes de Distillation à Plusieurs Effets* (Fig. 1). Dans cette solution, la source extérieure d'énergie produit de la vapeur dans une cellule dite "haute température". La vapeur produite dans cette cellule sert, en se condensant, de source de chaleur dans une deuxième cellule à pression, donc à température d'évaporation plus basse. On peut mettre ainsi plusieurs cellules en série. La quantité d'eau produite est approximativement égale à la production de la cellule initiale, multipliée par le nombre de cellules, pour un apport énergétique correspondant seulement à la vaporisation dans la première cellule.

L'étude plus précise du bilan thermique montre que la consommation spécifique est:

$$Y = k\Delta + \frac{r}{n}$$

où Y est la consommation spécifique (Kcal/kg)

k le rapport de la quantité d'eau d'appoint à la quantité d'eau distillée produite

Δ est l'écart de température dans une cellule entre l'eau de mer à la sortie du condenseur et l'eau en ébullition

r la chaleur de vaporisation de l'eau

n le nombre de cellules du poste

Des considérations pratiques de construction conduisent à limiter le nombre d'effets à 6 ou 7. La consommation spécifique ne peut donc être inférieure à environ 100 Kcal/kg.

1.2.2 *Procédé Flash* (Fig. 2). Dans le procédé flash, l'eau de mer préchauffée entre dans une chambre où la pression est maintenue inférieure à la pression de saturation correspondant à sa température. Il y a donc à l'entrée de l'eau de mer, détente instantanée qui entraîne une vaporisation brutale (le "Flash") d'une partie de l'eau de mer.

L'eau vaporisée vient se condenser sur un échangeur où circule l'eau de mer d'appoint qui se trouve ainsi préchauffée.

En pratique, l'eau de mer circule de cellule en cellule de pression de plus en plus faible. Dans chaque cellule une vaporisation se produit. L'eau de mer brute, au contraire, circule des cellules froides vers les cellules chaudes en accroissant progressivement sa température.

Entre la sortie du condenseur de la cellule la plus chaude et son admission dans la même cellule pour "flasher", l'eau de mer est réchauffée par une source d'énergie extérieure pour donner la différence de température voulue entre l'eau qui "flash" et l'eau dans le condenseur.

Le bilan thermique est tel que la consommation spécifique est:

$$Y = \left(\frac{r_m}{n\ dt} + \frac{1}{2}\right) C$$

où C est l'échauffement dans le réchauffeur

r_m est la chaleur moyenne de vaporisation

n est le nombre d'étages

dt est la chute de température par étage

Les calculs montrent que les surfaces d'échange et le nombre d'étages de détente croissent très rapidement lorsque l'on désire obtenir de faibles consommations spécifiques et les réalisations actuelles conduisent à des consommations spécifiques supérieures à 50 Kcal/kg.

Ce procédé de distillation a connu une remarquable extension pour les unités de grande capacité de production. Son principal avantage est une grande simplicité de fabrication et de fonctionnement. En effet, les seuls éléments mécaniques sont les pompes de circulation d'eau.

En outre, la consommation spécifique est modérée et l'énergie nécessaire est de l'énergie à faible température (température de réchauffeur comprise entre 80°C et 120°C). Cette énergie peut être obtenue par soutirage de vapeur sur un cycle de machine thermique, ce qui, dans certains cas, peut être un avantage économique.

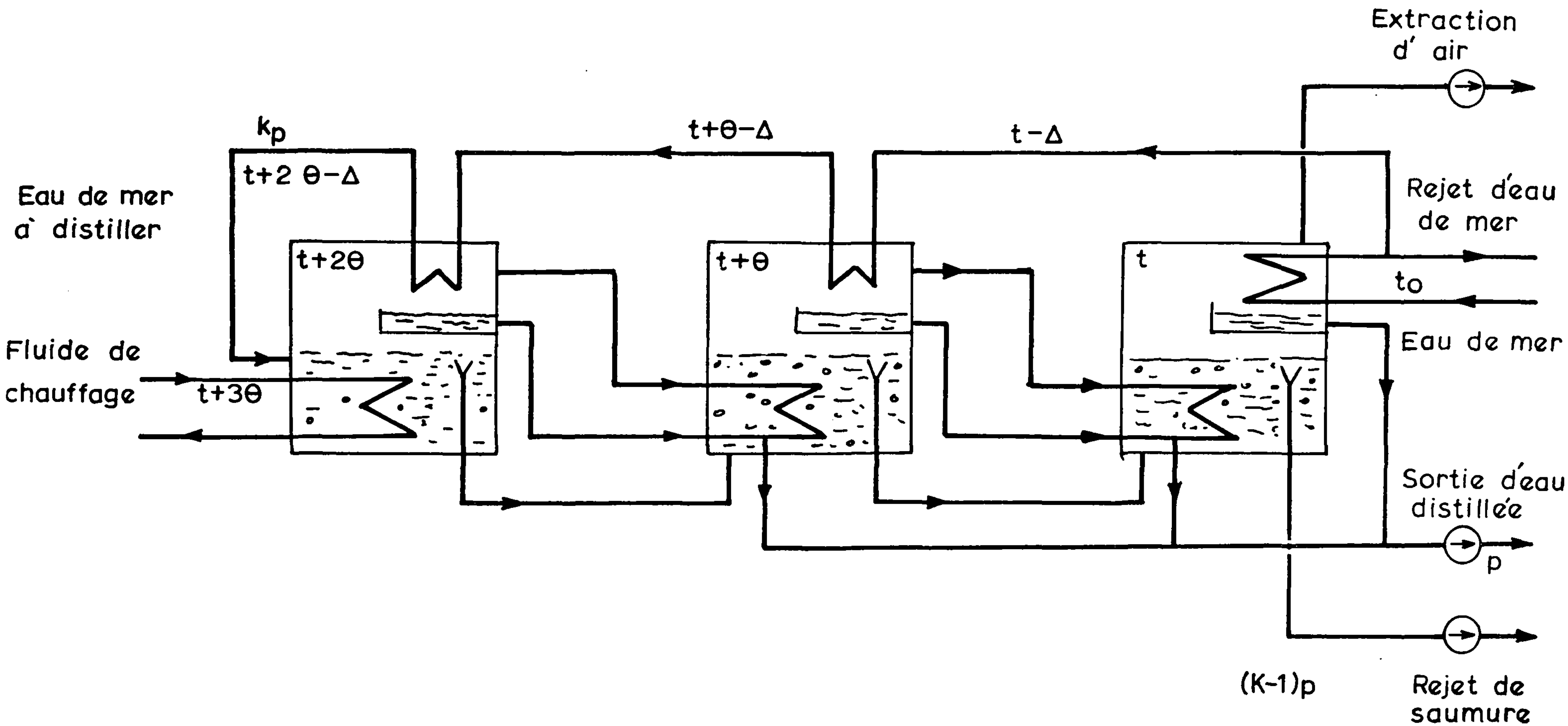


Figure 1—Poste de distillation à multiples effets.

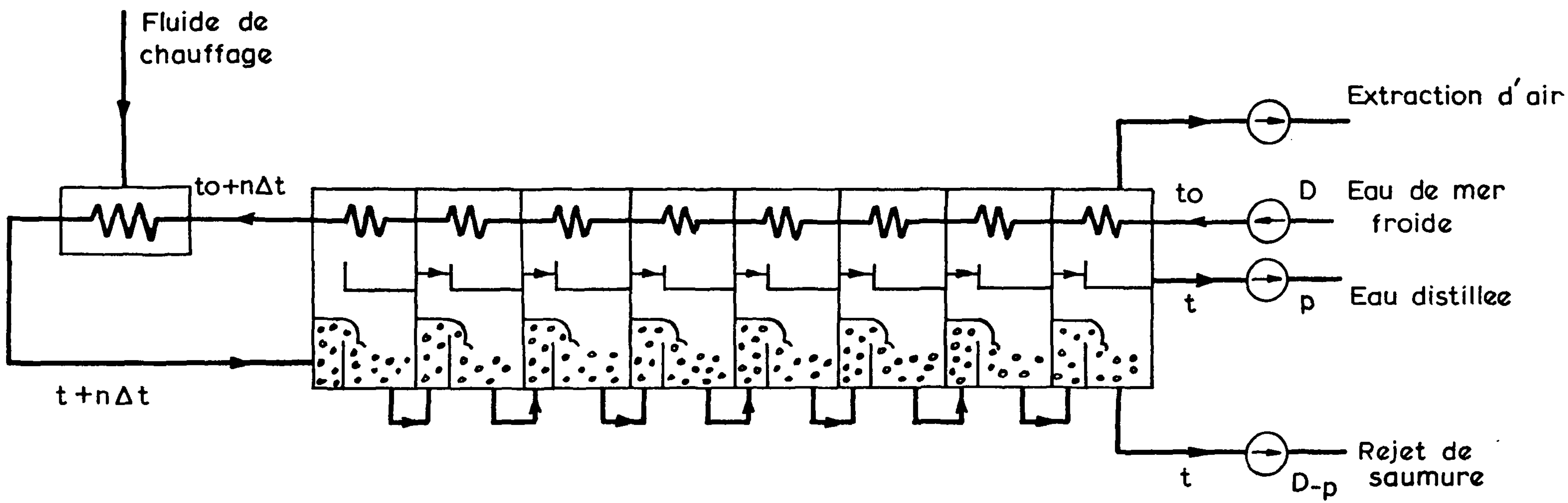


Figure 2—Poste de distillation Flash Cycle en boucle ouverte.

1.2.3 *Compression de Vapeur.* Le principe du procédé par compression de vapeur, objet du présent document, est développé ci-après.

2 Compression de vapeur

2.1 Principe (fig. 3)

Le principe de la distillation avec compression de la vapeur est le suivant:

L'eau à dessaler est portée à ébullition dans une enceinte thermiquement isolée. La vapeur produite dans l'évaporateur est aspirée par un compresseur qui accroît sa température de saturation suivant la relation qui existe entre la température d'équilibre eau-vapeur et la pression.

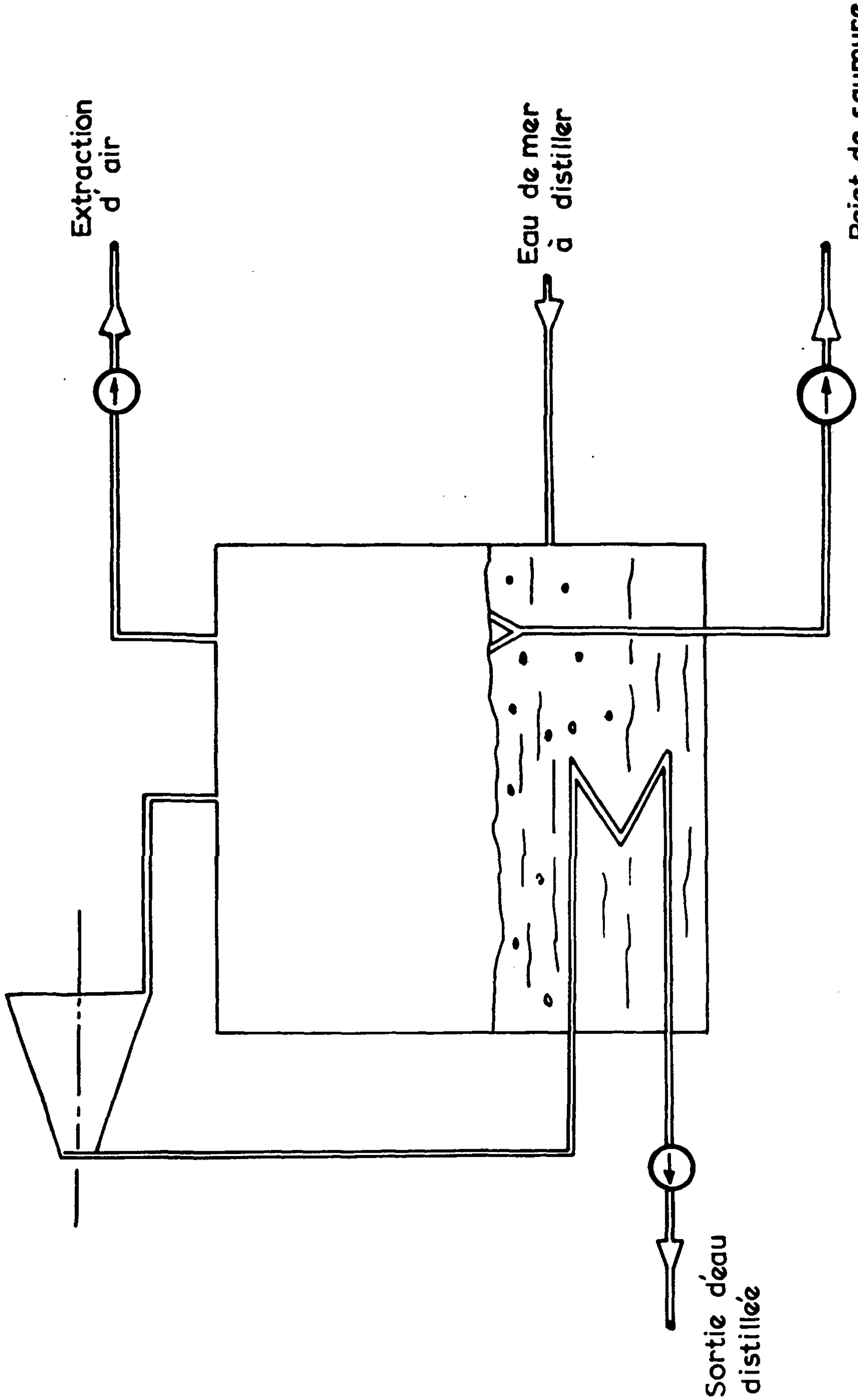


Figure 3—Poste de distillation à compression de vapeur.

La température de condensation étant plus élevée que la température d'évaporation, il peut y avoir transfert direct d'énergie de la vapeur se condensant à l'eau en ébullition. Le seul apport d'énergie théorique dans le cycle est l'énergie de compression de la vapeur.

En faisant décroître le taux de compression du compresseur, on peut théoriquement diminuer autant que l'on veut la consommation au prix d'un accroissement des surfaces d'échange (faible différence de température entre la température de condensation et la température de vaporisation).

2.2 Avantages du procédé

2.2.1 L'avantage du procédé réside essentiellement dans l'excellent rendement qu'il est possible d'obtenir du cycle.

Dans l'état actuel, il semble qu'une consommation spécifique de l'ordre de 10 kW/h par m^3 constitue, sur le plan économique, un optimum pour le prix de revient de l'eau produite.

Il s'agit bien sûr d'une énergie noble puisque cette énergie est prise sous forme mécanique.

Mais, étant donné les consommations en énergie calorifique des autres procédés, d'une part, et les rendements thermodynamiques des centrales modernes, d'autre part, l'avantage reste nettement en faveur de la "compression".

2.2.2 La simplicité du cycle thermodynamique de fonctionnement se concrétise par une simplification de la construction des appareils. En effet, il n'est plus nécessaire de prévoir de "multiples effets" de fonctionnement. Cet avantage doit se traduire par une réduction du prix de revient des fabrications.

2.2.3 Si l'installation de dessalement est l'auxiliaire d'une centrale électrique, le procédé sépare totalement le circuit thermodynamique de l'installation de dessalement de celui de la vapeur de la centrale, ce qui peut être souhaité dans certains cas (centrale nucléaire).

Enfin, le fonctionnement de l'installation de dessalement est indépendant de celui de la centrale (seule une alimentation électrique est nécessaire) ce qui simplifie les problèmes de démarrage.

2.3 Utilisations classiques du procédé

Le procédé de distillation avec compression de la vapeur est déjà très largement utilisé dans des installations de moyenne et faible capacité de production. Les différentes réalisations sont les suivantes:

—Installations de très faible capacité jusqu'à 60 m^3/j avec des compresseurs type ROOTS

—Installations de capacités jusqu'à 250 m^3/j avec des compresseurs centrifuges

Le développement de ce procédé est limité par les volumes très importants de vapeur à comprimer lorsque la température d'évaporation est basse, condition qu'il était jusqu'ici nécessaire de réaliser pour éviter l'entartrage rapide de l'évaporateur.

La thermocompression (compression par éjecteur à vapeur) permet de comprimer effectivement de très grands volumes, mais le faible rendement des éjecteurs fait perdre de leur intérêt aux installations de grande capacité utilisant ce type de compression.

La SIDEM réalise de très nombreuses installations de distillation avec thermocompression dans la gamme de production 60–500 t/j.

2.4 Nouveaux développements de la compression de la vapeur

Plusieurs éléments économiques et techniques nouveaux ravivent l'intérêt des installations de dessalement avec compression de vapeur:

2.4.1 *Problèmes Economiques et Sociaux.*

—La crise énergétique et le renchérissement du prix de l'énergie favorisent les procédés ayant les meilleurs rendements

—Le manque d'eau qui apparaît dans certains pays industrialisés souvent pauvres en énergie justifie le développement de procédés plus élaborés sur le plan technique que les installations "flash"

—La protection de l'environnement est mieux assurée avec le procédé de compression de vapeur car:

- Il y a moindre rejet de calories à la mer avec ce procédé qu'avec tout autre procédé
- La quantité d'eau chlorée passant dans l'appareil et rejetée à la mer est également beaucoup plus faible.

2.4.2 *Problèmes Techniques.* Sur le plan technique, les progrès réalisés avec les procédés de traitement pour la prévention de l'entartrage permettent d'envisager en toute sécurité une température de fonctionnement voisine de 110°C. A cette température les volumes spécifiques sont tels que l'on peut beaucoup plus facilement construire économiquement des compresseurs de vapeur pour des installations de grande capacité de production.

3 Programme de développement envisagé

3.1 Des installations de distillation avec compression de vapeur existent actuellement jusqu'à des capacités de 500 m^3/j. Au-dessus de 500 m^3/j, et jusqu'à 25 000 m^3/j, seules les installations "flash" sont pratiquement construites.

Nos études nous montrent la possibilité de couvrir dès maintenant **la totalité de la glamme jusqu'à 25 000 m^3/j par des appareils à compression de vapeur.** Pour tous ces appareils, les compresseurs seraient de simples surpresseurs centrifuges correspondant à des appareils de fabrication courante pour d'autres utilisations industrielles.

3.2 L'exploitation d'un grand nombre d'installations du type Flash, au cours de nombreuses années, a permis aux exploitants et aux constructeurs de dégager un certain nombre de règles portant sur les procédés ou sur la construction.

Ces règles servent à l'élaboration des dossiers techniques dans les appels d'offres, et leur application permet d'obtenir un fonctionnement satisfaisant des installations.

L'introduction d'un nouveau procédé pour des installations de distillation de grande taille, en concurrence avec le procédé Flash réputé bien connu, risque donc de se heurter à une certaine méfiance de nos clients.

Cette méfiance ne pourra être vaincue que lorsque la preuve aura été faite qu'une installation à compression de vapeur peut être construite et exploitée économiquement.

C'est pour effectuer cette preuve que la SIDEM envisage de construire une installation pilote d'une production de 5 000 m^3/j.

Cette capacité correspond aux besoins en eau de centrales nucléaires en bordure de mer de sorte que, après une période d'exploitation de 2 ans environ, l'installation pilote pourrait être ré-utilisée.

Subject 2

Hyperfiltration for the treatment of brackish water, especially polluted surface water

by Dr. Ir. D. Kuiper

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Introduction

Hyperfiltration (H.F.) or reverse osmosis is a process in which pressurised water is forced through a semi-permeable membrane. This membrane is a thin plastic film which rejects almost completely the dissolved compounds present in the water. In order to resist the high operating pressures (20 to 40 bar) the thin membrane is reinforced by a supporting material. At present various membrane configurations are commercially available.

The process was originally developed for desalting purposes only. However, as the membranes reject not only salts, but in addition almost every other compound present in the water, the process is an excellent potential tool for the production of high quality drinking water from polluted waters, such as surface waters and waste waters. Although this application is highly attractive, its practical operation is complicated as these polluted waters cause a fouling of the membranes, resulting in a strong decline of the production rate. This membrane fouling is the most important practical problem in the treatment of polluted waters and it determines whether a certain polluted water can be treated by hyperfiltration or not. Therefore, in practical operating conditions likely to involve membrane fouling, prevention of fouling by pretreatment as well as membrane cleaning procedures have to be considered most seriously if hyperfiltration is to be applied. In this paper these operating conditions are described in detail.

Principles

Membrane characteristics

The membrane behaviour is characterised by the salt rejection and the product water flow. The salt rejection ratio *SR* is defined as:

$$SR = \frac{C_f - C_p}{C_f} \qquad (1)$$

where C_f and C_p are the salt concentrations in the feed and product water respectively. *SR* has values between zero and unity, but is usually expressed in percentages.

The product water flow or water flux F_w is expressed as in m^3 per m^2 membrane area per day (m^3/m^2 d). Under practical conditions F_w is the in order of 1 m^3/m^2 d.

The water flux F_w is described as:

$$F_w = \frac{A.\,(\Delta p - \Delta \pi)}{\Delta x} \qquad (2)$$

where:

- A is the water permeability constant, which in turn is characteristic of the particular membrane used and depends on the membrane material and the process used to prepare the membrane.
- Δp is the applied pressure difference across the membrane.
- $\Delta \pi$ is the osmotic pressure difference across the membrane.
- Δx is the membrane thickness.

The water flux F_w is mainly governed by the factor $(\Delta p - \Delta \pi)$, which is the driving force in the membrane filtration. The osmotic pressure π is a natural phenomenon causing water to pass semi-permeable membranes from the diluted side to the more concentrated side of the membrane, thus equalising the concentrations on both sides of the membranes. The direction of the osmotic flow is opposite from the desired direction, which is from the more concentrated to the more dilute side. The osmotic pressure is, therefore, counteracting the applied pressure and consequently hyperfiltration is only possible when the applied pressure exceeds the osmotic pressure. The nominal value of the osmotic pressure is equal to the applied pressure when no water passes the membrane in either direction.

According to Van 't Hoff's law, the osmotic pressure is directly proportional to the molecular concentration of the dissolved compounds as illustrated in table 1.

TABLE 1: OSMOTIC PRESSURE OF NaCl SOLUTIONS

concentration in mg/l	mole/l	osmotic pressure atm.
35 000	0 6000	28 000
1 000	0 1700	0 800

This table shows that the osmotic pressure of brackish waters (e.g. 0,8 atm. for 1000 mg/l NaCl) is fairly low compared with the generally applied operating pressures of 20 atm. to 40 atm. For these applications the effect of the osmotic pressure can almost be neglected. This is not true where seawater is concerned, because in this case an osmotic pressure of 25 atm. has to be exceeded.

Concentration Polarisation

In hyperfiltration water is forced through a membrane and the retained solutes tend to accumulate on the membrane surface. This increased concentration on the membrane surface is called *concentration polarisation* which is a natural phenomenon of membrane filtration operations. In order to minimise concentration polarisation the feedwater is forced to flow along the membrane with a certain velocity. This is applied to all membrane filtration systems and causes a cross-flow filtration instead of a through-flow filtration, which is normally applied, for example for a rapid sand filter.

Figure 1 illustrates the concentration polarisation in a steady state. The transport of the solutes to the membrane takes place in the bulk water flux and is balanced by a diffusive flow-back into the bulk feed flow as well as by the solute flux through the membrane. A solute concentration gradient is built up in the "polarisation layer", the thin boundary layer on the membrane surface. Within this layer the solute concentration increases from C_f in the feed flow to C_m on the membrane surface.

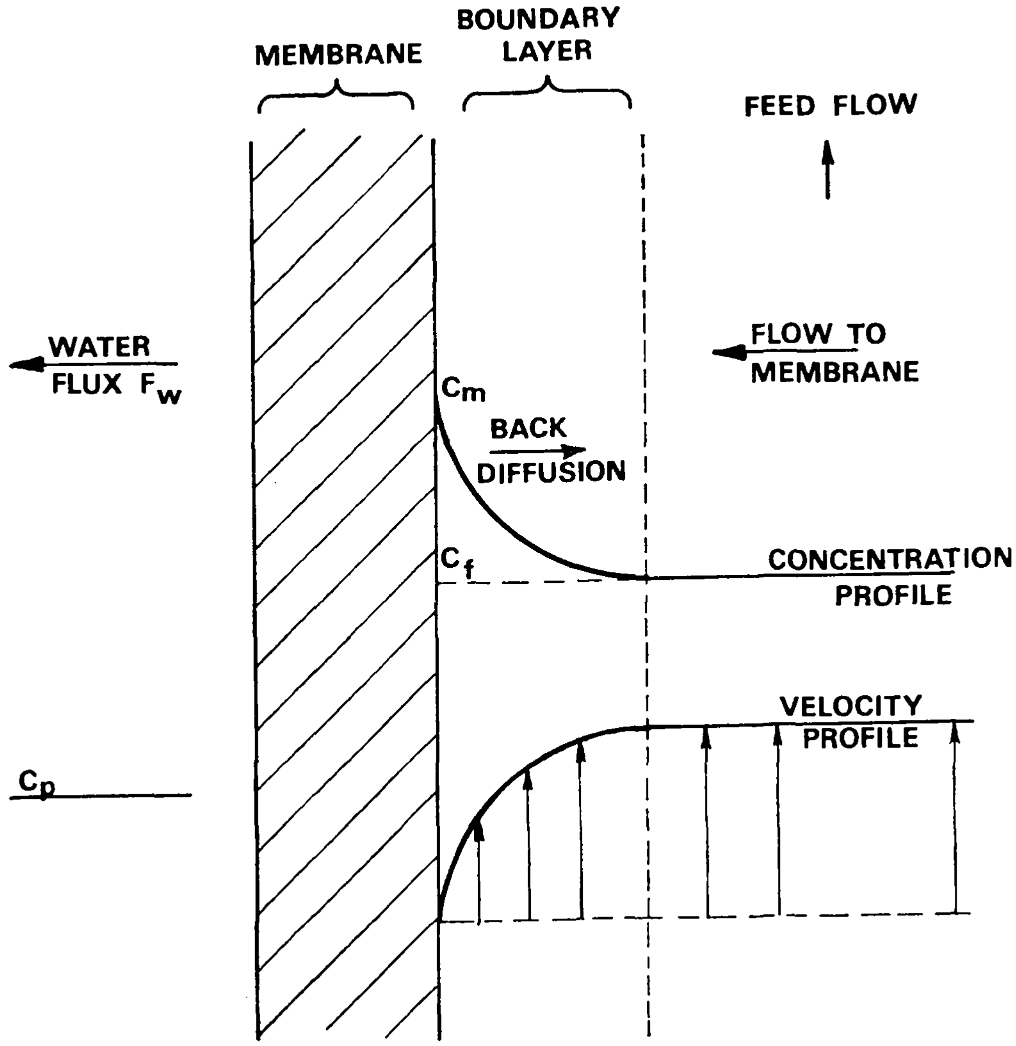

Figure 1—Diagram of steady state Concentration Polarisation.

Concentration polarisation presents a number of negative effects on hyperfiltration operations:

1. The increase of solute concentration on the membrane surface leads to an increase of the osmotic pressure difference $\Delta\pi$, which results in a decrease of water flux (see equation 2).

2. The solute concentration may become so saturated that inorganic salts, especially calcium salts, tend to precipitate on the membrane surface. The precipitate causes a hydrodynamic resistance to the water flow and consequently reduces the flux.

3. High molecular weight organic solutes and colloids tend to form a slime or gel layer on the membrane surface which offers an additional resistance to the water flux.

Under practical conditions the concentration polarisation must be controlled as it brings about a poor membrane performance as well as membrane fouling. Two measures to control concentration polarisation follow qualitatively from figure 1:

1. An increase of the bulk flow rate decreases polarisation. In fact, the concentration as well as the membrane fouling can be avoided by increasing the flow rate up to satisfactorily high values. However, the hydrodynamic friction increases markedly in proportion to the flow rate and consequently increases the pressure drop and pumping costs as well. Thus, under practical conditions one has to compromise between pumping costs and membrane fouling.

2. Reduction of the water flux F_w decreases polarisation. This can be achieved by applying a lower pressure, which saves energy costs. However, a low water flux necessitates a larger membrane area in order to obtain a certain desired product flow, which will result in higher investment costs. One must therefore optimise operation costs and investment costs.

Plant design

As mentioned before, concentration polarisation and membrane fouling necessitate the operation of membrane filtration as a cross flow filtration instead of the generally applied through flow filtration. This is illustrated by figure 2 for tubular membranes. The feedwater is passed along the membrane under high pressure and part of it is removed through the membrane. In consequence the feedwater further down the installation becomes more and more concentrated and is finally discharged as a concentrated "brine". Figure 3 presents the flow chart of a membrane filtration installation. The feedwater is split into two streams: a clean water stream and a concentrated water stream. The product water recovery is expressed as:

$$R = \frac{F_p}{F_f} \cdot 100\% \qquad (3)$$

where:

F_f and F_p are the feed and product flows respectively.

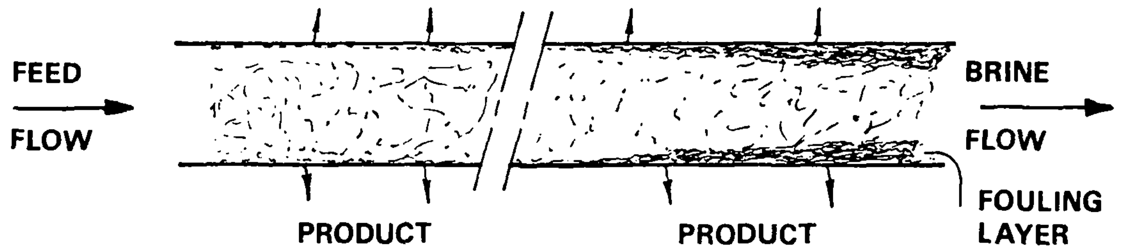

Figure 2—Membrane fouling in a tubular system.

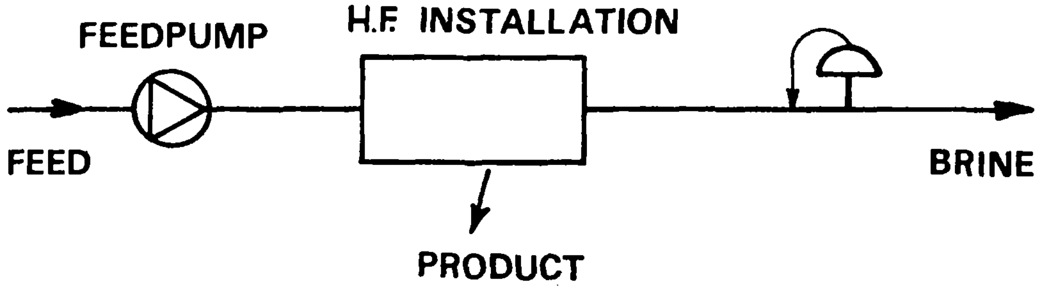

Figure 3—Flow chart for hyperfiltration system.

The recovery is an important factor in connection with the design and operation of an installation:

—a high recovery is preferred because this means a small brine flow. The brine must be disposed of and a higher brine flow may imply greater disposal costs.

—a high recovery with a small brine flow is preferred because the discharge of pressurised brine implies a loss of energy. To a lesser extent this is the same when the energy is recovered because the efficiency of energy recovery is very limited.

—a high recovery means that the brine is highly concentrated. In consequence, the product water at the downstream part of the installation will have a relatively high salt content which has a negative effect on the average product water quality.

—the high brine concentration at high recoveries causes greater membrane fouling at the downstream part of the installation. This effect is of particular importance in the case of saturation at higher concentrations of, for example, calcium salts, which may limit any recovery.

The desired or possible recovery is of great importance for the design of hyperfiltration installations. Another important factor is the necessary feed flow rate and the necessity for maintaining this flow rate in the whole installation. If a number of membrane module are placed in series, the flow rate will decrease because water is removed in passing the successive modules.

Figure 4 shows a so-called "Christmas Tree" module arrangement in which a gradually decreasing number of modules are placed in parallel in order to maintain thc desired flow rate. This is the original arrangement with the disadvantage of a marked pressure drop in the case where a fairly high flow rate has been chosen. The pressure is lower in the downstream part of the installation than in the entrance region and an additional number of modules will have to be installed in order to reach the desired production rate or recovery.

Figure 5 shows an installation in which the necessary flow rate as well as the pressure are kept constant by means of a number of booster pumps. The feedwater is concentrated step by step. This design is more efficient and flexible than that given in figure 4, but needs additional investment costs for the booster pumps.

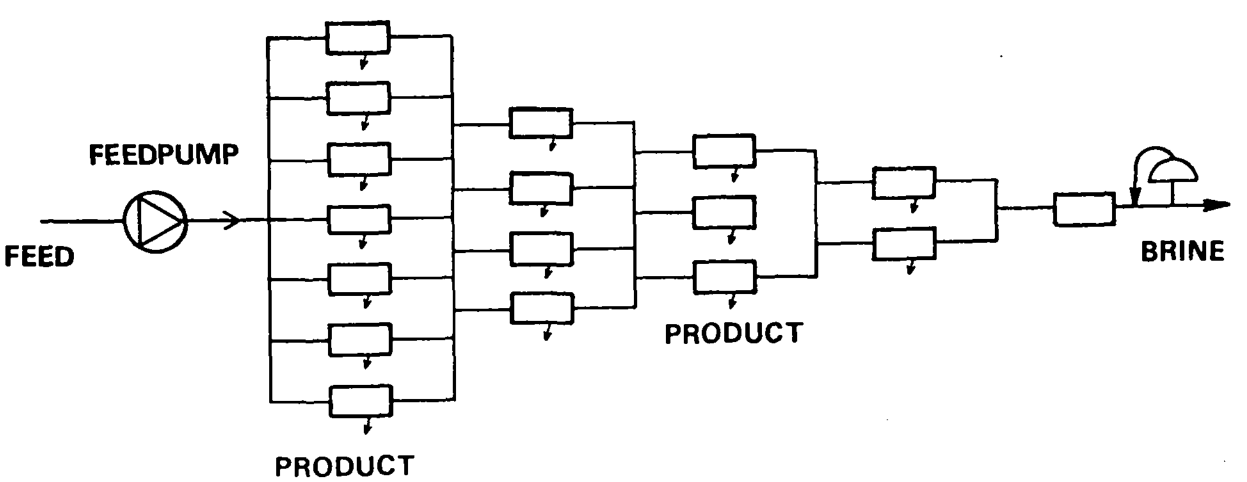

Figure 4—Membrane filtration installation with 'Christmas tree' module arrangement.

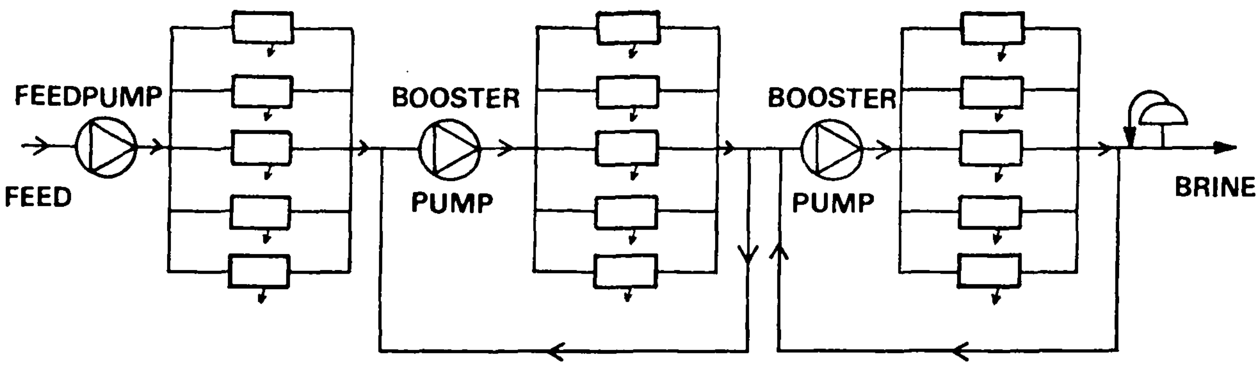

Figure 5—Membrane filtration installation with forced flow rate.

Energy consumption

The energy consumption is one of the main cost determining factors in hyperfiltration. The energy is used to pressurise the feedwater as well as to maintain a sufficient flow rate along the membranes.

The theoretical energy consumption to pressurise 1 m^3 of water to 1 atm. is 0,0275 kWh. Thus, the theoretical consumption for pressurising a feed flow of Q m^3/h to a pressure of P atm. is:

$$0{,}0275 \times P \times Q \text{ kWh} \qquad (4)$$

If the pump efficiency is about 65 %, the practical energy consumption for pressurising is:

$$0{,}04 \times P \times Q \text{ kWh} \qquad (5)$$

Under practical conditions the energy consumption is referred to the product water flow which does not equal the feed flow as the recovery is always less than 100 %.

In addition to pressurisation, energy is consumed because friction effects on the feed flow rate lead to a pressure drop.

This energy consumption may be considerable if high flow rates have to be maintained, for example in tubular systems (0,5′ I.D.) the pressure drop per m membrane is 0,0755 atm. at 2,5 m/s, whereas it is only 0,0042 atm. at 0,5 m/s.

Membranes and membrane materials

At present the majority of the commercially available membranes are made of cellulose acetate and polyamides. The cellulose acetate membranes are used for all types of membrane configurations, whereas polyamide membranes are mainly applied as hollow fibres. The original hyperfiltration membranes were made of cellulose acetate and although the manufacturing procedure has improved greatly during the past 15 years, cellulose acetate is still the most widely used membrane material despite all emphasis on new, improved materials. The applicability and the limitations of cellulose acetate and polyamide membranes are presented in table 2.

TABLE 2: COMPARISON OF CELLULOSE ACETATE AND POLYAMIDE MEMBRANES

Membrane	cellulose acetate	polyamide
maximum working pressure	80 atm.	30 atm.
maximum temperature	30°C	35°C
pH	3–7	4–11
maximum free chlorine concentration in feedwater	1 mg/l	0,1 mg/l

It can be expected that during the next years new membrane materials with somewhat improved characteristics will become available.

The membrane materials, especially cellulose acetate, are applied in a number of different membrane configurations. In all of these configurations, except the hollow fibres, the membranes are supported by a thin layer of porous material and by a pressure resistant material. The water is forced through the membrane and flows through the porous material to the thin channels or holes in the pressure resistant support material where it passes to the outside of the system. The most commonly used systems are described below.

1 Tubular system

In this system, the feedwater flows inside a tubular membrane which is supported at the outside by a pressure resistant material as illustrated in figure 2. The advantage of the tubular membrane configuration is the possibility of controlling the hydrodynamic conditions and consequently the membrane fouling at any point on the membranes. In addition, if the membranes are fouled they can be cleaned mechanically by means of foam ball swabbing, a cleaning procedure which is most effective in removing fouling matter from membranes. These advantages naturally imply that tubular systems are less compact than the other systems and are more expensive.

2 Spiral wound system

Of all the membrane configurations the spiral wound system is the most widely used and has the highest capacity when installed. In this system flat membranes are spirally wound in order to obtain a compact module. A spiral wound membrane and module assembly is shown in figure 6.

The membrane is supported on both sides of a backing material and sealed with a glue along 3 of the 4 edges of the laminate. At the open end the laminate is sealed to a central plastic tube which is then drilled. The membrane surfaces are separated by a screen material which acts as a brine spacer. The entire package is then rolled into a spiral configuration and wrapped in a cylindrical form with tapes as outer wrapping. The membrane cylinder is placed in a cylindrical pressure vessel. The feed flow runs parallel to the central tube, the product flow then going through the membrane and the backing material to the central tube. The advantage of this system is its compactness which allows for relatively low module prices. The system is used in the highest capacity plants now in operation in the world. The disadvantage of this system is that the brine spacer, which causes hydrodynamic conditions which vary on different places on the membrane surface, may result in an increased concentration polarisation and fouling. Moreover, fouled membranes cannot be cleaned mechanically.

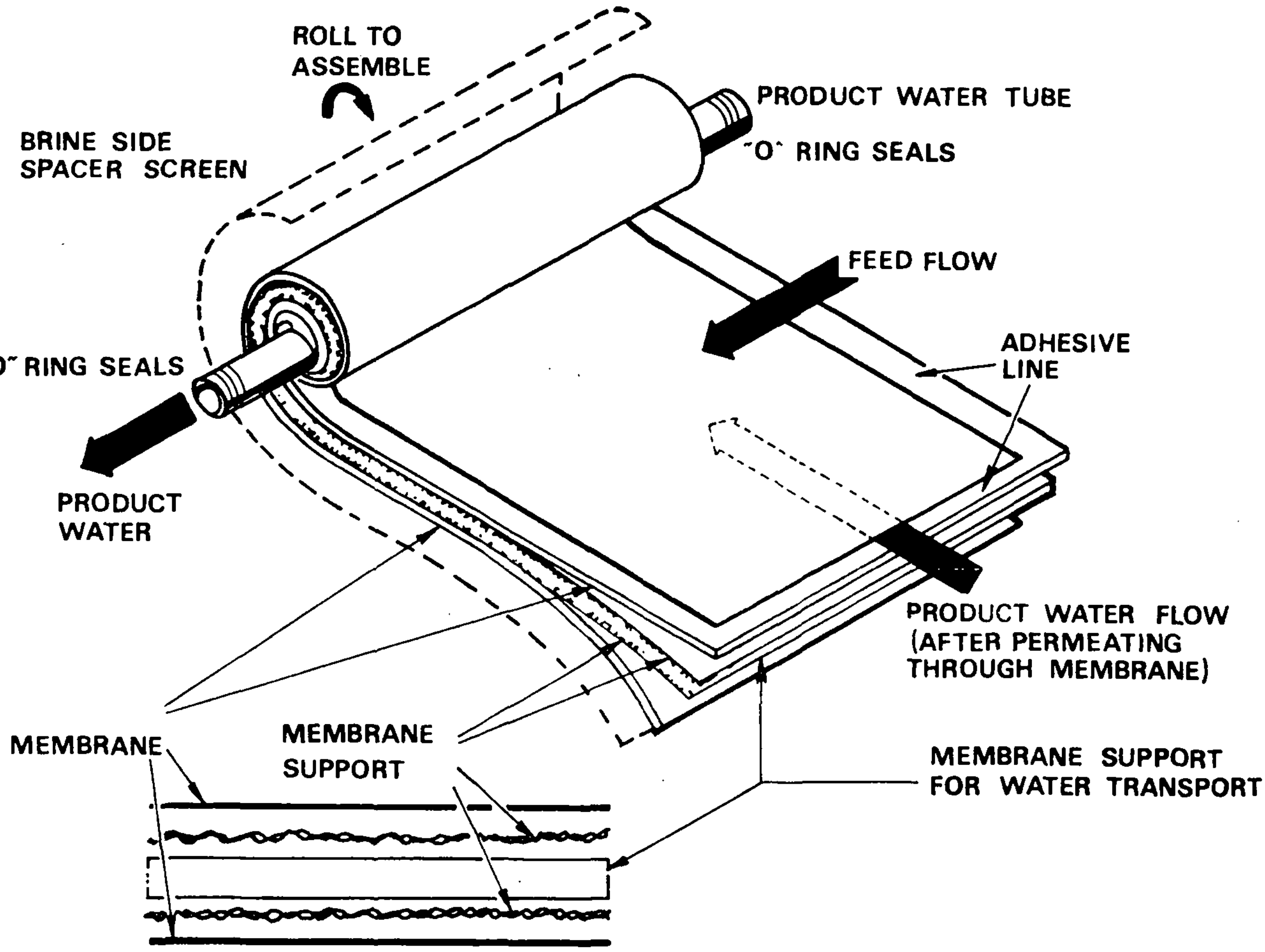

Figure 6—Spiral wound module.

3 Hollow fibres

The hollow fibre system is in fact a tubular system with such small tube diameters that the membrane itself can withstand the high operating pressure. The feedwater flows outside the fibre whereas the product water flows inside the fibre to an open end. Bundles of fibres are sealed by a flange and placed in a cylindrical pressure vessel as illustrated in figure 7. One module contains an enormous number of fibres which together represent a very large membrane area. Membrane materials can therefore be used with a relatively low flux (about 10 times smaller than for cellulose acetate membranes) and yet presenting a highly compact module. This low flux governs a limited concentration polarisation.

In addition, the bundle of closely packed fibres allows for short laminar feed flows. Both effects give limited or no membrane fouling at all when clean water is treated. However, if water containing suspended matter is treated the bundle of fibres will act as a mechanical filter which can be easily blocked by all sorts of suspended particles.

4 Spaghetti type system

This system can be considered to be a hollow fibre system with very thick fibres. A normal cellulose acetate (CA) membrane is cast on a plastic rod having a diameter of a few millimetres. A number of these rods are sealed by an end flange and placed in a pressure tube. The high

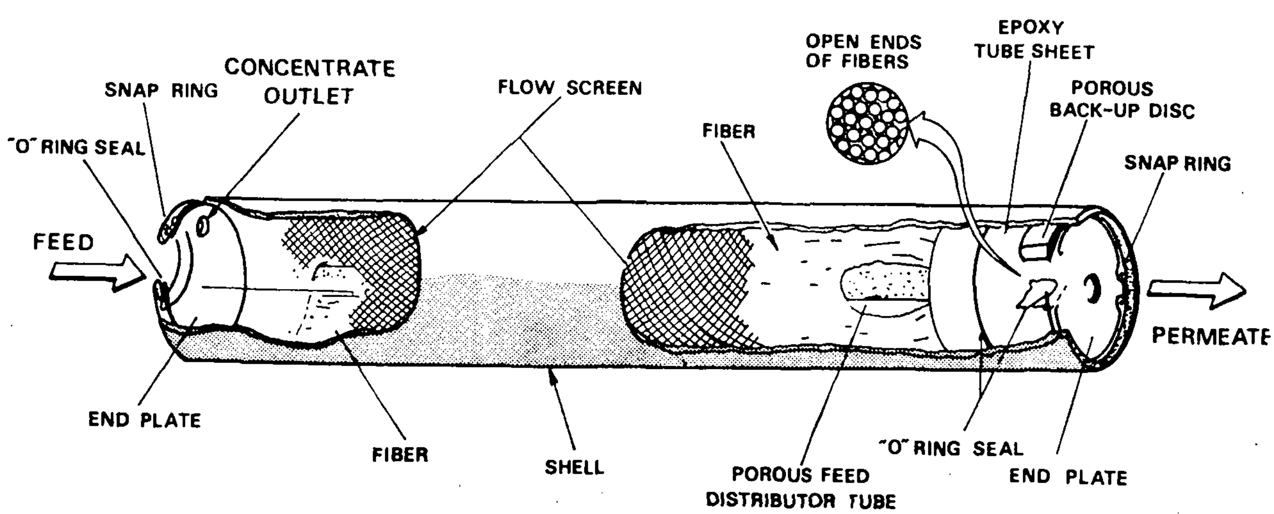

Figure 7—Hollow fibre module.

pressure is on the outside of the membrane. Water flows through the membrane and it passes through thin channels on the surface of the supporting rod to the outside of the pressure vessel.

The spaghetti system is compact and neither expensive nor sensitive to membrane fouling.

When reviewing the available membrane configurations, the following remarks can be made:

1. The tubular system is the most bulky and most expensive although membrane fouling can be prevented very readily. In cases where prevention is not possible, the membranes can be cleaned most effectively. Consequently, the tubular system is most appropriate for the treatment of polluted water or waters containing potential fouling matter, such as suspended matter and precipitating compounds.

2. The other extreme is the hollow fibre system, which is excellent for the treatment of clean water that contains absolutely no suspended matter, because fouling by homogeneously dissolved compounds is very small as the water flux is very low. Any suspended matter, however, will clog the fibre bundle.

3. The other systems, spiral wound, spaghetti and others, have in common that they are rather less sensitive to suspended matter, but when membrane fouling occurs, there is only small possibility of cleaning the membranes.

Membrane fouling and cleaning

Membrane fouling is a practical phenomenon which has a marked effect on the operation of hyperfiltration. Two main types of fouling occur:

I. By fouling matter which is already present in the feedwater in a similar form to that in which it appears in the fouling layer on the membrane. Examples are:

—suspended matter such as clay particles or sludge from biological waste water treatment;

—colloids such as humic substances or iron hydroxide.

II. By compounds homogeneously dissolved in the feedwater and precipitating on the membrane surface, as saturation is reached as a result of concentration polarisation. Examples are:

—inorganic salts such as calcium carbonate and sulphate;

—organic compounds such as homogeneously dissolved humic substances;

—co-precipitates of two or more compounds, e.g. iron (III) forms precipitates with various organic compounds such as humic substances.

In general, there are five methods to deal with these types of fouling:

1 Pre-treatment of the feedwater

This is of particular importance for type I fouling because a pre-treatment can remove suspended particles as well as colloids. Several pre-treatment systems are applied:

—Rapid sand filtration for a rough removal of suspended matter and sand particles.

—Coagulation with iron (III), alumina or lime followed by rapid sand filtration. This system can be applied to the treatment of polluted surface waters and also to effluents of biological waste water treatment plants. In addition to suspended matter it removes certain other unpleasant matter, such as bacteria, viruses, heavy metals and hazardous organic compounds. Coagulation with lime can also remove hardness and is used for waters containing too high calcium concentrations.

—Activated carbon filtration can be applied to remove organic colloids as well as dissolved organics. It is fairly effective for the reduction of organic matter, but may present difficulties because of bacterial growth on the activated carbon particles.

—Precoat filtration with diatomaceous earth is most effective for the complete removal of suspended matter and colloids. In order not to overload these filters it must be preceded by coagulation/filtration where polluted water is concerned.

2 Addition of chemicals

The procedure which follows will deal with type II fouling, especially fouling by calcium salts.

In order to prevent precipitation of calcium carbonate, acid is added until a pH-value of between 5 and 6,5 is reached. Precipitation of calcium sulphate can be prevented or retarded by the addition of polyphosphates. This procedure is advised by several membrane supplying companies, but neglects the fact that the brine is generally disposed of into surface waters and that brine containing an increased concentration of (poly) phosphates may present a serious source of pollution.

3 Feed flow rate

As mentioned before, concentration polarisation and membrane fouling can be considerably decreased by increasing the feedwater flow rate. However, a high flow rate gives a marked pressure drop, which means higher pumping costs, thus the feed flow rate has to be optimised in relation to membrane fouling.

4 Membrane cleaning

Fouled membranes can be cleaned more or less successfully by several methods depending on the type of fouling and the type of module:

—The effect of decreasing the pressure is that for a short time the water flows back from the membrane into the fouling layer which might loosen the layer from the membrane. This back-flow is an osmotic flow resulting from the low salt concentration in the membrane and the high concentration in the fouling layer.

—Flushing of the membrane channels at very high flow rates is sometimes effective for the removal of loosened fouling matter.

—The addition of cleaning agents improves the effect of lowering the pressure and flushing. The following cleaning agents are used:

- inorganic acids to dissolve calcium carbonate precipitates as well as iron hydroxide.
- citric acid for the same purpose and for the removal of organic matter.
- detergents—commercially available blends containing for example boric acid, enzymes and chelating agents e.g. E.D.T.A. All of these are used in order to remove organic matter as well as mixtures of organic and inorganic matter, such as the fouling matter from waste water treatment.
- mechanical cleaning implies foam ball swabbing, the effectiveness of which is very high. Foam balls are carried along by the water flow

through the channels, swabbing the membranes. As a consequence, mechanical cleaning can only be applied when tubular membranes are used. This is a major advantage to this type of membrane configuration. In many applications the fouling layer is not removed by flushing or detergents, whereas this is easily done by swabbing. Therefore, mechanical cleaning is most effective in removing fouling matter from membranes in combination with other cleaning methods. However, it is not advisable to allow excessive fouling of the membranes as the resulting cleaning procedure takes such a long time. Therefore, mechanical cleaning can be applied periodically if membranes are moderately fouled and this offers the possibility of cleaning the membranes completely on the occasions when they are extensively fouled.

5 Membrane configuration

The membrane configuration has a great effect on the fouling as discussed before. For type I fouling (suspended matter), the tubular system is most useful, because membrane cleaning is easier than the hollow fibres system, where the fibres act as a mechanical filter. The other systems foul easily but the degree of fouling depends on the concentration of suspended matter. For fouling, according to type II (dissolved fouling compounds) the hollow fibres are most useful because the least concentration polarisation occurs as a result of the low flux. All the other systems foul more easily because of a higher flux and again only the tubular system can be cleaned most easily and reliably.

Surface water treatment

In many countries all over the world industrialisation started along the big rivers and near the mouths of these rivers. The density of population grew and as a result the rivers became more and more polluted and could serve no longer as a source for plenty of good drinking water. In these highly populated areas the available groundwater sources are generally insufficient. As a result, there is an urgent need for good drinking water in quite a number of these areas. H.F. is an excellent tool to deal with this problem when applied in the right way.

In general, the river water to be treated has an increased salt content and is polluted with all sorts of suspended and organic matter, including hazardous compounds such as bacteria, viruses, carcinogenics, mutagenics, pesticides and heavy metals. If this water is treated by H.F., it has to be pretreated by coagulation and filtration as well as by chlorination, with the possible addition of an activated carbon post treatment. The complete treatment system must be considered as one process consisting of a succession of adjusted unit operations. Such a process has been extensively studied in the Netherlands for the treatment of Rhine river water and it appeared that the process was technically feasible, flexible and reliable. A great advantage is that almost every possible hazardous compound is removed by a succession of unit operations, all removing compounds by different mechanisms.

This means that the product water becomes of a very high quality and that the production is reliable. This is illustrated by table 3, which shows how attractive such

TABLE 3. THE REMOVAL OF SUBSTANCES BY THE TREATMENT PROCESS

Substance	Effect of:			
	lime clarification	hyper-filtration	activated carbon	chlorine
Bacteria and viruses	xxx	xxx	—	xxx
Suspended matter	xxx	xxx	xx	—
Total organic matter	xx	xxx	xxx	
Toxic organics	xx	xxx	xx	—
Inorganic salts	—	xxx	—	—
Toxic inorganics	xx–xxx	xxx	—	—
Phosphate	xxx	xxx	—	—
Nitrate	—	xx	—	—
Ammonia	—	x	—	xxx*
Cyanide	—	x–xx	—	xxx*
Urea	—	x	—	xxx
Phenols	—	x	xxx	
Taste and odour	x	xx–xxx	xxx	
Oil	x	xxx	xx	
Detergents	x	xxx	xxx	
Hydrocarbons	—	x–xx	xxx	—
Chlorinated hydrocarbons	—	x–xx	xxx	—
Volatile organic acids	—	x	xx	
Carbohydrates				
Amino acids				
Fatty acids	x	xxx	xx	
Proteins				

The effect of the removal is expressed as: xxx 90–100%
xx 50– 90%
x 10– 50%
— 10%

*The removal can be realised but the process has to be adjusted.

a treatment system is for the production of high quality drinking water from polluted water. The data for every compound are approximated and not quantitative as they depend on the operational conditions, which can be adjusted.

The question can now be asked whether such a complete treatment system is economically feasible, because the whole treatment seems rather expensive. Though accurate costs are out of the scope of this paper, the relative costs of different systems can be qualitatively estimated. It is believed that the system concerned is economical because the alternatives are also expensive if estimated. It is believed that the system concerned is economical because the alternatives are also expensive if the same quality and reliability are desired. The alternatives imply reservoirs, long transport pipelines and extensive physical chemical treatments which have poor flexibility. In addition, H.F. modules have a relatively short amortisation time compared with "conventional" water treatment installation. This enables the H.F. installations to be designed on the basis of the actual demand and not for an additional future capacity which is the case with conventional installations, which must be financed in advance. It should be mentioned that table 3 shows an excellent treatment performance for polluted water, not only for surface water, but also for e.g. biologically treated municipal waste water. This implies that the treatment system concerned can be used not only for polluted surface water but also for surface water containing considerable percentages of waste water, and also for the direct treatment of municipalities' water. The treatment system implies the potential possibilities for the re-use of waste water for municipal purposes.

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Comité permanent international pour les compteurs d'eau et comptage

Introduction

Le Comité Permanent des Compteurs et du Comptage a le plaisir de vous présenter les trois sujets suivants:

Sujet 1 Choix du modèle et de la dimension des compteurs d'eau par M LAUTERBACH, Directeur de la Gelsenwasser—R.F.A.

Sujet 2 Bancs d'essai pour compteurs d'eau par M SOLLMAN, Ingénieur au KIWA—Pays-Bas.

Sujet 3 Entretien des compteurs d'eau par M WILLIAMS, Vice-Président de l'Indianapolis Water Company—USA.

A cette occasion, le Comité croit utile de faire en quelques mots le point des travaux que poursuivent d'autres organismes internationaux concernant les compteurs d'eau.

A l'International Organization for Standardization (ISO), le sous-comité TC 30 SC 7 a engagé la procédure de publication, en projet, d'une partie de la norme. Cette partie concerne principalement les dimensions ainsi que les prescriptions relatives aux dispositifs indicateurs des compteurs d'eau froide. Plusieurs membres de notre Comité participent à ces travaux.

A l'Organisation Internationale de Métrologie Légale (OIML), les travaux du Comité FL6, concernant le projet de recommandation internationale relatif au compteur d'eau potable froide se poursuivent. Un contact a été établi avec cet organisme à la fin de 1974.

Enfin, la Commission des Communautés Européennes (CCE) a publié en décembre 1975, la directive relative aux compteurs d'eau froide.

S'il faut se réjouir de l'intérêt que portent depuis quelques années aux compteurs d'eau ces différents organismes internationaux, il faut cependant remarquer qu'ils s'occupent parfois de matières identiques et que malgré un réel effort de coordination, ils aboutissent à des dispositions discordantes.

Notre Comité projette de mettre sur pied un programme de travaux et de recherches, à moyen ou à long terme, qui pourra fournir la matière des sujets à présenter lors des congrès futurs.

Le Comité permanent des Compteurs et du Comptage profite de la présente occasion pour souhaiter que d'autres pays se joignent à ses travaux afin d'en élargir l'horizon.

A. ACHTEN, *Président.*

Choix du modèle et de la dimension des compteurs d'eau

par Dr. Ing. K. Lauterbach

Directeur de la Gelsenwasser

1 Introduction

Lors du Congrès AIDE, organisé il y a 15 ans à Berlin, les problèmes relatifs aux systèmes de compteurs ont été discutés. La discussion avait trait, d'une part, aux compteurs d'eau eux-mêmes et, d'autre part, à la métrologie en général. Il nous appartient maintenant de déterminer si l'on dispose, depuis, de nouvelles connaissances et expériences permettant d'avancer de nouvelles recommandations quant au choix du système des compteurs d'eau. Dans ce but, une brève enquête a été effectuée auprès des membres de l'AIDE.

A l'époque, Hutton avait exposé les problèmes existants et énuméré les critères et modes opératoires dont les différents services publics tenaient compte.

Dans les différents pays, les services d'eau utilisent toujours des compteurs à déplacement et des compteurs à turbine en tant que compteurs domestique. Par compteurs à déplacement on entend aussi bien des compteurs à disque que les compteurs à piston rotatif. Ces derniers ne sont que très peu utilisés comme compteurs à grande capacité. Pour les grandes capacités on utilise surtout des compteurs à hélice à savoir des compteurs Woltmann "parallèles" (c'est-à-dire à hélice parallèle à l'axe de la canalisation) et des compteurs Woltmann à hélice verticale (hélice perpendiculaire à l'axe de la canalisation) et ce, aussi bien en tant que compteurs individuels que comme compteurs combinés avec des compteurs domestiques.

2 Evolution des compteurs d'eau

On trouve toujours les mêmes modèles de compteurs qu'à l'époque sur le marché. Aussi convient-il d'abord d'examiner leur état d'évolution. Ceci permettra de savoir s'il est possible de donner aux services publics, suite à d'éventuelles modifications par exemple dans le secteur des matériaux ou des conceptions, des recommandations utiles pour le choix des systèmes des compteurs.

2.1 Matériaux et conceptions

L'utilisation toujours croissante de matières plastiques dans la fabrication des compteurs d'eau s'est révélée très bénéfique pour les utilisateurs de ces appareils de mesure. Il existe actuellement des compteurs domestiques qui, à l'exception de leur boîtier, sont entièrement réalisés en matière plastique. Ceci répond à une exigence importante des services publics relative à la résistance à la corrosion. Par ailleurs, ce matériau est également très résistant à l'usure par frottement. Compte tenu de sa surface lisse, ce matériau est également très approprié pour une utilisation dans des eaux ayant une tendance à précipitations.

Une plus grande légèreté des pièces mobiles est une autre caractéristique positive. En ce qui concerne en particulier les compteurs à turbine, un rotor plus léger signifie un meilleur démarrage et une usure des paliers moindre. Ces modifications aboutissent à une sensibilité de mesure plus grande, à une étendue de mesurage plus grande, à une plus grande constance et à une plus grande capacité de charge.

Dans ce domaine, cette évolution a conduit à des compteurs d'une capacité de $\frac{3}{5}$ m^3 et de $\frac{7}{10}$ m^3 qui sont utilisés comme compteurs dits "à étendue multiple" ou "à grande étendue". Cela signifie que les étendues de mesurage de deux dimensions de compteurs ont été réunies. Les limites inférieures d'étendue de mesurage et les débits de transition correspondent aux valeurs les plus petites. En plus, ces compteurs possèdent une limite inférieure d'étendue de mesurage plus basse que les "anciens" compteurs et laissent passer un débit supérieur à 5 ou 10 m^3/h respectivement, sous une perte de pression de 1 bar.

La comparaison des courbes d'erreur et de perte de pression de compteurs ayant un débit nominal de 5 m^3/h (compteurs à piston rotatif, compteurs à étendue multiple et compteurs à turbine anciens modèles) montre le changement intervenu (figure 1).

Pour l'enregistrement de la consommation à la fois à des débits très petits et très grands, les services publics disposent de compteurs dits "combinés". Sur la base des différentes normes existant dans les différents pays, ces derniers se distinguent généralement par le fait que dans le domaine de basculement le taux d'erreur est très grand. Selon les normes américaines par exemple, ce taux d'erreur est limité à 15% dans cette zone.

Depuis quelque temps, il existe des compteurs combinés dont les indications restent à l'intérieur du taux d'erreur admissible (étendue inférieure de mesurage) du compteur principal, même dans la zone de basculement, c'est-à-dire à l'endroit ou le mesurage passe du petit compteur au grand compteur ou vice versa.

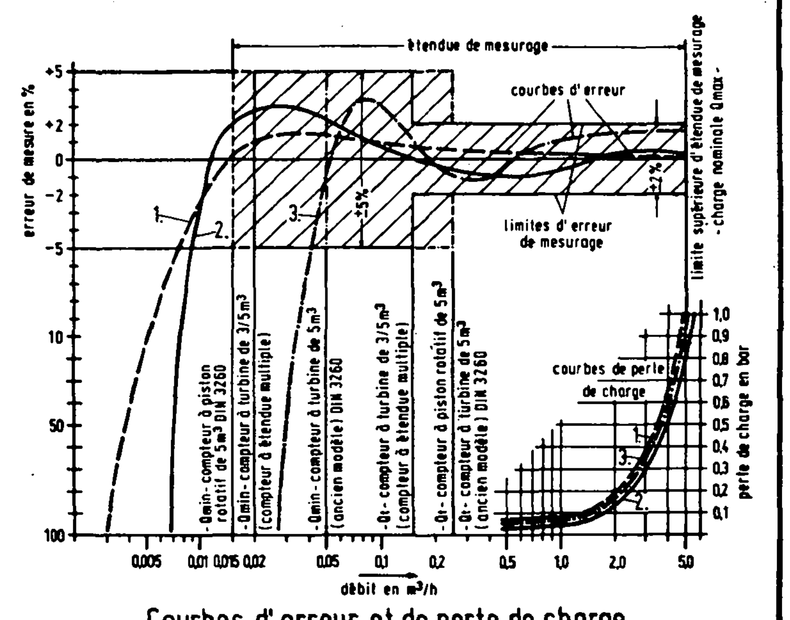
2.2 Capacité de charge

Dans différents pays on exécute des essais accélérés d'usure afin de déterminer quelles charges un compteur peut supporter et comment sa précision change dans ces conditions.

En partie, ces essais sont exigés par la législation des pays concernés. Dans ces tests les compteurs sont soumis à des essais d'endurance dans lesquels sont simulés les types de charge que le compteur devra supporter ultérieurement. Pour l'homologation des compteurs de $Q_n < 15$ m^3/h la directive C.C.E. exige que les différents types de compteurs subissent 100 000 mises en charge à un débit nominal de Q_n ($=Q_{max}/2$) avec des durées de fonctionnement et d'arrêt de 15 secondes, et plus de 2 heures de fonctionnement à Q_{max} sans interruption.

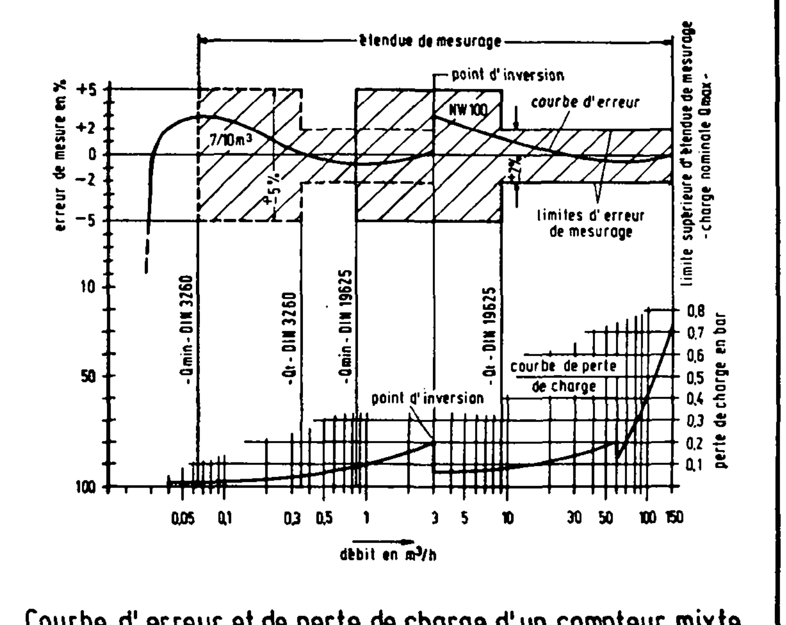
Les compteurs d'une capacité $\geqslant 15$ m^3/h sont soumis pendant plus de 800 heures à un débit permanent de Q_n et pendant plus de 2 h à un débit Q_{max}. Lors de ces essais la courbe d'erreur originale ne doit pas accuser une variation supérieure à 1,5% entre Q_t et Q_{max}, ni de variation supérieure à 3% entre Q_{min} et Q_t.

Les Services Municipaux de la Ville de Munich ont également effectué des essais avec des charges intermittentes (O à divers débits) sur 17 compteurs à turbine d'une capacité de 5 m^3/h de 6 marques différentes. Après un total d'environ 400 000 cycles à 3 débits différents (5, 3, 1 m^3/h) et avec un volume débité total de 5 000 m^3, le taux d'erreur de 16 compteurs se trouvait encore à l'intérieur des erreurs maximales tolérées, soit $\pm 2\%$ entre Q_t et Q_{max}, et $\pm 5\%$ entre Q_{min} et Q_t.



Courbes d'erreur et de perte de charge
1. Compteur à piston rotatif de 5 m³
2. Compteur à turbine de 3/5m³ (compteur à étendue multiple)
3. Compteur à turbine de 5m³ (ancien modèle)

Figure 1



Courbe d'erreur et de perte de charge d'un compteur mixte -compteur Woltman NW 100 Qmax de 150 m³/h - avec un compteur à turbine à étendue multiple de 7/10m³

Figure 2

Le compteur défaillant, dans lequel le capteur était cassé, n'accusait qu'une erreur de – 11 %, donc > à – 5 %, à la limite inférieure de l'étendue de mesurage (Q_{min} = 20 l/h). Des essais effectués sur des compteurs à piston rotatif et des compteurs à turbine d'une capacité de 5 m³/h avec une charge permanente de 5 m³/h pendant plus de 2 000 heures ont montré qu'ils étaient à peine usés. La courbe moyenne d'erreur des compteurs à piston rotatif s'était déplacée entre Q_{min} et Q_{max} d'environ 0,1 % vers les erreurs négatives, tandis que la courbe moyenne d'erreur des compteurs à turbine accusait dans la même étendue de charge un déplacement d'environ 0,1 à 0% vers les erreurs positives.

Des essais de charge effectués avec des compteurs Woltmann ainsi que le contrôle de la précision après une certaine durée de service dans le réseau montrent que ces compteurs possèdent également une grande constance.

Ainsi deux compteurs Woltmann à hélice perpendiculaire NW 50 de Q_{max} = 30 m³/h ont été soumis à un essai d'endurance avec un débit de 48 m³/h pendant

1200 heures environ, ce qui fait un volume total débité de 58 000 m^3. La courbe moyenne d'erreur s'était déplacée entre Q_t et Q_{max} de 0,3% et à la limite Q_{max} de 1,58% vers le négatif.

Deux compteurs Woltmann à hélice d'axe parallèle NW 80 de $Q_{max} = 100\ m^3/h$ ont été soumis à un essai d'endurance à 50 m^3/h pendant plus de 1 100 heures, ce qui fait un volume débité total d'environ 55 000 m^3. La courbe moyenne d'erreur s'était déplacée entre Q_t et Q_{max} de 0,2% vers le positif, tandis que le déplacement était à peu près nul à la limite Q_{min}.

Quatre compteurs Woltmann à hélice d'axe parallèle NW 150 de $Q_{max} = 300\ m^3/h$ montés en parallèle dans le réseau, ont fonctionné durant 37 mois, chacun des compteurs étant soumis à une charge moyenne d'environ 100 m^3/h, ce qui correspond à un volume débité total de $2{,}6 \times 10^6\ m^3$. Au bout de cette période la courbe moyenne d'erreur s'était déplacée à Q_{max} de 0,3%, à Q_t de 0,6% et à Q_{min} de 3% vers le négatif. Bien qu'il existe entre les différentes marques des différences qui sont principalement dues à des différences dans la conception, les matériaux et la qualité de fabrication, on se rend compte que les compteurs d'eau peuvent être soumis à des charges importantes.

Bien qu'il reste à souhaiter des améliorations de la constance des appareils et des caractéristiques techniques de mesure, l'évolution dans le domaine des compteurs d'eau permet de constater que l'on dispose de modèles de compteurs permettant aux Services d'Eaux d'assurer de manière optimale le service de mesurage, à condition toutefois de choisir le modèle et la dimension adéquats.

3 Critères relatifs au choix des compteurs d'eau

Le plus souvent, les services publics se basent sur les considérations suivantes:

1. Les coûts d'approvisionnement, de surveillance et d'entretien doivent être maintenus le plus bas possible.
2. L'étendue de mesurage des compteurs doit être suffisante pour mesurer avec la plus grande précision possible aussi bien les plus petits débits—par exemple des fuites dans des installations domestiques défectueuses—que de très grands débits—par exemple lors de l'opération simultanée de plusieurs chasses d'eau.
3. Les compteurs doivent présenter un maximum de constance, c'est-à-dire qu'ils doivent rester à l'intérieur des tolérances d'erreur admissibles pendant une durée prolongée de service dans le réseau afin d'étaler les coûts d'échange, de réparation et de contrôle sur une période aussi longue que possible.

Malheureusement il n'existe pas de modèle répondant à l'ensemble des exigences posées. Par conséquent, chaque service public devrait essayer de faire le choix adéquat sur la base des conditions de service d'une part, et des caractéristiques de mesurage et de comportement des modèles de compteurs d'autre part.

Concernant cette question, l'enquête effectuée n'a pas apporté de réponse uniformément applicable à toutes les conditions d'approvisionnement en eau.

3.1 Choix des modèles

3.1.1 *Compteurs domestiques.* La plupart des compteurs sont utilisés pour le mesurage de la consommation domestique. Les services publics se posent toujours la question de savoir s'il faut choisir, pour les besoins domestiques, des compteurs à déplacement ou à turbine. Sur le marché européen le prix d'achat des compteurs à déplacement est d'environ 30 à 40% plus élevé que celui des compteurs à turbine. Les frais de réparation s'élèvent à environ 60% du prix d'achat pour les deux types de compteurs (frais de réparation et d'essais uniquement).

Suite à l'évolution survenue depuis la moitié du siècle dernier, on utilise surtout aux Etats-Unis, en France et en Grande-Bretagne et dans les pays subissant l'influence économique de ces derniers, des compteurs à déplacement et dans les autres pays occidentaux et orientaux des compteurs à turbine.

Suite au rapprochement des pays et à leurs interconnexions économiques qui s'expriment en l'occurrence par l'établissement de directives et de normes de mesurage communes pour les compteurs d'eau, la question de savoir s'il faut utiliser des compteurs à déplacement ou des compteurs à turbine devient plus pressante. Dans ce domaine il convient de mentionner l'étude effectuée par l'OIML qui travaille actuellement sur une recommandation relative aux compteurs d'eau froide, ainsi que les travaux de l'Organisation Internationale de normalisation (ISO) qui établit actuellement une norme relative aux compteurs d'eau froide qui comprendra entre autres les instructions de montage pour les compteurs d'eau. Sur un plan local, c'est-à-dire celui du marché commun, une "Directive relative aux compteurs d'eau" a été adoptée fin 1974 et tous les pays membres doivent la mettre en application.

Les Services publics utilisant des compteurs à piston rotatif soulignent que compte tenu de leur plus grande sensibilité, c'est-à-dire leur limite inférieure de mesurage basse, ces compteurs sont très appropriés pour l'enregistrement des faibles débits. Par contre, les utilisateurs de compteurs à turbine font valoir que ces compteurs sont moins sensibles à l'influence des eaux que les compteurs à piston rotatif.

Chacun de ces types, avec ses qualités propres, convient donc pour l'enregistrement des consommations domestiques.

Par le principe même de mesure, le compteur volumétrique présente plus d'avantages pour l'enregistrement des débits très faibles que le compteur à turbine. Conformément aux normes et à la documentation des constructeurs européens, le compteur à piston rotatif possède une plus grande sensibilité que le compteur à turbine, ce qui est surtout le cas pour le compteur de 3 m^3. Par exemple, selon les normes DIN, un compteur à piston rotatif de 5 m^3 possède une limite inférieure d'étendue de mesurage de $15\ l/h = 0{,}003\ Q_{max}$. Un compteur à turbine de 5 m^3/h ancien modèle possède une limite inférieure d'étendue de mesurage de $50\ l/h = 0{,}01\ Q_{max}$, tandis qu'un compteur moderne de $\frac{3}{2}\ m^3$ s'approche de la sensibilité du compteur à piston rotatif car sa limite inférieure d'étendue de mesurage est de $20\ l/h = 0{,}004\ Q_{max}$ (cf. figure 1).

Conformément aux standards américains AWWA C700–71 et AWWA C708–75, les limites inférieures d'étendue de mesurage sont identiques pour des compteurs à déplacement et des compteurs à turbine de dimensions à peu près similaires ($Q_{max} = 4{,}54\ m^3/h = 20$ gpm), soit un débit de $57\ l/h = \frac{1}{4}$ gpm $= 0{,}0125\ Q_{max}$.

Malgré la "concurrence" existant depuis plusieurs dizaines d'années entre ces deux types de construction, il n'existe à ce jour aucune donnée précise pour toutes les conditions de prélèvement disant clairement si pour une eau "idéale" il ne faut choisir que l'un ou l'autre modèle. Seules des mesures comparatives très approfondies dans des réseaux identiques comportant les deux modèles pourront aider à éclaircir cette question. Lors de ces mesures il conviendra de tenir compte des précisions d'enregistrement du volume total en fonction des courbes d'erreur des compteurs et leur variation par suite de l'usure et de l'influence de la qualité des eaux au cours de la durée de service (constance de mesurage). Les essais de laboratoire ne seront pas suffisants à cet effet.

Conformément aux réponses obtenues lors de l'enquête, il existe peu de doute que la composition et les caractéristiques de l'eau, qui ne jouent qu'un rôle subordonné pour le compteur à turbine, ont une signification importante pour le choix par exemple d'un compteur à piston rotatif. Sa fiabilité et sa constance de mesurage dépendent dans une large mesure de l'éventualité que l'eau traversant le système de mesure comporte des particules solides ou/et dépose des précipitations par suite de sa composition chimique. L'apparition d'égratignures et de rainures sur le piston rotatif et les parois de la chambre de mesure—éventuellement combinée avec un blocage du piston—ainsi qu'un polissage des surfaces de contact entravent sérieusement la précision de mesure et partant également la constance de mesure. La courbe d'erreur se déplace vers le côté négatif.

En ce qui concerne les compteurs à turbine les caractéristiques de l'eau influencent également la précision de mesure. Compte tenu du fait que le matériau a été remplacé par de la matière plastique, l'agressivité de l'eau n'a plus d'influence sur les arbres et tourillons de certaines marques de compteurs à turbine à jet multiple domestiques. La précision de mesurage de ces compteurs sera donc uniquement influencée par les sédiments emportés par l'eau et par des dépôts. Dans la mesure où il n'y a pas de surcharges excessives, l'usure au niveau des paliers dans le secteur de la chambre de mesure est faible. Dans la plupart des cas, les changements ainsi occasionnés ont, en liaison avec les retrécissements provoqués par des dépôts dans les orifices d'admission de la chambre de mesure, un effet tel que la courbe d'erreur de mesurage se déplace vers le positif. Ce fait a été confirmé par de nombreux essais. Dans un atelier en République Fédérale d'Allemagne par exemple, les courbes de 400 compteurs prélevés au hasard dans le réseau (avec une eau à faible tendance à précipitations) se trouvaient après 6 ans de service encore à l'intérieur des taux d'erreurs admissibles, cependant que la courbe moyenne d'erreur des compteurs s'était déplacée d'environ 0,5% vers le positif, en diminuant de la limite Q_{max} vers la limite inférieure de mesurage Q_{min}.

3.1.2 *Compteurs d'eau à grande capacité.* En ce qui concerne le choix du modèle de compteur d'eau à grande capacité utilisé pour la consommation industrielle, les opinions divergent peu. Pour ces besoins, il n'existe pratiquement que deux modèles: le compteur Woltmann à hélice d'axe parallèle et le compteur Woltmann à hélice perpendiculaire. Sur le marché européen, le coût d'achat des deux modèles est à peu près identique. Le coût d'entretien s'élève à environ 60% du prix d'achat. Les limites inférieures d'étendue de mesurage—plus élevée pour le compteur Woltmann à hélice d'axe parallèle que pour le compteur Woltmann à hélice perpendiculaire—n'ont pas toujours de l'importance car ces compteurs sont généralement utilisées dans des cas où même les prélèvements les plus faibles sont élevés. Dans ces cas c'est souvent la perte de pression différente—plus faible pour le compteur Woltmann à hélice d'axe parallèle que pour le compteur Woltmann à hélice perpendiculaire—qui influence le choix.

Les compteurs combinés sont préconisés lorsqu'il faut tenir compte d'une part de débits faibles et d'autre part de débits élevés, lorsqu'il faut par exemple prévoir de l'eau pour éteindre les incendies.

Compte tenu des conditions de pression dans le réseau, la valeur de la perte de pression des différents modèles de compteurs combinés est également importante pour le choix du compteur. Le coût d'achat est environ le triple de celui d'un compteur à grande capacité correspondant. Le coût d'entretien et de réparation ne s'élève qu'à environ 40% du prix d'achat.

3.2 Limites de charge appliquées et recommandées

Les directives, normes et prescriptions légales existantes ainsi que les recommandations des constructeurs varient selon les pays et souvent également selon les entreprises de distribution d'eau.

Ainsi les normes américaines stipulent que les compteurs à déplacement et les compteurs à hélice (compteurs à hélice à jet multiple et compteurs Woltmann) ne peuvent atteindre que brièvement la limite supérieure de charge (Q_{max}), tandis qu'il n'est pas recommandé de dépasser une charge de service continue d'environ 0,3 Q_{max} pour les compteurs à déplacement, d'environ 0,65 Q_{max} pour les compteurs à hélice à jet multiple et 0,5 Q_{max} pour les grands compteurs à hélice.

Selon les dires des spécialistes japonais questionnés les compteurs pourraient être chargés temporairement à Q_{max} et de façon continue à 0,7 Q_{max} dans ce pays.

La Directive C.C.E. est conforme aux exigences des standards américains en ce qui concerne la charge Q_{max}, tandis qu'elle prévoit pour une charge prolongée, continue ou discontinue une limite admissible de 0,5 Q_{max}.

Les recommandations publiées par les constructeurs européens admettent pour tous les modèles de compteurs une charge temporaire de Q_{max}—pour les compteurs à turbine domestiques également une charge temporaire supérieure à Q_{max}—tandis qu'une charge entre 0,5 Q_{max} et Q_{max}, est donnée comme limite pour une charge continue pour des compteurs de Q_{max} <30 m^3/h et qu'une charge entre 0,3 et 0,5 Q_{max} est donnée comme limite pour une charge continue pour des compteurs $\geqslant$ à 30 m^3/h.

3.3 Méthodes et conditions de base pour le choix de la dimension

Le choix de la dimension d'un compteur doit être fait de telle manière que non seulement les plus petits débits soient enregistrés, mais aussi que le compteur permette les plus grands débits prévus tout en garantissant les conditions de pression nécessaires aux consommateurs. D'éventuelles contraintes ou mêmes surcharges risquant d'endommager le compteur ou rendant nécessaire son échange prématuré (raccourcissement de la durée de vie) sont à éviter.

Bien que les limites de charge des compteurs soient connues, il ressort des réponses des représentants de presque tous les pays questionnés que beaucoup de réseaux comportent des compteurs surdimensionnés. Dans certains cas, le choix de la dimension du compteur avait certainement été guidé par la crainte de surcharger éventuellement le compteur. Toutefois pour la plupart des distributeurs il existe toujours, par suite du manque de données de base précises, la difficulté de dimensionner les compteurs de telle manière que les limites de charge données soient utilisées au maximum, c'est-à-dire la difficulté de choisir le compteur le plus petit possible dans la mesure où les conditions de pression le permettent. Ceci est surtout valable pour les compteurs domestiques dont la dimension définitive devrait pouvoir être déterminée avant montage.

La quantité, la durée et la simultanéité des prélèvements varient suivant les habitudes des utilisateurs. Compte tenu de la multitude de types d'installations de prélèvement existants, il sera difficile d'arriver à des données de base uniformes et valables pour tous.

3.3.1 *Compteurs domestiques.* Pour des pavillons et villas ainsi que pour des appartements individuels (un compteur par appartement) un petit compteur de

$Q_{max} = 3\ m^3/h$ sera généralement suffisant, les points de prélèvement spéciaux demandant une dérivation. Si les conditions de pression le permettent, cette dimension de compteur peut également être choisie lorsqu'il existe une chasse d'eau ayant un débit momentané d'environ 1 l/sec. Un certain nombre de distributeurs montent comme plus petites dimensions un compteur de Q_{max} d'environ $5\ m^3/h$. Dans les conditions européennes actuelles, cette dimension paraît raisonnable pour l'utilisation de compteurs à turbine si l'on monte par exemple des "compteurs à étendue multiple", permettant également d'enregistrer des débits très faibles.

Toutefois, dès qu'un grand nombre d'appartements doit être approvisionné, il se pose la question de la dimension du compteur.

Pour la détermination de la dimension des compteurs d'eau domestiques, il existe trois méthodes:

1. Suivant la consommation quotidienne et mensuelle attendue.
2. Suivant les méthodes de dimensionnement des canalisations.
3. Suivant le nombre d'appartement.

3.3.1.1 *Consommation attendue.* Le choix de la dimension du compteur suivant la consommation, par exemple la consommation quotidienne, mensuelle ou même annuelle, est certainement basé principalement sur des recommandations données antérieurement par des fabricants. Dans ces considérations, on tient souvent compte des indications données par les consommateurs.

Suivant une recommandation qui est essentiellement suivie dans les pays européens, un compteur de $3\ m^3$ ne doit pas débiter par jour plus de 3 à $6\ m^3$ et par mois plus de $30 \times 3 = 90\ m^3$. La même base s'applique aux autres compteurs jusqu'à $Q_{max} = 20\ m^3/h$. A part le fait que les compteurs actuels peuvent être plus chargés, il n'est souvent pas possible avec ce mode de choix, de tenir compte des débits de pointe ou d'en tenir compte de manière optimale.

Par ailleurs, il sera nécessaire de connaître pour le dimensionnement la consommation quotidienne et mensuelle effective. Elle est souvent calculée sur la base de statistiques, c'est-à-dire suivant la consommation par tête en utilisant éventuellement des statistiques propres. Mais comme le nombre d'utilisateurs et l'installation intérieure des appartements (par exemple machine à laver) peuvent changer, un prédimensionnement sur la base de données de consommation reste généralement un calcul et une conception approximatifs. Pour cette raison on renonce souvent à cette méthode de dimensionnement puisqu'elle mène souvent à des compteurs surdimensionnés.

3.3.1.2 *Bases de dimensionnement pour installations domestiques.* Dans différents pays il existe des bases de calcul pour le dimensionnement des canalisations, c'est-à-dire pour les canalisations domestiques intérieures elles-mêmes et aussi la canalisation de raccordement.

De telles bases de calcul existent notamment depuis plusieurs dizaines d'années en France, en Suisse, aux Pays-Bas et en Allemagne. Sur ces bases, qui reposent partiellement sur des considérations et estimations théoriques, les points de prélèvement installés sont pris en compte avec leur débit pour la détermination de la charge maximum des canalisations en supposant une certaine simultanéité de leur sollicitation. Toutefois, dans les différents pays les débits considérés pour le même robinet varient souvent.

Le document français NF/P41-201 tient compte, pour la détermination du débit maximum attendu, d'un facteur de simultanéité de $y = 1/\sqrt{x-1}$, x étant le nombre des points de prélèvement par lequel la capacité de débit des points de prélèvement est multiplié. Le document DVGW W 308 suppose que pour n points de prélèvement la charge maximum n'est que de $\sqrt{n}$ fois plus grande. L'unité de charge (1 BW) est le débit d'un robinet de $\frac{3}{8}''$ avec un débit de 0,25 l/sec. Pour Z valeurs de charge on calcule alors un débit maximum tel que $q = 0{,}25\sqrt{Z}$ l/sec.

De ces deux documents, appliqués au dimensionnement des compteurs, il résulte les dimensions de compteurs représentées à la figure 5 selon le nombre d'appartements (avec un équipement sanitaire équivalent). Les résultats sont presque identiques. Il est tenu compte d'une charge admissible des compteurs de moins de 0,5 Q_{max} pour le document français et de moins de 0,7 Q_{max} pour la directive allemande. Les recommandations hollandaises et suisses préconisent à peu près les mêmes dimensions de compteur.

Si l'on transfère ces bases de dimensionnement à la détermination de la dimension du compteur, les compteurs seront sans aucun doute surdimensionnés. Ceci est également valable pour les cas où la dimension du raccord est identique à la section calculée de la canalisation de branchement.

Par contre, il est nécessaire de prévoir pour le dimensionnement des canalisations un facteur de simultanéité qui ne soit pas trop petit afin d'être sûr que même lors de conditions de pression défavorables dans le réseau, on dispose encore au point de prélèvement le plus élevé ou/et au point de prélèvement le plus éloigné toujours de suffisamment d'eau avec une pression satisfaisante (pression d'écoulement $\geqslant 0{,}5$ bar). Par ailleurs, il convient d'éviter un sous-dimensionnement des canalisations car une modification ultérieure des installations intérieures représente toujours, aussi bien pour les service distributeur que pour le consommateur, des inconvénients et des coûts élevés.

Toutefois, afin d'enregistrer également les débits les plus faibles, il est nécessaire, lors de la détermination de la dimension du compteur de le choisir le plus petit possible et de profiter autant que le permettent les conditions de pression dans le réseau, de sa charge maximale admissible. Ceci est valable même au risque de placer de temps en temps un compteur trop petit. Il est toujours possible de placer ultérieurement à peu de frais un compteur plus grand qui sera échangé rapidement.

Etant donné qu'un surdimensionnement des compteurs résulte des méthodes de dimensionnement des canalisations certains distributeurs en Allemagne ont procédé à une réduction du facteur de simultanéité tout en maintenant les bases de calcul. En Suisse, certains distributeurs ont fait de même et les Services Municipaux de Zurich et de Lucerne utilisent pour les différents nombres d'appartements les dimensions de compteur représentées à la figure 5.

3.3.1.3 *Nombre d'appartements.* D'autres services municipaux, comme ceux de Munich et de Berlin, ont effectué des recherches approfondies afin de déterminer pour des centres d'habitation de différentes tailles la simultanéité de l'utilisation des points de prélèvement. Ils ont déterminé les charactéristiques de prélèvement et le genre de charge par importance, durée et simultanéité.

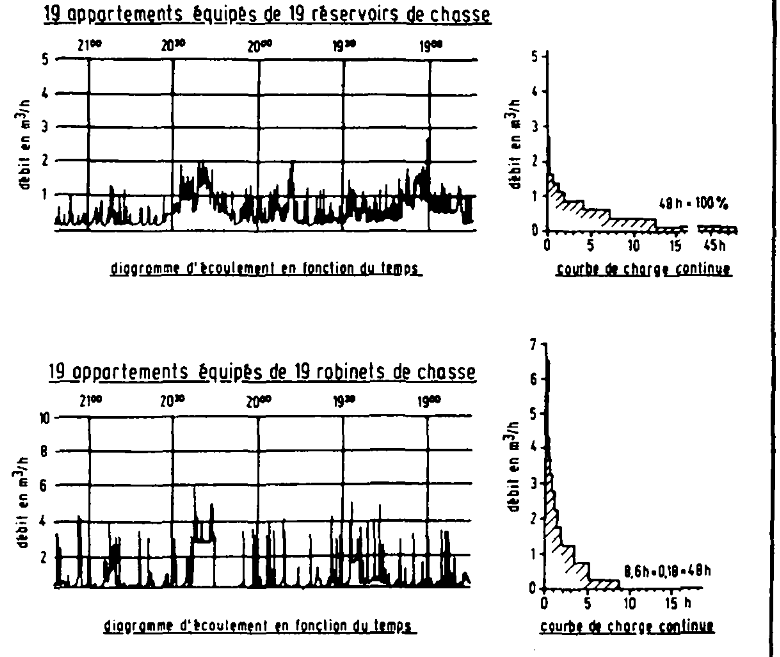
Pour cette étude, les deux entreprises se sont servies d'un compteur à générateur de mesure, c'est-à-dire d'une combinaison d'un compteur à piston rotatif avec un générateur de mesure auquel était raccordé un enregistreur électrique à tracé continu, enregistrant même des prélèvements très brefs, tel que celui d'une chasse d'eau. Actuellement, on pourrait utiliser pour une telle étude un capteur à impulsion.

L'entreprise de Munich a effectué pendant plusieurs jours des mesures en traçant des diagrammes de charge dans environ 500 maisons différentes parmi lesquelles il y avait des petits commerces, tels que des cafés, salons de coiffure, laiteries, boulangeries, boucheries et autres magasins d'alimentation (figure 3). Afin de mieux pouvoir comparer entre eux le grand nombre de diagrammes de charge obtenu (écoulement en fonction du temps), l'entreprise de Munich a conçu des courbes de charge continue servant de modèle de référence pour le dépouillement des résultats de l'étude. Dans ce but les diagrammes d'écoulement en fonction du temps ont été immédiatement dépouillés à l'aide d'un appareil électrique accessoire de construction propre en vue de l'établissement de courbes de charge continue. Une courbe de charge continue (figure 3) représente la durée des différents débits, additionnés à l'intérieur de la période observée. Un tel diagramme montre la part des différents débits dans le volume total débité.

Nombre d'appartements équipés de réservoirs de chasse	Nombre d'appartements équipés de chasses d'eau à robinet	Compteurs d'eau Q_{max} m^3/h
0– 25	0– 1	3
26– 70	2– 25	5
71–120	26– 50	7
121–200	51– 80	10
plus de 200	81–200	20

Comme l'entreprise de Munich, celle de Berlin a également contrôlé, lors de nombreuses mesures effectuées (environ 200), des maisons équipées de chasses d'eau à robinet et des maisons équipées de réservoirs de chasse, en comparant les résultats entre eux.

Pour la détermination de la simultanéité, c'est-à-dire du facteur de simultanéité, Berlin s'est contenté des diagrammes de charge.



Diagrammes d'écoulement en fonction du temps - extraits et courbes de charge continue de centres d'habitation similaires équipés de réservoirs de chasse ou de robinets de chasse (Services Municipaux de Munich)

Figure 3

Au cours de l'étude il a été constaté que dans des centres d'habitation équipées de chasses d'eau à robinet, les pointes de charge étaient sensiblement plus élevées que dans des maisons similaires équipées de réservoirs de chasse (figure 3). D'autre part, on a constaté dans des maisons équipées de chasses d'eau à robinet—en supposant que les installations étaient étanches—un temps de fonctionnement sensiblement plus court des compteurs que dans des maisons équipées de réservoirs de chasse, ce qui est dû à des temps de remplissage plus longs des réservoirs de chasse et à des fuites plus probables dans ces dernières installations.

En tenant compte d'une capacité maximale de charge des compteurs d'environ 0,7 Q_{max} et d'une perte de pression d'environ 0,5 bar résultant des débits de pointe attendus, on trouve selon l'étude de Munich le tableau de dimensionnement suivant (cf. également figure 5):

Pour le dépouillement des résultats, l'entreprise de Berlin a subdivisé sur le diagrammes (figure 4) chaque secteur de charge concerné en secteurs de même débit.

Dans les maisons équipées de chasses d'eau à robinet on s'est basé sur le débit caractéristique pour une chasse à robinet (2,9 m^3/h), et dans les maisons équipées de réservoirs de chasse on s'est basé sur le débit du plus grand robinet existant au moins une fois dans chaque appartement (en général $\frac{1}{2}'' = 1{,}44$ m^3/h). Le débit spécifique correspondant à ce point de prélèvement a été affecté du facteur 1, et à un débit de 1,5 fois supérieur a été affecté le facteur 1,5 etc.... Plus tard, ces facteurs ont été repris dans le calcul comme facteur de simultanéité.

Pour le dimensionnement, l'entreprise de Berlin a pris en considération le secteur de débit dans lequel tombent les débits de pointe déterminés par comptage.

51 appartements équipés de 51 réservoirs de chasse de 3/8" et de 100 robinets de décharge de 1/2".

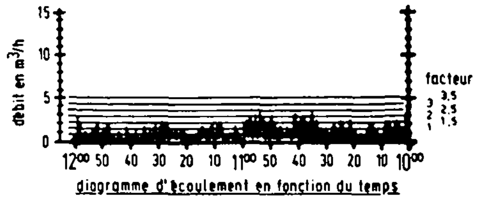


diagramme d'écoulement en fonction du temps

date facteur	1	1.5	2	2.5	3	3.5
m³/h jusqu'à	1.44	2.16	2.88	3.60	4.32	5.04
vendredi	146	83	49	14	1	-
samedi	161	94	66	28	5	-
dimanche	146	86	33	9	5	1
moyenne par jour	151	88	49	17	4	0.33

nombre des pointes de débit

48 appartement équipés de 48 robinets de chasse 3/4"

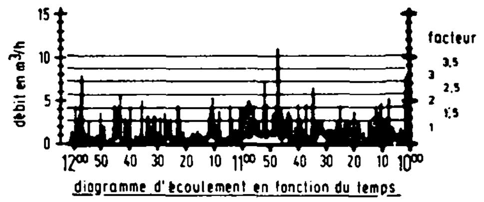


diagramme d'écoulement en fonction du temps

date facteur	1	1.5	2	2.5	3	3.5
m³/h jusqu'à	2.90	4.35	5.80	7.25	8.70	10.15
vendredi	231	220	70	15	1	-
samedi	274	239	55	16	3	-
dimanche	305	239	54	19	8	1
moyenne par jour	270	233	60	17	4	0.33

nombre des pointes de débit

Diagrammes d'écoulement en fonction du temps - extraits et pointes de débit de centres d'habitation similaires équipés de réservoirs de chasse ou de robinets de chasse (Services Municipaux de Berlin)

Figure 4

Les nombreuses études effectuées ont démontré que la valeur du débit de pointe à atteindre, dont il faudra tenir compte pour le dimensionnement, est le produit de la quantité de débit du point de plus grand prélèvement installé et d'un facteur de simultanéité qui résulte du nombre des points de plus grand prélèvement installés (nombre d'appartements).

De plus, avec cette base de dimensionnement, il est tenu compte de ce que la valeur moyenne de débit trouvée doit se situer en dessous de 0,7 Q_{max} du compteur. Ce mode de dimensionnement donne le tableau suivant (voir également figure 5):

Les deux entreprises n'ont pas tenu compte de débits extrêmes temporaires en disant que pour ceux-ci la réserve de 0,7 Q_{max} à Q_{max} est assez grande et que même des surcharges localisées seraient sans danger. A Munich et à Berlin il s'agissait du dimensionnement de compteurs à turbine.

En cas de conditions de prélèvement spéciales, par exemple en présence de consommateurs de quantités d'eau importantes et de maisons à grand nombre d'étages—à Munich des maisons de plus de 7 étages—les dimensions augmentent de manière adéquate. Dans de tels cas on effectue ultérieurement, si nécessaire, des

Nombres des appartements équipés de chasses d'eau à robinet ou de robinets de décharge	Facteur	Chasses d'eau à robinet chaque taille m³/h	Compteurs d'eau Q_{max} m³/h	Robinet de décharge ½" m³/h	Compteurs d'eau Q_{max} m³/h	Robinet de décharge ⅜" m³/h	Compteurs d'eau Q_{max} m³/h
8	1	2,9	5	1,44	5	0,9	5
9– 19	1,5	4,35	7	2,16	5	1,35	5
20– 35	2	5,80	10	2,88	5	1,80	5
36– 45	2,5	6,96	10	3,45	5	2,16	5
46– 85	3	8,7	20	4,35	7	2,70	5
86–125	3,5	10,15	20	5,04	7	3,15	5
126–165	4,5	13,05	20	6,48	10	4,05	7
plus de 165	5	14,50	20	7,20	10	4,50	7

mesures de contrôle en vue de la détermination définitive de la dimension des compteurs.

Dans les deux zones d'approvisionnement en question, les pression du réseau sont supérieures à 4 bars. Depuis plus de 15 ans, le dimensionnement des nouvelles installations ainsi que la modification d'installations surdimensionnées antérieures—une action qui est terminée maintenant—se fait dans les deux zones sur ces bases. Jusqu'à ce jour, aucune réclamation ou perturbation particulière n'a été enregistrée.

A Munich le cycle d'échange des compteurs est de 8 ans—durée de vie maximum prévue par la léglislation allemande—tandis qu'à Berlin, compte tenu des caractéristiques de l'eau, ce cycle est actuellement de 6 ans.

Suite aux expériences acquises à Munich et à Berlin, il a été constaté que la constance de mesure n'est pas influencée par l'utilisation de la partie supérieure de l'étendue de mesure des compteurs.

Un certain nombre de distributeurs en Allemagne, disposant des mêmes conditions de pression, utilisent également avec succès les bases des études effectuées à Munich et à Berlin. D'autres distributeurs, pour lesquels les conditions de pression sont plus défavorables, utilisent également ces bases, avec toutefois un nombre réduit d'appartements pour la dimension correspondante du compteur.

Dans d'autres pays également, comme par exemple en France, en Espagne en Italie et en Belgique, le dimensionnement du compteur est également effectué en fonction du nombre d'unités de logements tout en faisant dans certains cas des distinctions selon le nombre de points de prélèvement installés dans les appartements. Dans ces cas il s'agit généralement d'un niveau différent des équipements d'installations sanitaires.

La Ville de Rome, par exemple, fait la distinction entre trois types d'appartement; pour les deux premiers types le nombre des appartements correspondant à une même dimension de compteur est pratiquement identique, tandis que pour le troisième type d'appartement (environ 30%), un nombre plus réduit d'appartements correspond à chaque dimension de compteur. A la figure 5 est représenté le nombre maximum d'appartements du type moyen correspondant à chaque dimension de compteurs,

Dans un certain nombre de Villes en Espagne, telles que Barcelone, Séville, Valence et autres, on tient compte pour la détermination de la dimension de compteurs d'eau domestiques des "Normes relatives aux installations intérieures de distribution d'eau par compteurs". Cette norme fait la distinction entre 5 types d'appartement suivant le nombre de points de prélèvement installés en tenant compte de la somme de leurs plus petits débits: A <0,6 l/sec; B <1,0 l/sec; C <1,5 l/sec; D 2,0 l/sec; E 3,0 l/sec. Le nombre d'appartements pour une dimension de compteur est également fixé suivant le type d'appartement. La figure 5 tient compte du type d'appartement moyen C.

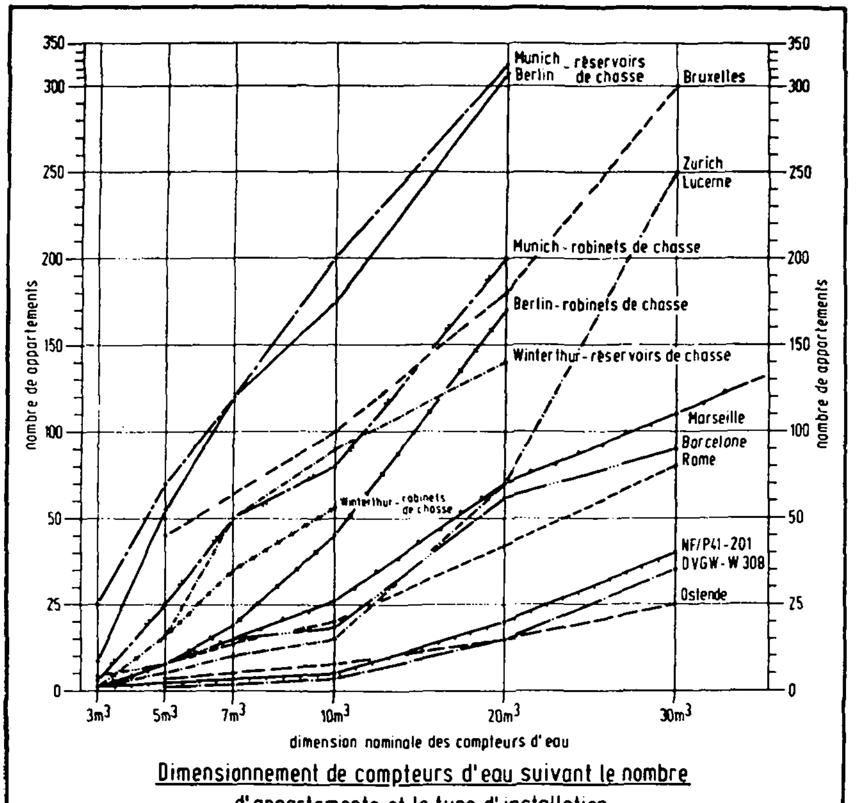
La Ville de Marseille dimensionne la taille des compteurs d'eau (compteurs à piston rotatif) pour l'utilisation domestique également sur la base du nombre d'appartements, et ce jusqu'à des compteurs de $Q_{max} = 100\ m^3/h$. A la figure 5 sont représentées les dimensions de compteur qui en résultent jusqu'à une valeur de $Q_{max} = 30\ m^3/h$.

La Ville de Bruxelles a établi pour le dimensionnement des compteurs (compteurs à piston rotatif) un tableau résumant la correspondance des compteurs jusqu'à un débit nominal de $Q_n\ (Q_{max}/2) = 10\ m^3/h$, tandis que le dimensionnement de compteurs de taille plus importante est effectué sur la base d'une formule. Suivant le tableau on trouve un compteur:

$$Q_n \left(\frac{Q_{max}}{2}\right) = 2{,}5\ m^3/h \text{ pour un nombre d'appartements } = 40;$$

$$Q_n \left(\frac{Q_{max}}{2}\right) = 5{,}0\ m^3/h \text{ pour un nombre d'appartements } = 100;$$

$$Q_n \left(\frac{Q_{max}}{2}\right) = 10{,}0\ m^3/h \text{ pour un nombre d'appartements} = 180.$$



Dimensionnement de compteurs d'eau suivant le nombre d'appartements et le type d'installation

Figure 5

Pour des compteurs de taille plus grande (compteurs combinés) on utilise pour la détermination du débit de pointe attendu la formule empirique suivante:

$$Q\ (m^3/h) = \frac{60\ .\ n\ .\ k}{1000}$$

Dans cette formule:

n = le nombre d'appartements

k = 0,9 pour $150 < n < 300$ appartements

0,8 pour $300 < n < 500$ appartements

0,7 pour $n > 500$ appartements

tout en choisissant la dimension du compteur telle que Q_n soit supérieur à $Q/1{,}2$.

La courbe de dimensionnement qui en résulte est représentée à la figure 5, de même que le dimensionnement pratiqué habituellement à Ostende (Belgique).

On a également reproduit des "courbes de dimensionnement" utilisées par la Ville de Winterthur (Suisse) se rapprochant des courbes de Munich, avec toutefois un nombre réduit d'appartements par dimension de compteur. Ces courbes sont basées sur l'hypothèse que les débits de pointe se trouvent en dessous de 0,7 Q_{max}.

Des expériences acquises dans les années 1960, on peut conclure que la détermination de la dimension de compteurs domestiques sur la base de la consommation horaire, quotidienne ou mensuelle ainsi que suivant les bases de dimensionnement pour canalisations domestiques, donne en général des compteurs trop grands. Les expériences des entreprises de distribution d'eau correspondant à ces méthodes sont représentées à la figure 5. La charge moyenne pour des centres d'habitations comprenant de petits commerces, devrait en général être inférieure à Q_n ($= Q_{max}/2$). Toutefois, pour le dimensionnement des compteurs de consommation domestique il est important de connaître le débit de pointe probable. Pour cela il est nécessaire d'effectuer des mesures comme celles qui ont été entreprises par exemple à Munich, Berlin ou Zurich afin de déterminer le facteur de simultanéité en fonction par exemple du nombre d'appartements, du nombre des points de grand prélèvement installés et du débit de pointe que l'on peut attendre sur cette base. A Zurich on a mis les débits maximaux trouvés en rapport avec la consommation dans chaque appartement.

Au-delà du dimensionnement des compteurs, la méthode de Munich permet de déterminer par dépouillement des courbes de charge continue, la part des différents débits dans le volume total débité. Il devient ainsi possible dans une large mesure de déterminer la précision de mesure du volume débité total en liaison avec les courbes d'erreur individuelles des compteurs. Des mesures répétitives effectuées à certains intervalles de temps permettent de connaître les changements provoqués par l'usure et les caractéristiques de l'eau. Ainsi peut être déterminée la constance des compteurs permettant d'arriver, en tenant compte des coûtes d'achat, d'entretien et d'échange, à une appréciation et à un choix optimal des compteurs. Aux Pays-Bas, il est prévu dans les entreprises de distribution de déterminer de cette manière la durée de vie dans le réseau la plus rentable. Selon les indications données par les entreprises de distribution, la durée de vie des compteurs dans le réseau varie fortement, à savoir de 5 à 15 ans.

3.3.2 *Compteurs d'eau à grande capacité.* Ce qui est valable pour le dimensionnement des compteurs destinés à l'enregistrement de la consommation domestique, est en principe également valable pour l'enregistrement de la consommation dans des bâtiments administratifs, des grands magasins, les écoles, des hôpitaux, des cliniques, des hôtels, des casernes et autres bâtiments où des compteurs d'une capacité supérieure à $Q_{max} = 30\ m^3/h$ sont nécessaires comme compteurs individuels ou comme compteurs combinés.

Tandis que la Ville de Marseille se base pour le dimensionnement des compteurs prévus pour des hôpitaux, des écoles, des maisons de retraite par exemple, sur la consommation quotidienne moyenne et la consommation de pointe en tenant compte du nombre de lits ou d'élèves ainsi que du nombre de points de prélèvement pouvant être utilisés simultanément, la Ville de Bruxelles se base pour le dimensionnement de compteurs pour des grands bâtiments, tels que par exemple des bâtiments de bureau, sur la surface construite et choisit un compteur:

$$Q_n\left(=\frac{Q_{max}}{2}\right) = 2{,}5\ m^3/h \text{ pour une surface construite } > 4\,500\ m^2$$

$$Q_n\left(=\frac{Q_{max}}{2}\right) = 5{,}0\ m^3/h \text{ pour une surface construite } > 6\,000\ m^2$$

$$Q_n\left(=\frac{Q_{max}}{2}\right) = 10{,}0\ m^3/h \text{ pour une surface construite } > 18\,000\ m^2.$$

Pour les grandes surfaces le débit de pointe (15 min) se calcule avec la formule: $Q\ (m^3/h) = S\ (m^2)/1600\ (h/m)$, S étant la surface construite (excepté les surfaces de parking et de stockage). La dimension du compteur est choisie de telle sorte que le Q_n soit supérieur à $Q/1{,}2$, Q étant la charge maximum attendue.

Les Services Municipaux de la Ville de Zurich ont mesuré les débits maximaux d'un certain nombre de restaurants, d'hôtels, de 2 piscines couvertes et d'un hôpital. Les résultats ont été mis en relation avec les points de prélèvement installés et les quantités débitées pour obtenir ainsi les bases de dimensionnement.

Pour le dimensionnement des compteurs prévus pour de tels bâtiments, il est conseillé d'effectuer dans des installations similaires déjà existantes des mesurages de débits avec détermination simultanée des débits de pointe—comme cela a été fait par exemple à Zurich—et de les mettre en relation avec, par exemple les débits de tous les points de prélèvement installés ou le nombre de têtes ou le nombre de lits, afin de permettre de choisir en utilisant ces bases, la dimension à peu près correcte du compteur. Dans tous les cas un mesurage de contrôle devrait permettre d'aboutir à la dimension définitive du compteur. Par ailleurs, il conviendra de tenir compte, en plus des conditions "normales" de prélèvement, d'éventuelles conditions spéciales dues, par exemple, à des installations de lutte contre les incendies ou autres.

Les informations données par les services de distribution d'eau indiquent qu'il faut étudier cas par cas le dimensionnement des compteurs à grande capacité des grands commerces et industries qui le plus souvent requièrent seulement des consommations de longues durées et de pointe et pour lesquels on peut souvent prévoir des compteurs individuels. Pour ce genre de choix, les indications données par les consommateurs en ce qui concerne la consommations horaire quotidienne, mensuelle et annuelle, servent de base. Dans la plupart des cas on effectue des mesures de contrôle—qui devraient d'ailleurs toujours être faites—pour déterminer les caractéristiques de prélèvement.

Au lieu de choisir et d'installer un seul compteur, certains distributeurs préfèrent installer 2 ou plusieurs compteurs de plus petites dimensions montés en parallèle chez le même abonné. En plus de la possibilité d'échanger un compteur éventuellement défectueux pendant la journée, l'avantage du montage en parallèle de compteurs réside dans le fait que la surveillance des indications est plus sûre et plus simple: les compteurs montés en parallèle fonctionnent normalement de manière simultanée, leurs débits temporaires ont un certain rapport entre eux, ils se "contrôlent" mutuellement, les indications divergentes décèlent les défauts existants. Par ailleurs le montage et l'échange de compteurs deviennent plus simples et moins chers. La réparation et le contrôle deviennent également plus avantageux car si ces travaux sont effectués par des moyens propres, les magasins de pièces de rechange et les moyens de contrôle peuvent être plus réduits.

Selon les entreprises de distribution d'eau, la capacité maximale de charge continue ou de pointe des compteurs à grande capacité varie—ceci s'applique également aux compteurs combinés. Un certain nombre d'entreprises dimensionnent les compteurs de telle manière que le débit de pointe attendu reste en-dessous de $Q_{max}/2$—pour cela comme pour le choix du modèle, les conditions de pression dans le réseau jouent également un certain rôle—, tandis que d'autre distributeurs admettent des débits de pointe jusqu'à la limite Q_{max}. Toutefois, tous les distributeurs font le dimensionnement de telle manière que les prélèvements continus restent en-dessous de 0,3 à 0,5 Q_{max}. A la figure 6 est représentée une base de dimensionnement utilisée, entre autres, en Allemagne.

A cause des quantités considérables généralement distribuées par l'intermédiaire de compteurs à grande capacité, on attribue une attention particulière à la durée de vie de ces compteurs dans le réseau. Un grand nombre de distributeurs n'accordent aux compteurs à grande capacité qu'une durée de vie de 2 à 3 ans ou même les échangent plus tôt lorsqu'une certaine quantité de volume débité, par exemple 5 000 × Q_{max} (m^3), est atteinte.

Pour la plupart des distributeurs le choix de la dimension des compteurs représente le problème primordial. Il s'agit ici d'arriver à un enregistrement aussi complet que possible de tous les débits par une utilisation maximale de la capacité de charge des compteurs, en tenant compte des conditions de pression locales.

La détermination de la dimension des compteurs d'eau domestiques se fait selon trois méthodes: sur base de la consommation attendue, suivant des bases de dimensionnement pour des canalisations intérieures ou suivant le nombre d'appartements. La dernière méthode est considérée comme la base de dimensionnement la plus sûre. Les expériences dont on dispose actuellement indiquent que la détermination optimale de la dimension des compteurs est le mieux assurée si l'on détermine au moins les débits de pointe des différents types de consommateurs. A l'aide d'un tableau comme celui élaboré par exemple à Munich et à Berlin, ou à l'aide d'une formule élaborée par ses propres soins comme cela a été fait à Bruxelles, il devient alors possible de dimensionner par avance un compteur "sur mesure".

Pour les compteurs d'eau à grande capacité, le modèle et la dimension sont presque toujours à fixer avec l'utilisateur et en tenant compte des données de prélèvement probable.

Des mesurages de contrôle ultérieurs confirmeront la justesse du choix. Du point de vue technique ces contrôles apparaissent également justifiées car le nombre de tels raccords est assez réduit.

Tableau de charge pour compteurs à grande capacité

type de compteur	* charge mini Q_{min} m^3/h		dure limitée Q_{max}	charge maxi à 10 heures de service par jour		à 24 heures de service par jour		moyenne admissible de charge continue			† échange prématuré à cause d'un niveau élevé à m^3
								par jour	par mois	par an	
WS et WP	WS	WP	m^3/h	m^3/h	m^3/j	m^3/h	m^3/j	m^3/j	m^3/mois	m^3/a	
NW 50	0,35	1,6	30	15	150	12	288	219	6 570	75 000	150 000
NW 80	0,65	3.0	100	50	500	40	960	730	21 900	250 000	500 000
NW 100	0,85	4,5	150	75	750	60	1 440	1 095	32 850	375 000	750 000
NW 150	1,5	7,0	300	150	1 500	120	2 880	2'190	65 700	750 000	1 500 000
NW 200	12,0		600	300	3 000	240	5 760	4 380	131 400	1 500 000	3 000 000
WP NW 300	35,0		1 500	750	7 500	600	14 400	10 950	328 500	3 500 000	7 000 000

* selon DIN 19625

† *Note:* Echange régulier des compteurs tous les 2 ans.
Si un échange prématuré s'avère nécessaire, il convient de contrôler s'il n'est pas préférable d'installer un compteur plus grand.

4 Conclusions

En résumé on arrive aux conclusions suivantes concernant le choix du modèle et la dimension des compteurs.

En ce qui concerne les compteurs d'eau domestiques, la question du choix du modèle, c'est-à-dire compteur à déplacement ou compteur à turbine ne peut être résolue que localement par le rassemblement d'expériences et de mesurages de comparaison approfondis.

Toutefois, aussi bien pour les compteurs domestiques que pour les compteurs à grande capacité on peut dire que le choix correct du compteur et le maximum de précision d'enregistrement des volumes débités ne sont assurés que lorsque l'on accorde également l'attention nécessaire à la constance des compteurs pendant leurs durées de vie dans le réseau et lorsque le cycle d'échange est fixé en tenant compte de la variation admissible de la précision de mesure.

Summary

This paper deals with the question whether—since the Congress in Berlin at which Mr. Hutton presented the problems of the choice of water meters—there are new technologies and experiences available that allow the making of new statements on this subject.

Following an inquiry amongst the members of IWSA it can be stated that the development of the fabrication of meters has advanced. Because of the extensive utilization of synthetic material (2.1) the measuring range of several types of meters, e.g. vane water meters, has been extended, the accuracy of measurement improved (figures 1 and 2) and the capacity raised.

Fundamental criteria (3) for the choice of meters according to type and size are still: as low as possible

prime, control and maintenance costs; sufficient measuring range; utmost precision of indications as well as utmost longevity so that costs for replacement, repair and control can be kept as low as possible.

When the type of meter for measuring house consumption (3.1.1.) has to be chosen there is still the question: displacement or vane water meter. Meanwhile the lower limit of the measuring range of the vane meter has approached that of the displacement meter, e.g. rotary-piston meter.

However, only extensive comparison measuring, among the types of meters mentioned, within the pipe system and with special regard to the accuracy of indication in relation to the error curves of the meters and their deterioration from erosion during their service life, will lead to the right choice. Laboratory tests are not sufficient.

With regard to large-scale meters as well as to combinations of water meters (3.1.2) the choice of type depends mainly on the pressure ratio and the required measuring range of the meter.

Chapter 3.3. indicates methods and fundamentals employed today and shows how to choose the right size of meter.

In order to determine the right size of domestic water meter (3.3.1), before installation one has to take partly as a basis the daily and monthly expected consumption (3.3.1.1.). Existing records of consumption have to be considered also. Other water supply enterprises (3.3.1.2) take as a basis for the choice of the size of meter existing rating records indicating connection pipelines and the pipes behind the meter. Yet further waterworks (3.3.1.3) take as a basis the numbers and types of apartments, considering the type and the flow capacity of existing intakes within the installation.

There are 2 examples, the waterworks of Munich (figure 3) and Berlin (figure 4), showing how the peak intake relevant to the size of the meter can be determined by measurements carried out within the pipe system, thus leading to fundamentals for dimensioning. Figure 5 shows some "dimensioning curves" being applied for the determination of the size of meters.

As to the choice of the size of large-scale water meters (3.3.2), the same principles can be applied as for domestic water meters with the difference that each case has to be handled separately, because the intake characteristic is different. Figure 6 shows one of the fundamentals for dimensioning practised today.

Regarding the selection of meters according to type and size the following can be summarized: (4).

As to house water meters, i.e. displacement or vane meters, the question of choosing the type can only be solved by extensive local comparative measurements.

Most waterworks choose water meters mainly according to their size. The reason for this is to measure an extensive range of all water flows, thereby using the capacity of the meter to its greatest extent in consideration of the local pressure ratio.

The size of domestic water meters has to be determined according to three methods, that is, consumption expected, rating basis for the pipe system or number of apartments. The latter method is to be regarded as the safest. Present experience points to the fact that the determination of the size of the meter is most successful if the peak consumption of different groups of consumers has been ascertained. By means of a table, e.g. as in Munich and Berlin, or according to a formula developed in Brussels, the meter can be "tailor-made" in advance.

Regarding large-scale meters, in nearly all cases the type and size has to be determined in accordance with the client and in consideration of the probable intake characteristic. The correctness of this method must be proved later by control measurements. These control measurements represent an acceptable expenditure, because the number of large-scale meters is not very important.

For both domestic water meters and large-scale meters it can be stated that the installation of the right meter and the utmost precision in registering the intake quantity can only be achieved if attention is paid to the long term reliability—i.e. exactness of indication of the meter during its service life within the pipe system—and if its replacement is determined in consideration of the tolerance of accuracy of indication.

Sujet 2

Bancs d'essai pour compteurs d'eau

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1 Introduction

A chaque instrument de mesure on demande une certaine précision. Cette précision dépend largement de la destination de cet instrument; d'autre part les possibilités techniques de fabrication interviennent également beaucoup dans la précision.

Tout ceci est valable pour les compteurs d'eau dont se servent les distributions d'eau pour déterminer la redevance à payer par leurs abonnés.

Il va de soi que, pour déterminer la précision de mesure des compteurs, on a besoin d'un banc d'essai. En outre, d'autres aspects des compteurs d'eau exigent des installations d'essai, pour déterminer par exemple la perte de charge et l'endurance.

Dans cette communication sont traités un grand nombre d'aspects qui influencent la précision de la détermination de l'exactitude du compteur.

Quelques aspects concernant la détermination de la perte de pression des compteurs d'eau sont aussi traités, avec quelques informations sur ces accessoires hydrauliques dits "prises de pression".

Ensuite, le présent document traitera des méthodes pour déterminer l'endurance des compteurs d'eau. Il y est fait référence à une directive de la Communauté Européenne qui donne des prescriptions relatives aux essais d'endurance des compteurs d'eau.

2 Précision des compteurs d'eau

Pour plus de clarté, la présente communication se restreint aux compteurs d'eau les plus connus, c'est-à-dire les compteurs volumétriques et les compteurs de vitesse en ce qui concerne les consommations domestiques, et les compteurs volumétriques et les compteurs Woltmann en ce qui concerne les consommations industrielles.

Dans ces limites, les compteurs traités sont inclus dans la définition suivante, figurant dans la directive de la Communauté Européenne:

"Compteurs d'eau froide utilisant un procédé mécanique direct faisant intervenir des chambres volumétriques à parois mobiles ou l'action de la vitesse de l'eau sur la rotation d'un organe mobile (turbine, hélice, etc.)."

La précision d'un compteur d'eau doit être considérée en relation avec le débit. En effet, cette précision n'a pas une valeur constante, qu'on pourrait par exemple exprimer en fonction de la charge maximale. On exprime la précision de mesure des volumes débités en fonction des différents débits, ce qui se traduit le mieux sous forme d'une représentation graphique. (voir figure 1: courbe de précision).

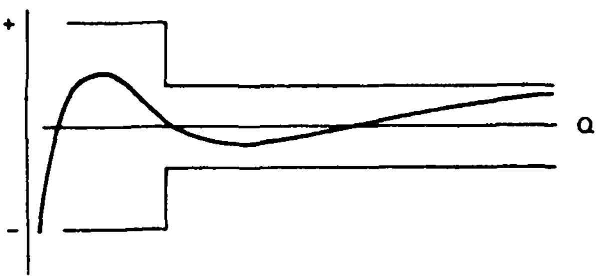

Figure 1—Courbe de précision d'un compteur d'eau.

Bien que les valeurs puissent être différentes, les courbes de précision de compteurs de même type, de même calibre et de même série de production sont très similaires, tandis que les courbes de compteurs de systèmes de fonctionnement différent ne présentent pas cette ressemblance, ce qu'on peut voir sur la figure 2.

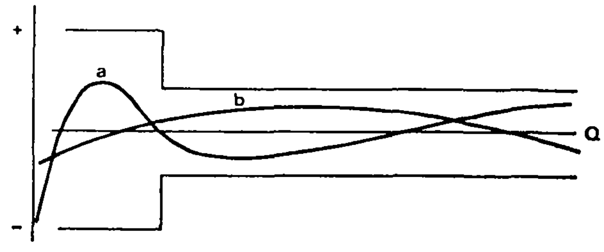

Figure 2—Courbes de précision de différents types de compteurs d'eau (a=compteur de vitesse; b=compteur de volume).

Depuis longtemps les exigences quant à la précision des compteurs d'eau considérés sont équivalentes sinon pareilles. Lors du Congrès de l'AIDE tenu à Brighton (U.K.) en 1974 la contribution de Monsieur A. W. Achten a largement traité de l'évolution des normes et des directives concernant les courbes de précision.

En conséquence, on se contentera par la suite de se référer à la susdite contribution.

Afin de bien éclairer la suite, le tableau 1 ci-après donne à titre d'exemple les exigences figurant dans la directive de la Communauté Européenne.

TABLEAU 1

Classes métrologiques	Débits	Q_n < 15 m³/h	Q_n ⩾ 15 m³/h
A	Q_{min}	0,04 Q_n	0,08 Q_n
	Q_t	0,10 Q_n	0,30 Q_n
B	Q_{min}	0,02 Q_n	0,03 Q_n
	Q_t	0,08 Q_n	0,20 Q_n
C	Q_{min}	0,01 Q_n	0,006 Q_n
	Q_t	0,015 Q_n	0,015 Q_n

Ce tableau donne donc, pour les compteurs dans les différentes classes métrologiques, les relations entre Q_{min} et Q_t d'une part et la valeur de référence Q_n d'autre part. Q_n est ici la moitié du débit maximal Q_{max} tandis que Q_{min} est le débit minimal auquel le compteur tient la précision exigée. Q_{min} et Q_{max} définissent les limites de l'étendue de la charge du compteur.

Dans l'étendue de la charge on distingue deux zones correspondant à deux valeurs de l'exactitude: ±5% et ±2%.

Le débit auquel la valeur de la précision change de ±5% à ±2% est appelé débit de transition Q_t.

Pour les compteurs domestiques, connus dans beaucoup de pays comme 3 m³ ou ½", dont la dénominaion selon le tableau est de $Q_n = 1{,}5\ m^3/h$, on peut donner l'exemple suivant:

TABLEAU 2

Classe B $Q_n = 1{,}5\ m^3/h$		
Q_{max}	=	3 m³/h
Q_t	=	0,120 m³/h
Q_{min}	=	0,030 m³/h

Ces compteurs doivent respecter les valeurs d'exactitude suivantes:

de 0,03 à 0,120 m³/h: ±5%

de 0,120 à 3 m³/h: ±2%

Il faut remarquer que les valeurs de la précision sont données en pourcentages du volume débité.

Pratiquement, il s'est révélé que la précision dans le mesurage comptable est bien meilleure qu'on s'y attendrait au vu des courbes de précision.

La raison en est que la courbe de fréquence des différents débits présente beaucoup de variations. Ce sont les différentes consommations (douches, bains, lave-vaisselles etc . . .) qui provoquent ces variations.

C'est sur base des recherches de la KIWA (avec un appareil développé par cet institut) qu'on peut illustrer l'influence avantageuse de ces variations dans la mesure du débit consommé.

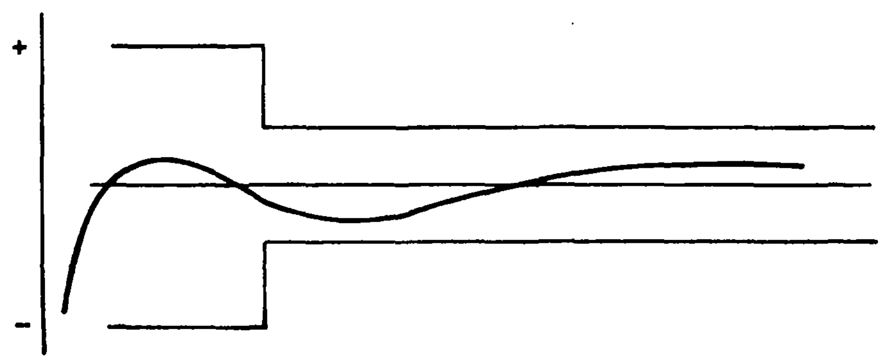

Figure 3—Courbe de précision d'un compteur de vitesse en service.

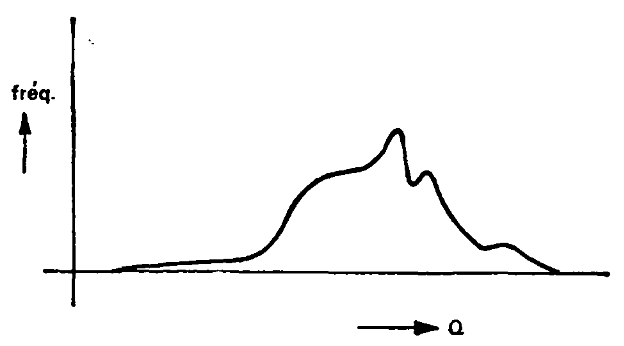

Figure 4—Courbe de fréquence des différents débits d'utilisation

A l'aide de la courbe de précision et de la courbe des fréquences des débits, le calcul de l'erreur totale dans le mesurage comptable donne 0,16%.

3 Détermination de la précision d'un compteur d'eau

Déterminer la précision d'un compteur d'eau consiste à étalonner ce compteur. Lors de l'étalonnage on fait la comparaison entre le compteur et un étalon, soit la comparaison entre les indications données par le compteur essayé et le compteur étalon (dispositif d'étalonnage) en provoquant un écoulement d'eau faisant fonctionner à la fois ce compteur et ce dispositif d'étalonnage.

Toute erreur affectant cette détermination de la précision d'un compteur d'eau (relevé de la courbe d'erreur de mesurage en fonction du débit) relève d'une des quatre catégories suivantes:

1. Erreurs du compteur.
2. Erreurs de l'étalon.
3. Erreurs de la méthode de comparaison.
4. Erreurs d'exécution (influence humaine).

On peut subdiviser chaque catégorie en plusieurs erreurs. Ci-après, on donne une énumération de ces différentes erreurs.

A. Erreurs du compteur

En plus de l'erreur principale du compteur, c'est-à-dire l'imprécision qu'on veut connaître à l'aide de l'étalonnage on peut distinguer les sources d'erreur suivantes:

a. Jeu angulaire dans le mouvement de l'aiguille tournant le plus vite.

b. Excentricité de la position de l'aiguille mentionnée sous a.

c. Frottement dans les rouleaux (seulement dans les compteurs avec totalisateur "à tambour").

B. Erreurs de l'étalon

Malgré le fait que l'étalon constitue la base de l'étalonnage, il faut néanmoins citer les sources d'erreurs suivantes:

1. Influence de la température.
2. Exactitude d'étalonnage de l'étalon lui-même (par exemple étalonnage de la cuve par le Service Métrologique).
3. Capillarité des tubes de niveau.
4. Situation au début de l'étalonnage (influence du "zéro sec" ou du "zéro noyé").
5. Inclinaison par rapport à la verticale des réservoirs de jaugeage, tubes de niveau, etc. . . .

C. Erreurs de la méthode de comparaison

Il faut distinguer les sources d'erreurs suivantes:

1. Longueurs droites de tuyaux à l'amont et à l'aval du compteur.
2. Variations dans les débits (pression) pendant l'étalonnage.
3. Influences de régimes transitoires dont on peut distinguer deux types:
   —régimes transitoires dus à l'ouverture et à la fermeture d'une vanne, réglant l'écoulement de l'eau à travers le compteur

—régime transitoire dû au basculemeut de la tuyauterie de sortie de l'écoulement à un autre réservoir.

4. Vibrations dues à l'environnement.
5. Présence d'air dans le compteur et/ou dans la tuyauterie de l'installation d'étalonnage.

D. Erreurs d'exécution (influence humaine)

Lors de la détermination de la précision des compteurs, l'homme aussi introduit des erreurs, lorsque l'exécution des essais n'est pas automatique. Ces sources d'erreurs sont:

1. Aptitude insuffisante de l'oeil (astygmatisme).
2. Parallaxe à la lecture du compteur et de l'étalon.
3. Interprétation surtout quand les indices (aiguilles) se trouvent entre deux traits sur l'échelle.
4. Appréciation de l'état de l'installation (par exemple: étanchéité du clapet dans le fond de la cuve de jaugeange).

4 Considérations rélatives aux erreurs élémentaires

On peut constater de l'énumération donnée dans le paragraphe précédent que pratiquement une seule méthode d'étalonnage est considérée dans la présente contribution.

Cette méthode, dite "classique", est schématisée à la figure 5 et implique la procédure suivante:

i. Lecture du compteur et calcul du volume débité selon les deux lectures. Contrôle du niveau dans la cuve.

j. Calcul de l'erreur de la façon suivante:

$$\frac{\text{Différence entre les deux lectures—volume dans la cuve}}{\text{volume dans la cuve}} \times 100\%$$

a. Erreurs du compteur

Pour combattre les erreurs du compteur dues au jeu angulaire et/ou de l'excentricité il est nécessaire que l'aiguille qui tourne le plus vite fasse un nombre entier de révolutions.

Vis-à-vis du frottement il y a deux conditions à respecter:

—ne pas toucher le compteur pendant l'écoulement ni avant la lecture

—éviter l'étalonnage pendant que le tambour (si présent) des les plus petites unités se déplace de 9 à 0 (à ce moment ce tambour entraîne le ou les tambours précédents).

b. Erreurs de l'étalon

En ce qui concerne les erreurs de l'étalon dues à l'influence de la température il est assez rare qu'il soit nécessaire de corriger les résultats de l'étalonnage. Néanmoins il est en général assez facile de juger de la nécessité d'appliquer de telles corrections parce qu'elles se rapportent à des phénomènes physiques.

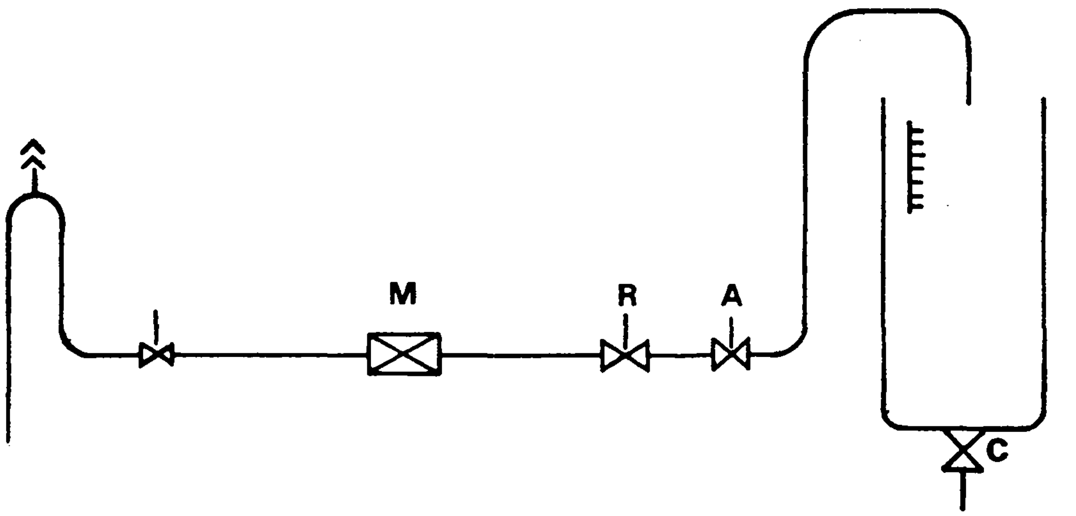

Figure 5—Méthode d'étalonnage "classique".

Procédure

a. Evacuer l'air de la tuyauterie et du compteur.

b. Remplir la tuyauterie et le compteur "M" avec de l'eau.

c. Règler le débit auquel on veut déterminer la précision du compteur avec une vanne de réglage (R).

d. Fermer la vanne d'arrêt (A).

e. Après vidange de la cuve de jaugeage, fermer la soupape dans le fond de la cuve (C).

f. Lecture du compteur.

g. Ouverture de la vanne d'arrêt (A) à grand-ouvert (la cuve commence à se remplir). Ajustage—si nécessaire —de la vanne de réglage pendant l'écoulement.

h. Au moment où le niveau dans la cuve atteint le niveau indiqué, fermer le robinet d'arrêt (A).

Quant aux autres erreurs, dues à l'étalon il y a certain nombre de régles à respecter à ce sujet:

—tenir compte (si nécessaire) de l'exactitude déclarée par le Service de Métrologie à propos du volume des cuves de jaugeage

—utiliser le volume total de la cuve ou utiliser la partie supérieure de la cuve (de 50% à 100%—"zéro noyé") dans ce dernier cas on doit bien sûr réduire l'exactitude déclarée des cuves

—garantir la position verticale de la cuve (ajouter un dispositif de contrôle—fil à plomb)

—employer des tubes de niveau suffisamment larges, un diamètre intérieur de 12 mm minimum est recommandé. Nettoyer régulièremence tubes et les électrodes s'il y en a

—pour les cuves construites en matériaux absorbant l'eau, il faut veiller à ce que ces matériaux ne sèchent pas. Dans ce but, les cuves doivent toujours être pleines d'eau lorsqu'elles ne sont pas utilisées

—quant les cuves sont utilisées entièrement ("zéro sec"), il importe qu'on utilise toujours la cuve de la même façon; cela veut dire par exemple que l'intervalle de temps entre la fermeture de la soupape dans le fond de la cuve et l'arrêt de l'écoulement est fixe.

c. Erreurs de méthode

Parmi les erreurs citées dans le paragraphe III on peut distinguer trois sources d'erreur principales:

—l'air (dans le compteur et la tuyauterie)

—les perturbations dans l'écoulement

—les régimes transitoires.

L'air dans le compteur et la tuyauterie peut entrer pendant le montage du compteur mais peut aussi être entraîné par les pompes. Il n'est pas toujours possible d'éviter l'introduction de l'air, mais il est indispensable que l'on prenne des précautions pour l'enlever à l'aide d'une purge ou d'un extracteur.

Les perturbations dans l'écoulement peuvent avoir une grande influence sur le compteur, surtout le type Woltmann horizontal. Ces perturbations sont engendrées par des irrégularités dans les tuyaux, directement en amont et—avec moins d'influence—directement en aval des compteurs.

Parmi ces irrégularités, les vannes (partiellement ouvertes) et les courbes sont les plus connues. Cependant on possède peu d'informations sur ce problème à cause d'une part de la grande variété d'irrégularités possibles et de la grande variété de susceptibilités des compteurs à ce sujet, mais d'autre part et surtout, à cause des frais de recherche très élevés.

Sur base des données et recherches des différents fabricants il y a quelques recommandations pratiques qu'on peut suivre pour déterminer les longueurs droites en amont et en aval des compteurs. Ces indications—sans engagement—figurent dans les tableaux 3 et 4. (La valeur D dans les tableaux est le diamètre intérieur du tuyau, correspondant au diamètre nominal du compteur.)

TABLEAU 3

Détermination de la précision		
longueur droite sans irrégularité	Compteur Woltmann	
	horizontal	vertical
en amont	$10\times D$	$3\times D$
en aval	$3\times D$	$1\times D$

TABLEAU 4

Détermination de la pression (prise de pression)	
longueur droite sans irrégularité	
en amont de la prise de pression	$8\times D\ldots 10\times D$
en aval de la prise de pression	$3\times D\ldots 5\times D$

Il faut souligner que les grandeurs mentionnées dans ces tableaux font l'objet d'habitudes dans certains pays mais qu'il faudrait des études plus poussées pour en confirmer la valeur scientifique.

Quant aux régimes transitoires c'est avec des représentations graphiques qu'on peut le mieux expliquer les deux situations possibles, à savoir:

—régime transitoire à débit constant (figure 6)

—régime transitoire à débit non-constant (figure 7).

Dans le premier cas, il y a quelques recommandations à suivre pour éviter une trop grande influence du régime transitoire, à savoir:

—$t_1 \approx t_2 \leqslant 0{,}5$ secondes, et courbes symétriques pendant le basculement de l'écoulement

—$t \approx 50t_1$.

Dans le deuxième cas, les règles suivantes sont pratiquées aux Pays-Bas:

—temps d'étalonnage $\geqslant 3$ minutes

—$t_1 \approx t_2 \leqslant 1$ seconde.

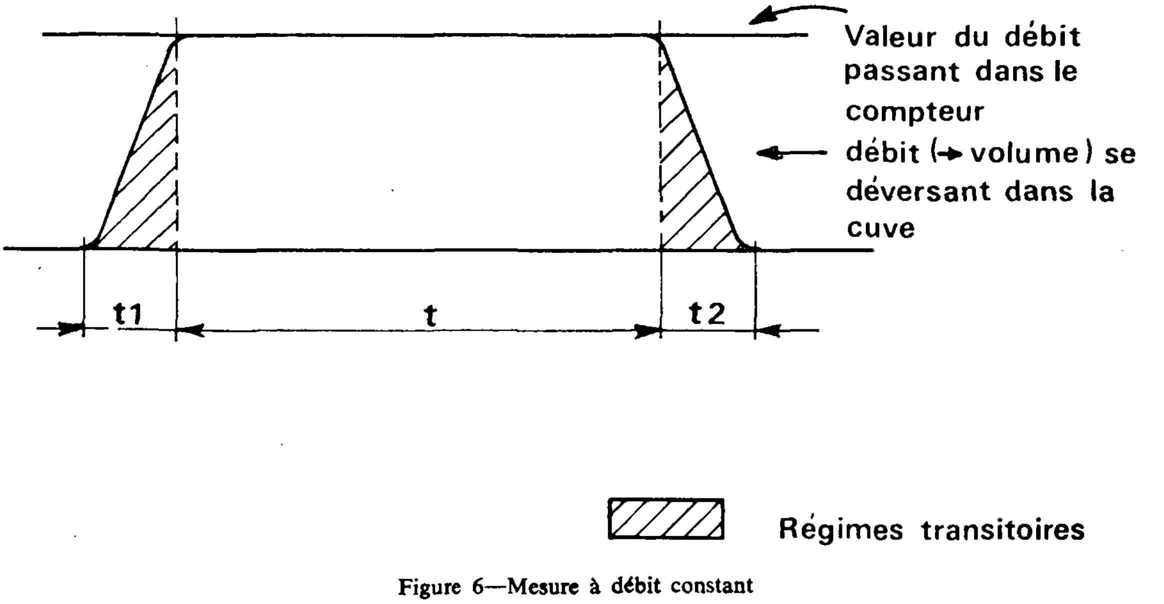

Figure 6—Mesure à débit constant

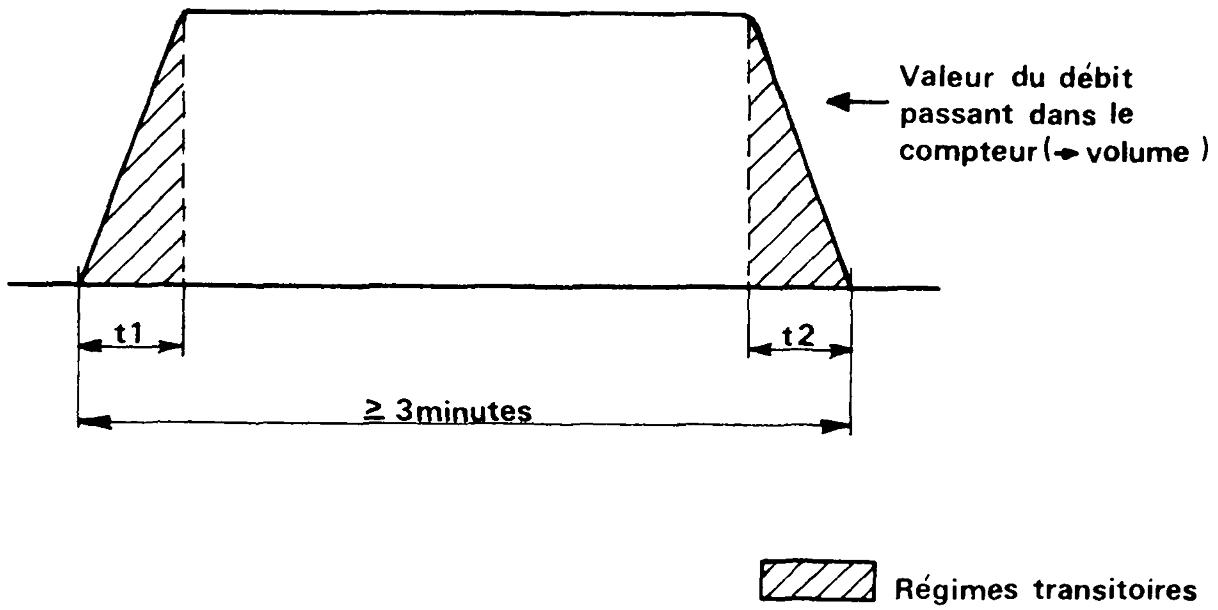

Figure 7—Mesure à débit non-constant (incluant le démarrage et l'arrêt).

d. Erreurs d'exécution (influence humaine)

Bien que cela soit évident, mentionnons qu'il est nécessaire de vérifier la vue du contrôleur au banc d'essai (astygmatisme). Pour éviter l'influence de la parallaxe on peut utiliser un tube par lequel le contrôleur fait la lecture. Néanmoins la pratique a montré qu'un contrôleur expérimenté peut très bien éviter la parallaxe.

Etant donné le fait que l'aiguille et l'échelle de vérification sont assez petites on doit tenir compte d'une imprécision d'environ 1 mm pour l'ensemble des deux lectures.

Si on suppose que l'imprécision introduite lors de la lecture du compteur ne doit pas dépasser 10% de l'erreur maximale tolérée on peut déduire que l'aiguille de l'échelle de vérification doit parcourir une distance de 200 mm entre Q_{min} et Q_t et une distance de 400 mm entre Q_t et Q_{max}.

Pour les mêmes raisons une hauteur de 10 mm dans la cuve de jaugeage doit représenter moins de 1% du volume débité, une hauteur de 15 mm étant préférable.

5 Économie à l'étalonnage

Il est bien évident que l'imprécision admissible à l'étalonnage dépend fortement des erreurs maximales tolérées pour le compteur (dans l'hypothèse considérée ici 5% et 2%).

En outre, le nivellement des fréquences des débits de fonctionnement n'exige pas une trop grande sévérité concernant cette imprécision admissible.

Pour cette raison on pourrait s'écarter de la coutume générale consistant à choisir pour l'imprécision une valeur maximale de 10% de l'erreur maximale tolérée pour le compteur. Sous les réserves nécessaires on pourrait peut-être choisir une valeur de 20 ou 25%.

L'imprécision à l'étalonnage peut donc être considérée comme une partie de l'erreur maximale tolérée du compteur. La figure 8 donne une idée de cette représentation.

La surface du grand rectangle représente l'erreur maximale tolérée pour le compteur (soit $\pm 5\%$ ou $\pm 2\%$), tandis que le petit rectangle représente l'imprécision totale admissible pour l'étalonnage (par exemple 20% de l'erreur maximale tolérée).

Selon l'analyse faite dans le chapitre II de cette contribution on peut subdiviser le petit rectangle de cette même figure en 4 parties représentant les quatre catégories d'erreurs.

De cette subdivision on peut conclure qu'il y a un grand nombre de possibilités de répartition de l'imprécision sur les différentes catégories, tout en respectant bien sûr le pourcentage admis (20%).

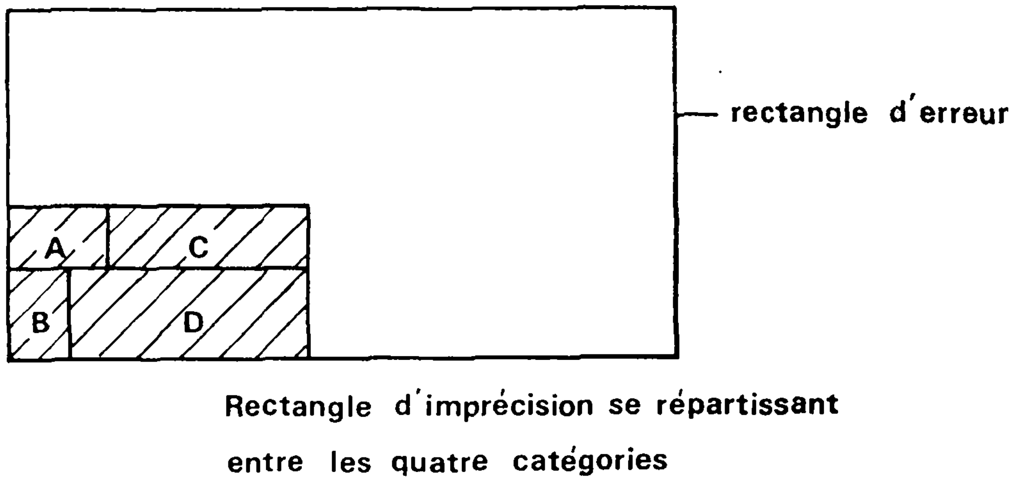

Figure 8—Rectangles d'erreur et d'imprécision.

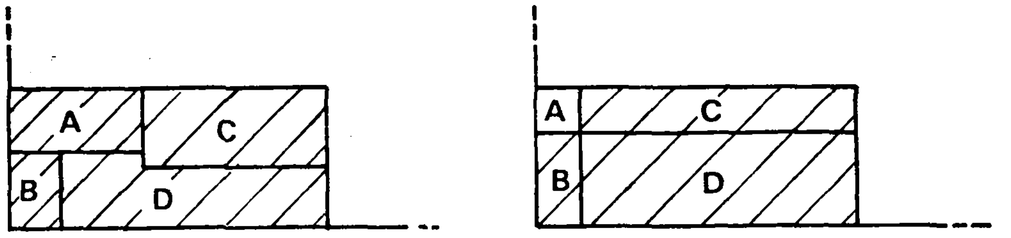

Figure 9—Deux possibilités de répartition des quatre catégories dans le rectangle d'imprécision.

Ceci est représenté à la figure 9.

Si l'on considère le fait que la plus grande cause d'erreur à l'étalonnage est constituée par la lecture du compteur, on voit bien que l'on peut faire des économies à l'étalonnage si l'on peut améliorer cette lecture.

Les fabricants de compteurs et les ateliers de réparation ont pour cette raison fait des développements dans cette directive. Entre d'autres solutions, on peut signaler les méthodes et réalisations suivantes:

—ajouter une échelle de vérification (dans le sens des sous-multiples, par exemple une échelle dont l'échelon est le 0,1 l)

—obtenir un signal électro-magnétique de haute fréquence du compteur (par exemple de l'hélice)

—obtenir un signal optique de haute précision du compteur (par exemple de l'étoile tournante).

De la sorte on peut aboutir à une situation dans laquelle la lecture du compteur a été améliorée. Cela signifie, toutes choses égales par ailleurs, une diminution du petit rectangle d'étalonnage dans la figure 9.

Mais on peut profiter de cette situation pour diminuer la sévérité relative à une ou plusieurs autres imprécisions.

Il vaut la peine de bien étudier cette possibilité parce qu'en général elle permet de diminuer le temps d'étalonnage qui est assez important (frais en personnel, occupation de l'installation).

6 Perte de pression

Bien que la détermination des pertes de pression n'appartienne pas au domaine de la précision, il est utile de signaler quelques conditions à respecter lors de ces essais.

D'une façon générale, pour déterminer la perte de pression, on détermine la pression en amont et en aval du compteur, à différents débits. Les prises de pression existant dans ce but sont sensibles aux perturbations dans l'écoulement et c'est pour cette raison qu'il faut respecter des longueurs droites en amont et en aval de chaque prise de 8 . . . 10 × D et 3 . . . 5 × D respectivement. D étant le diamètre intérieur du tuyau concerné (voir aussi tableau 4).

Il y a plusieurs modèles existants pour les prises de pression, les schémas de principes importants sont donnés dans les figures 10 (modèle allemand—DVGW), figure 11 (modèle français—CSTB) et figure 12 (modèle néerlandais—KIWA).

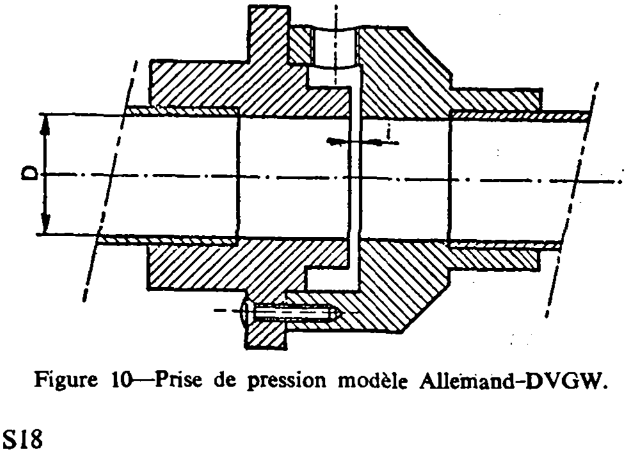

Figure 10—Prise de pression modèle Allemand-DVGW.

L'expérience a montré que ces trois modèles ne donnent pas de différence en pratique.

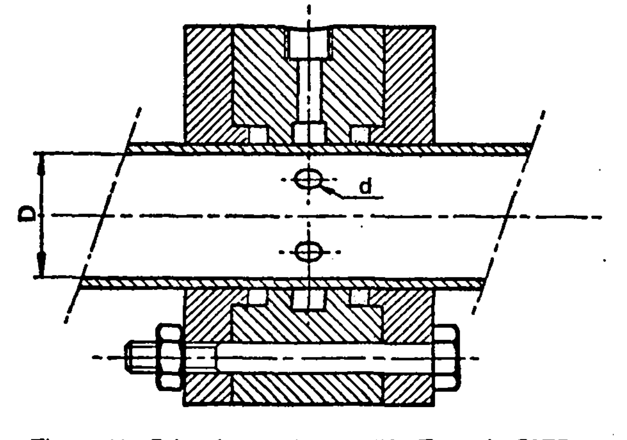

Figure 11—Prise de pression modèle Francais-CSTB.

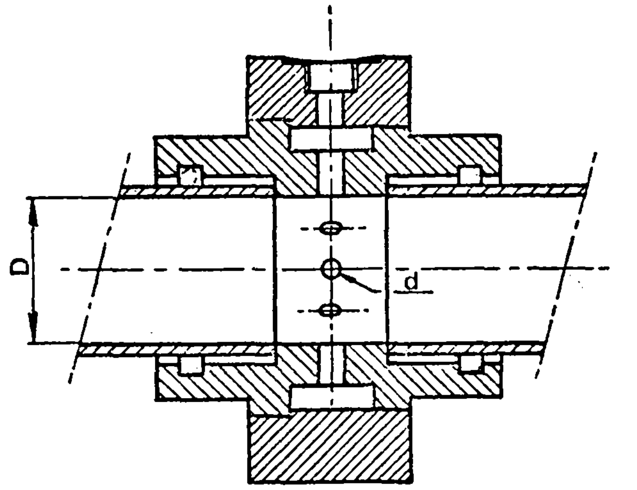

Figure 12—Prise de pression modèle Néerlandais-KIWA.

7 Endurance

Tout acheteur de compteur d'eau aime connaître la solidité de cet appareil dans les conditions normales d'emploi. Etant donné que ces conditions présentent de grandes différences d'un service d'eau à un autre, c'est surtout l'expérience qui doit fournir les indications valables aux acheteurs.

Dans le cas où cette expérience est absente (nouveau type, marque inconnue), les experts devront se contenter d'un jugement à propos des détails techniques de construction. En général un tel jugement tient compte de l'influence de la nature de l'eau sur l'usure ainsi que des possibilités d'entretien du compteur.

Neanmoins, les services d'eau ont besoin d'une méthode concrète relative à l'endurance.

La méthode la plus simple est de faire parcourir par le compteur un grand volume d'eau au débit maximal. Selon notre expérience cette méthode ne révèle que les faiblesses de construction, et ne donne aucune information sur l'endurance et la précision du compteur. C'est pourquoi on a commencé à pratiquer une méthode dans laquelle le compteur est soumis à des mises en marche et arrêts répétés, à peu près comme un emploi normal. Sur base de recherches sur six à sept milliers de compteurs domestiques on a pu montrer une certaine relation entre le comportement pendant des années de service et les résultats des essais d'endurance.

Un exemple de tels essais figure dans la directive CCE, déjà citée.

A côté de cette méthode, il semble utile de faire des essais—en ce qui concerne les compteurs domestiques de vitesse—avec des débits changeant de sens.

Bien que l'on ne dispose pas encore de résultats suffisants à propos de cette méthode, il semble qu'elle puisse donner des informations plus rapides en ce qui concerne un jugement prompt des compteurs d'eau.

8 Remarques finales

Partant de l'idée que le but de cette contribution était de vous brosser un tableau d'ensemble des bancs d'essai pour compteurs d'eau, ce document ne donne que des généralités à ce sujet, soulignant quelques considérations importantes pour les distributeurs d'eau.

Un grand nombre d'aspects méritent des recherches approfondies. Comme point le plus important, il faut signaler l'attention à accorder aux longueurs droites en amont et en aval des compteurs (ou prises de pression), parce que ces longueurs peuvent largement influencer l'imprécision de mesurage. D'autre part, si ces longueurs, sont trop importantes, les frais d'installation sont exagérés. Il serait intéressant de faire en collaboration des recherches dans ce domaine afin d'obtenir des résultats le plus tôt possible avec des dépenses réduites.

En plus de ces points importants, d'autres aspects mériteraient aussi de faire des recherches détaillées afin d'aboutir à des méthodes plus efficaces et plus économiques.

TABLEAU 5

Débit nominal Q_n m³/h	Débit à l'essai	Genre d'essai	Nombre d'interruptions (cycles)	Durée des interruptions	Durée d'1 période d'écoulement	Durée de démarrage et de ralentissement
$Q_n \leq 10$	Q_n	Discontinu	100 000	15 secondes	15 secondes	0,15 (Q_n)* secondes 1 s minimal
	$2Q_n$	Continu			100 heures	
$Q_n > 10$	Q_n	Continu			800 heures	
	$2Q_n$	Continu			200 heures	

* (Q_n) étant un nombre égal à la valeur de Q_n en m³/h

Summary

When testing the accuracy of water meters, a great number of failures or influences may interfere with the result of testing. These failures or influences may be classified into four main groups i.e.

1. *Failures from the meter:*
   a. Angular play of the counter.
   b. Eccentricity of the pointer.
   c. Friction in the rollers.
2. *Failures from the standard:*
   a. Influence of temperature.
   b. Accuracy of the standard.
   c. Capillarity of gauge glass.
   d. Starting situation before gauging ("wet" or "dry" zero).
   e. Oblique position of reservoir/gauge glass.
3. *Failures from the system:*
   a. Undisturbed lengths of pipe on upstream and downstream side of the meter.
   b. Variations in flow rate through meter under test.
   c. Influence of transitional state within two possibilities:
      —when opening and closing supply valve
      —when changing direction of outlet-pipe to another tank
      —external vibrations
      —presence of air in meter and/or pipe system.
4. *Failures from human influence:*
   a. Insufficient visual faculty.
   b. Parallax at reading of meter and standard.
   c. Insufficient judgment of readings.
   d. Insufficient judgment of technical functioning of installation.

Based on a rough review of different failures it is indicated that the permissible total uncertainty in the final test result depends largely on the value for the meter accuracy. Whilst this total is mostly 10 to 20% of the permissible meter accuracy, this indicates that a particular failure may be slightly greater when another failure is restricted, all with respect to the permissible total uncertainty. Thus there is the possibility of carrying out water meter tests more economically.

Besides accuracy testing, the testing of head loss and endurance is also of great importance.

With the aid of technical information it is indicated that there are several possibilities for pressure determination in hydraulic measurements. The available information on methods for endurance testing is growing slowly but is far from being fully sufficient. Thus there is a need for further investigation by water authorities to provide the elements for setting up a quick assessment procedure for water meters.

A further need remains in relation to the undisturbed lengths on the upstream and downstream side of water meters, when tested for accuracy and hydraulic measurements. In view of the extensive and expensive research in this field it would far be better if this research were carried out on a collaborative basis.

Water meter maintenance

by M. R. Williams

Vice-President, Indianapolis Water Company

In order to report current water meter maintenance practices, a questionnaire was mailed to water utilities of 110 cities in the United States. The survey included both privately and municipally owned water utilities ranging in size from 10 000 service connections to 1 800 000 service connections providing a good cross section of the industry. Seventy-six or about 69 percent of the utilities surveyed responded by returning completed questionnaires, which reveals a great deal of interest in this subject.

Even though the questionnaire covered all sizes of meters, the answers were primarily limited to the small meters with 75,60 l/h capacity, which is the meter predominately used by utilities in the United States to measure residential consumption. For this reason and for the sake of brevity, only this meter will be discussed here. Following are highlights of the survey:

(1) Only about ½ of the utilities surveyed test all new meters before they are placed in service; the others either test them on a sampling basis or depend altogether on the integrity of the manufacturer's test. Most utilities require the manufacturer to supply a "Certification of Test" for all new meters delivered.

(2) All of the utilities surveyed require new meters to be tested at 3 standard rates of flow—low, intermediate, and high. Most of the utilities require new meters to record with an accuracy of not less than 95 percent nor more than 101,5 percent at a low-flow test rate of 0,90 l/h. For most, an accuracy of not less than 98,5 percent nor greater than 101,5 percent is required for the intermediate and high rates of flow, which range from 3,75 l/h to 11,34 l/h for the intermediate flow test, and 34,02 l/h to full flow for the high flow test.

(3) When asked if they have a regular meter change programme, 59 replied "Yes". Forty-seven of these said they routinely change meters on the basis of years of continuous service ranging from 3 to 20 years, with most falling in the 10- to 15-year bracket. The remaining 12 have a programme of changing on a years-of-service or registration-limit basis, whichever comes first. The registration limit most often mentioned is 2 832 m³. The utilities that replied "No" to this question presumably remove meters when they become suspicious or break. Fourteen of the companies that have a regular change programme reported that the programme is not up to date.

(4) Forty-eight utilities reported their meters are tested on being removed from service under a regular meter change programme based on years of service or a registration limit. The others send the meters directly to the repair bench for dismantling and repair or replacement without a test. Out of the 48 utilities that test all meters, 29 send them to the repair bench even though the tested meters meet the required accuracy standards on all three test flows. The remaining 19 said their meters are returned to service without being sent to the repair bench if the meters meet the required accuracy standards. Apparently the feeling here is that since the meter is functioning properly and no wear is indicated, the meter cannot be improved by dismantling it and replacing parts. Such a meter is treated the same as a repaired meter and is returned to service for another full service period. This procedure is highly debatable.

(5) In general, the repair procedure used by most of the utilities surveyed includes complete disassembly, mechanical, manual or solution cleaning, examination of parts, replacement of parts that are worn, and reassembly. About 40 percent of the utilities used an "assembly line method of repair"; the rest reported repairs made on a one repairman per meter basis.

(6) With few exceptions, the accuracy standards and rates of flow used in testing repaired meters are the same as for new meters except that an accuracy of not less than 90 percent is required for the low test flow of repaired meters (not less than 95 percent is required for new meters).

(7) Only 19 of the utilities surveyed reported they had made a study to determine the most economical period of time meters should be left in continuous service. Many of the others indicated they were either in the process of making such a study or planned to make one at some future date. Most others indicated a need for such a study. Several of the utilities submitted copies of their completed studies. The methods used and results obtained from the studies varied, but each had one thing in common—the study caused the utility to make improvements in its meter maintenance programme, which resulted in increased revenue or lower meter maintenance cost, or both. In order to illustrate the varied nature of the studies, two of them are discussed briefly below:

(1) One major utility tabulated the test results of all residential meters returned to its meter shop having service periods of between 15–23 years or with registration exceeding 5 664 m³. Such tabulating extended over a period of 2½ years. Test accuracy standards of 98,5 percent to 101 percent at a flow rate of 7,56 l/h and 65 percent at a flow rate of 0,47 l/h were used. After studying the test results, the utility concluded that the service period of its residential meters could be extended from 10 years to 15 years with no significant loss of meter accuracy. This, of course, reduced annual meter maintenance costs substantially.

(2) Another utility decided, after careful study, that because of its high meter repair cost it was profitable to replace all of its old mechanical driven meters with new magnetic driven meters over a period of ten years. It was concluded that this replacement programme would result in a substantial increase in meter registration with a corresponding increase in revenues. The replacement programme allowed the utility to drastically reduce the number of employees in its meter repair division and practically eliminate its inventory of meter repair parts.

It is obvious that water utility operators nationwide recognize the importance of purchasing meters that operate with a high degree of accuracy at low rates of flow. Experience has shown that under-registration because of a poorly designed or a carelessly repaired meter is most likely to occur at rates of flow below 3,78 l/h. If such a meter is not rejected at the test bench and is allowed to be placed in service, it will run for many years before it finally stops, and for all of these years, it will lose revenue for the utility.

The problem of accuracy decline at low rates of flow is clearly illustrated by bar graphs in Figure 1. Referring to these graphs, which are based on a random sample of test results of approximately 200 mechanical drive meters for each service life shown, you will see that such meters in the Indianapolis system are recording a high degree of accuracy at high rates of flow after 20 years of continuous service. High accuracy is maintained at the intermediate flow rates for at least ten years, but drops off slightly after

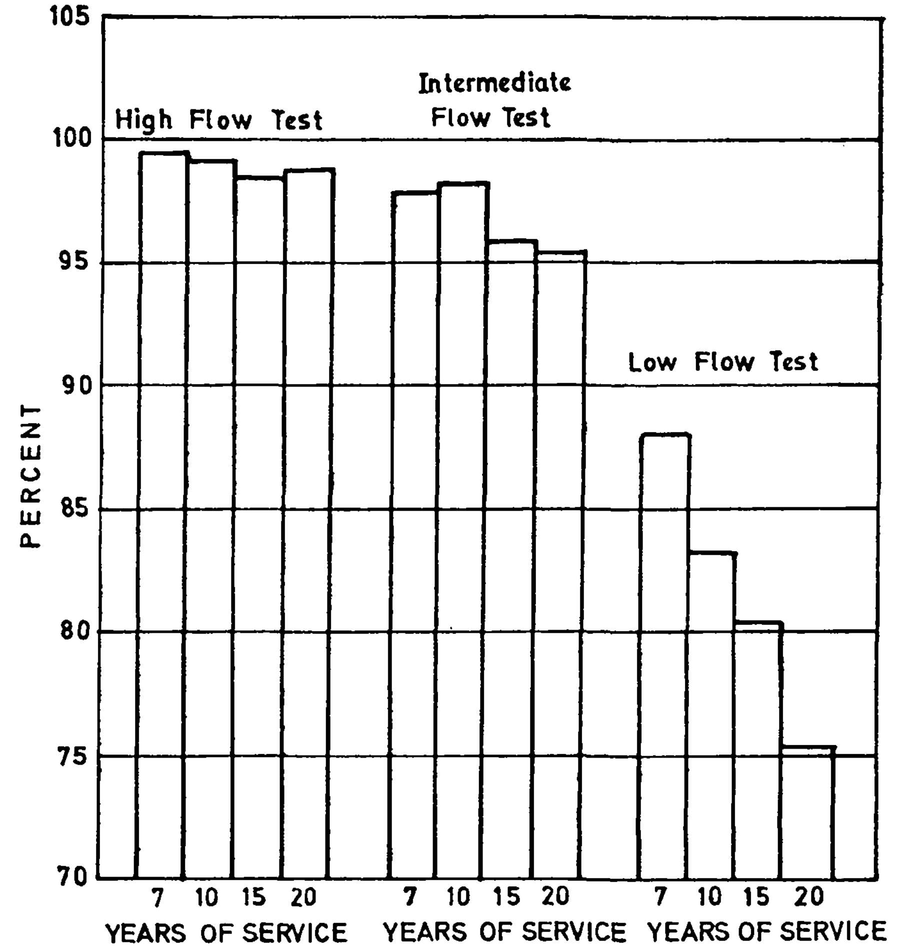

Figure 1—Average meter accuracy at standard rates of flow by years of continuous service

*Source—Study by Indianapolis Water Company to determine economical service life of residential mechanical drive meters.

that point of time. Accuracy reduction at low rates of flow is more pronounced. After only 7 years of continuous service, low flow accuracy drops to 87,2 percent, and after 20 years has dropped to 75,2 percent.

A number of studies have been made to determine the percent of water used at various rates of flow. Table 1 considers the results of 4 different studies reported by W. D. Hudson, of Pitometer Associates, in 1964. According to this report, 23,2 percent of the total amount of water passing through residential meters is at rates less than 3,78 l/h. Thirteen percent is used at a 0,90 l/h rate or less. Undoubtedly, piping and fixture leaks account for a large portion of the water passing through meters at low rates of flow.

The question of how long to leave a meter in service has long troubled the waterworks industry. Most utility managers will agree that to be fair to both the consumer and the utility, meters must be maintained at some regular interval. For many utilities, the period between tests has been established by state public utility commissions or has been arbitrarily selected by the utility. The survey reveals that few have actually made an economic evaluation to determine the most economical change period.

The length of time a meter will accurately measure water varies considerably from utility to utility. Water quality plays an extremely important part in accuracy. Meter parts wear unusually fast if the water supply carries sand or fine sediment. Some waters have corrosive properties that cause rapid deterioration. Other waters have a deposit providing a protective coating on the interior parts of the meter, which actually improves the accuracy for a period of time. The quality of the meter and amount of care taken in testing and repairing it are directly related to service life and sustained accuracy.

The question of "When to remove and to test a meter?" then must involve a study made by each utility to establish an economic balance between revenue loss because of reduced accuracies and the cost to remove, transport, test, and repair (or replace) the meter. There are two methods that can be used for studies such as this: One method involves removing from service meters of various service ages and testing them in the meter shop. The test accuracies are then compared with new or repaired meter accuracies and the difference in registration gives the degree of error. The second method is to install repaired or new meters in tandem with meters of various service lives in the customers' homes. This allows the under-registration to be read directly. Each method has its advantages and disadvantages.

The Indianapolis Water Company has completed a study (using method 1 described above) to determine the economic service life of its 135 000 mechanical residential meters. This study indicates that a 9-year service life provides the proper balance between revenue loss and the cost of removing and repairing meters. Another study using system 2 above is in progress, but the results are not yet available. Following is a brief outline of the procedure used in the completed study together with illustrations:

Step I

(A) Using an acceptable statistical sampling of test results on meter records determine average meter accuracy for each of the 3 standard rates of flow. The illustration below was based on a sample of 200 meters tested during the 10th year of continuous service.

Illustration

Test Rate of Flow	*Average Percentage of Accuracy*
Minimum (0,9 l/h)	83,7
Intermediate (3,78 l/h)	98,3
High (37,80 l/h)	99,5

(B) Using a similar sampling method establish average accuracy for each standard rate of flow for repaired meters:

Illustration

Test Rate of Flow	*Average Percentage of Accuracy*
Minimum (0,90 l/h)	96,5
Intermediate (3,78 l/h)	99,8
High (37,80 l/h)	99,7

(C) Determine the percent of water used at each test rate of flow. The percentages used in this illustration are taken from Table 1.

Illustration

Test Rate of Flow	*Percentage of Total Consumption*
Minimum	16,4
Intermediate	63,1
High	20,5

(D) Determine average under-registration per meter by subtracting weighted average of in-service meters from weighted average of repaired meters. The weighted averages are computed by multiplying the percentage of water used at each flow rate by the average accuracy at each standard flow rate.

Illustration

	Percent
Repaired meter weighted accuracy	99,2
In-service meter weighted accuracy	96,1
Average under-registration	3,1

(E) Determine average annual water consumption for the size of meter being studied.

Illustration

339,84 m^3/yr.

(F) Apply the under-registration percentage to the average water consumption.

Illustration

339,84 m^3/yr × (0,031) = 10,53 m^3/yr.

(G) Compute revenue loss in the 10th year by multiplying the under-registration by the applicable water rate:

Illustration

10,53 m^3/yr (\$0,61/2,83 m^3) = \$2,27/yr.

Note: Repeat this procedure for meters with other years of continuous service.

Step II

Calculate average cost of changing, transporting, testing, and repairing meters:

Illustration

Average cost of changing and bringing meter to meter shop	\$ 5,00
Average cost of testing and repairing meter	\$ 9,00
Total cost	\$14,00

Figure 2 shows graphically the results of the Indianapolis Water Company study. On this graph, the "Average Accumulated Revenue Loss per Meter" curve was established by computing the average revenue loss of groups of meters with service ages from 1 to 15 years. It can be seen the Average Accumulated Revenue Loss per meter curve intersects the Average Meter Repair and Change Cost curve at 9 years continuous service, which is the period of time Indianapolis Water Company mechanical water meters should be left in service to provide a proper balance between lost revenue and the cost of meter maintenance. In other words, a meter should be removed from service, and repaired when the estimated accumulated revenue loss because of under-registration equals the estimated cost of removing and repairing the meter.

It should be noted here that the same procedure may be used to determine the most economical meter replacement time interval by comparing the accuracy of in-service meters with new meter accuracy.

Meter manufacturers have improved their product considerably in recent years. Improved design such as more wear resistive materials, lower head losses, magnetic drives and sealed registers have resulted in improved and longer sustained accuracies. It is almost certain that the magnetic drive meters developed in the early 1960's have sustained accuracy capabilities which far exceed the old mechanical drive meters. A study of the test results of a sampling of magnetic drive meters in service in the Indianapolis system for 15 years may have an economical service life of fifteen years as opposed to the 9-year service life of mechanical meters. If this is confirmed, it may be more economical to replace the old mechanical meters with new magnetic meters than to repair the old mechanical meters every 9 years.

Ideally, a meter change programme would be one that allows old mechanical meters to be replaced with new magnetic meters on an orderly change-out programme. There are literally millions of old mechanical meters in service in the United States which eventually must be retired. This will not happen, however, until the cost of new meters more nearly matches the cost of repairing the old meters. Meter companies are challenged to manufacture a less expensive meter. There already appears to be a trend in this direction. By substituting synthetic materials for bronze and making the meter smaller, this is being accomplished. Perhaps in the not too distant future, utilities will justifiably discontinue the practice of repairing old mechanical meters and instead replace them with meters that have longer economical service lives.

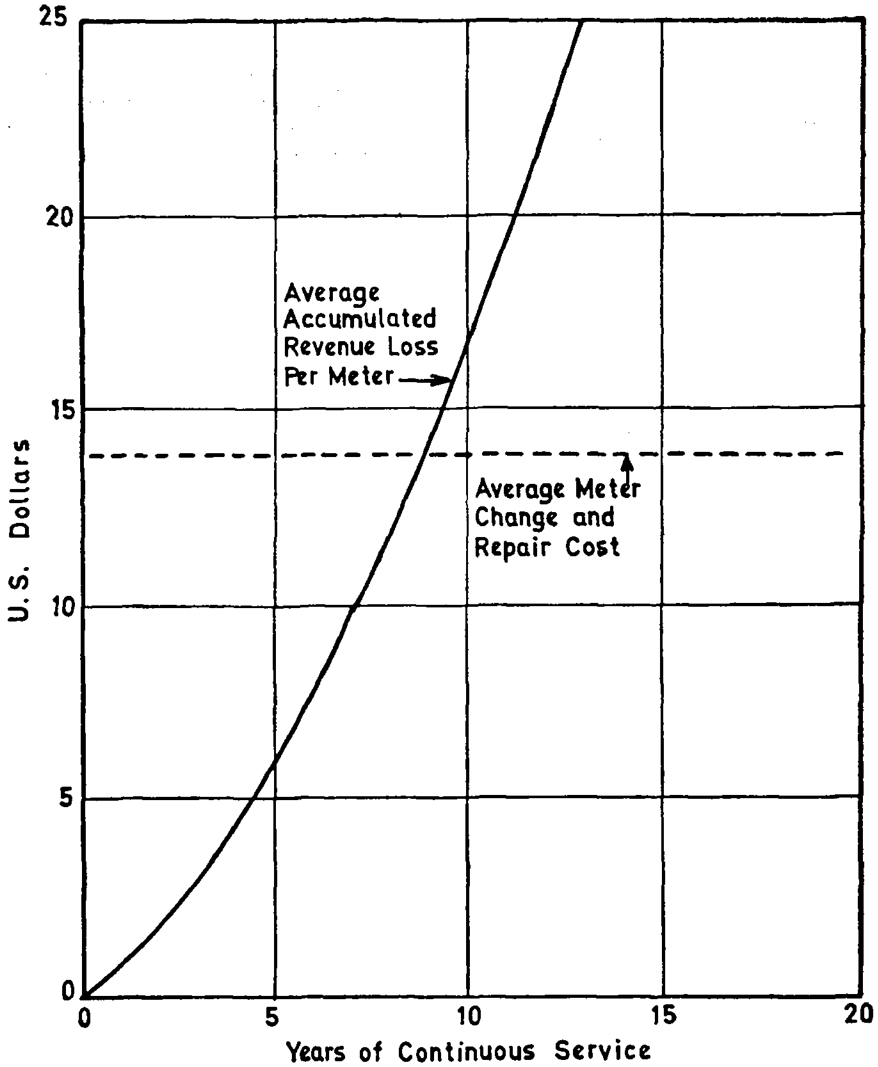

Figure 2—*Effect of meter age on revenue loss—average cost of meter change and repair*

*Source—Study by Indianapolis Water Company to determine economical service life of residential mechanical drive meters.

Summary

Characteristically, residential water meters lose accuracy at the low ranges of flow because internal parts wear. The rate at which wear takes place varies considerably from utility to utility because of the different properties of the water supplies. Studies indicate that as much as 23 percent of the total amount of water passing through residential meters is at rates of flow in which the meter is least efficient. Because of this, if meters are left in service too long, large amounts of revenue will be lost by the utility. Each utility should establish a regular interval between meter tests to minimize this revenue loss. No single time interval or registration limit will apply to all utilities. Each utility must make its own study and establish a testing interval that will economically balance loss of revenue with the cost of removing, testing, and repairing (or replacing) its meters.

TABLE 1

PERCENTAGE OF TOTAL FLOW THROUGH DOMESTIC METERS AT VARIOUS RATES OF FLOW

Rate of flow l/h	Water used percent of total
— - 0,90	13
0,90- 1,86	3,4
1,86- 3,78	6,8
3,78- 7,56	13,3
7,56-15,12	43
More than 15,12	20,5

Source: Hudson, W. D. "Reduction of Unaccounted-for Water", Journal American Water Works Association, Vol. 56, No. 2, February, 1964.

Résumé

Le présent rapport traite de la surveillance de petits compteurs aux Etats-Unis et de la question de savoir combien de temps de tels compteurs peuvent rester en service continu. Les compteurs en question ont une capacité de 75,60 l/h; ce sont ces compteurs qui sont principalement utilisés par les services de distribution d'eau aux Etats-Unis pour mesurer la consommation domestique.

Afin de rendre compte de la pratique actuelle d'entretien des compteurs d'eau domestiques, un questionnaire a été envoyé aux sociétés de distribution d'eau de 110 villes des Etats-Unis ayant entre 10 000 et

1 800 000 branchements. Septante-six des sociétés consultées ont répondu. Les réponses aux questions sont reprises en bref ci-dessous:

1° Seulement ⅓ des sociétés consultées essayent les nouveaux compteurs avant leur mise en service. Dans les autres cas, les essais sont faits par les fabricants.

2° Toutes les sociétés interrogées exigent des essais des nouveaux compteurs à trois débits (bas, intermédiaire, élevé). Le taux admissible d'erreur pour chaque débit essayé est donné.

3° Cinquante-neuf des sociétés disposent d'un programme régulier de remplacement des compteurs, basé sur le nombre d'années de service continu ou basé sur un total enregistré. Les sociétés n'ayant pas un programme régulier de changement de compteurs attendent probablement l'arrêt des compteurs avant de les enlever pour réparation.

4° Quarante-huit sociétés essayent tous les compteurs qui sont enlevés suivant un programme régulier de remplacement. Les autres sociétés les envoient directement au banc de réparation sans essai.

5° On donne une brève description de la procédure de réparation utilisée par la plupart des sociétés. Environ 40% des sociétés réparent leurs compteurs selon la méthode de "montage à la chaîne". Les autres font la réparation sur base d'un réparateur par compteur.

6° La plupart des sociétés exigent les mêmes taux de précision et les mêmes débits d'essais pour les compteurs réparés que ceux exigés pour les nouveaux compteurs; on n'exige néanmoins qu'une précision de 90 pour cent pour l'essai des compteurs réparés au petit débit (95% pour les nouveaux compteurs).

7° Seulement 19 des sociétés ont fait une étude pour déterminer la durée la plus économique de maintien des compteurs en service. Deux de ces études sont commentées.

L'expérience a démontré que le défaut d'enregistrement pour le calibre du compteur en question se produit le plus probablement à des débits de moins de 3,78 l/h· Le problème de la baisse de précision aux faibles débits est illustré par un graphique linéaire (fig. 1) basé sur les résultats des essais de petits compteur à entraînement mécanique de la Compagnie des Eaux d'Indianapolis. En annexe, il y a également un tableau (tableau 1) montrant les résultats de 4 études différentes pour déterminer la part d'eau consommée par des compteurs domestiques à différents débits.

Les distributeurs d'eau se sont longtemps préoccupés de la question de savoir: "Combien de temps faut-il laisser un compteur en service?". Pour beaucoup de sociétés, la période entre les essais a été établie par des Commissions de services publics ou a été établie arbitrairement par la société. A l'heure actuelle peu de sociétés ont mené à bien une évaluation économique. Deux systèmes utilisés fréquemment pour faire une telle évaluation sont commentés. Une étude de la Compagnie des Eaux d'Indianapolis pour déterminer la durée de vie économique de ses 135 000 compteurs à entraînement mécanique est illustrée. Les résultats de cette étude illustrée graphiquement, démontrent que les compteurs d'Indianapolis ne devraient pas rester en service continu pour une période de plus de 9 ans.

Une autre étude de la Compagnie des Eaux d'Indianapolis démontre que l'enregistrement des compteurs à entraînement magnétique (pour usage domestique) est plus précis que celui des anciens compteurs à entraînement mécanique. Les compteurs à entraînement magnétique ont une durée de vie de 15 ans contre seulement 9 ans pour les compteurs à entraînement mécanique. Si ceci se confirme, il pourrait être plus économique de remplacer les anciens compteurs à entraînement mécanique par les nouveaux compteurs à entraînement magnétique que de réparer les anciens compteurs à entraînement mécanique.

En conclusion, il est caractéristique que les compteurs domestiques perdent leur précision aux petits débits à cause de l'usure des pièces intérieures. Le moment auquel l'usure se manifeste varie considérablement d'une société à l'autre à cause des propriétés différentes des eaux fournies.

Des études démontrent que jusqu'à 23% de toute l'eau passant par les compteurs domestiques, passe à des débits auxquels le compteur est le moins efficient. Si le compteur reste donc trop longtemps en service, la société de distribution d'eau fera des pertes importantes. Chaque service d'eau devrait établir un intervalle régulier d'essai des compteurs afin de minimaliser cette perte de revenus.

Chaque service d'eau doit faire sa propre étude et établir un intervalle d'essai mettant en équilibre économique la perte de revenus et les frais pour enlever, essayer et réparer (ou remplacer) ses compteurs.

International Standing Committee on Education and Training of Waterworks Personnel

Subject 1

Guidelines for the Development of Training Programmes in Developed and Developing Countries

by Mr. H. W. Barker Great Britain
Dr. H. Moser Germany
Mr. J. B. Abrahamse Netherlands
Mr. C. Dyard France
Dr. J. Van Damme W.H.O. International Reference Centre, The Hague

1 Introduction

1.1 The decision to launch this piece of work was taken by the Standing Committee on Education and Training at its meeting during the Brighton Congress of 1974. The Committee was influenced in this by certain of the points made by Mr. H. W. Barker in presenting his paper on "Training of Waterworks Personnel in Developing Countries".

1.2 While mindful of the mandatory principle of training that states that a major pre-requisite to soundly based schemes of industrial training is thorough and objective manpower planning, the Committee accepted Mr. Barker's point that the lack of a manpower plan as a basis for training need not mean that no training can or should take place. By all means, therefore, let there be pilot and ad-hoc training programmes and courses but, to stand the test of time, there must ultimately be an overall system of training for all personnel.

1.3 This paper therefore aims to present a simple concept whereby water managers in member countries may, by reference to a master Task Schedule, seek and obtain, if available, from the Working Group of the Standing Committee training advice, guidance or an existing training programme relevant to a particular task or group of tasks identified by the water manager as a priority training requirement. This concept is thus one of attempting to set up some system for achieving faster progress in implementing pilot and ad-hoc training programmes and courses.

1.4 It is still the hope of the Standing Committee that funds and resources may eventually be assembled to mount, preferably in a developing country, a comprehensive training project of the kind envisaged by the Directors of Institutions collaborating with the W.H.O. International Reference Centre for Community Water Supply during their Conference at Bilthoven in the Netherlands in April 1973. However, more than three years have elapsed since the drafting of this high priority project but there is little evidence to suggest that a "Bilthoven Training Project" is any nearer the action stage today than it was when accorded such high priority in 1973. This is not meant to imply or infer that interest has waned or priorities have changed in the minds of those national and international organisations striving to encourage the spread of waterworks training in those regions of the world where the improvement of water and sanitation services are severely handicapped by the lack of trained personnel or indeed of an adequate number of personnel. This reference to delay, difficulty and shortage of the necessary resources required to mount a major training offensive is made simply to re-inforce the idea of doing the best that is possible, even though this may mean ad-hoc solutions, as an interim and intermediate measure.

2 A Task Orientated or Modular Approach

2.1 The major component of the programme of work of the Standing Committee on Education and Training is therefore to be the drawing together from among its considerable membership of all existing training analyses, manuals, programmes and training modules with intent to form a bank of training data available to individual countries or water authorities seeking advice or information about training for a given task or job. There will, of course, be many problems to be faced and overcome in attempting to provide a service of this kind. Not least of these will be the problems of language, differing organisational structures and operational practices. It is acknowledged that it is one thing to have a fully documented training programme or training course, even supported by the appropriate training media, handouts and audio-visual aids, but it is quite another thing, perhaps without trained staff, to make the package work. Notwithstanding these difficulties the attempt should be made. Limited though progress may be, some progress is better than none at all. Equally it is timely to remind ourselves that such constraints and differences form the common denominator of the objects and purpose of this International Association.

3 Evolving a Task Schedule

3.1 The centre piece of this paper is a Task Schedule which it is hoped will form the basis for an ad-hoc task orientated or modular approach to training for a particular job or function. The Schedule represents the first stage in establishing the most rudimentary form of job analysis. It is of paramount importance that the water expert who reads this paper should regard and accept the Task Schedule in this light. The major observation on the Schedule is that it is incomplete; indeed it can be argued that the rate of change and technological innovation in the field of public water supplies is such that a Task Schedule of this kind will never be complete and up-to-date at a point in time.

3.2 Of the numerous "notes for guidance" that could be suggested for using or understanding the task matrix, the following are considered to be the more relevant:

(a) Seven levels of work have been identified and are shown on the Schedule. It is accepted that in some countries there may be more categories while in others there will be fewer. The Water Technician is perhaps a comparatively recent phenomenon of the developed countries but at least no water authority functions without managers, supervisors and operators. In some countries there may be a single class of Water Technician and in such a case that country is requested to regard the work of the Higher Technician and the Technician as forming one group of tasks.

(b) Against each level of work is shown a number of functions. Once again it is hoped that these approximate to the way in which the work of water supply is organised and structured. It is inevitable that this categorisation will not fit all water organisations. Again the aim has been to simplify and to try to ensure that ultimately most if not all water supply tasks have been identified and grouped together on some reasonably uniform and functional basis. However, this is not to suggest that the groupings as shown are in any way inflexible or compartmentalised.

(c) The technical content of the supervisor's job can be drawn from either the technician level of work or from the craft/operator levels of work. The Schedule is meant to illustrate the supervisor's position as probably being the focal point in the hierarchy of work and that his activities are as much managerial as they are technical.

(d) No attempt has been made to provide task breakdowns for craftsmen categories. The number and types of craft skills employed within member countries are likely to vary more widely than for other levels of work. Additionally schemes of training and education for apprentice craftsmen are usually provided on a national rather than an industry basis. However, the Working Group believes that whereas craft training, by its nature, does not fit an ad-hoc system, many units of training illustrated in the Schedule may be of considerable relevance to those craftsmen employed within a water authority, e.g. craft plumbers may need training in the application of water byelaws to the inspector of water installations as an endorsement subject to their general craft training.

(e) Definitions of the levels of work given by the Task Schedule have been taken from the Glossary of Training Terms produced by the Standing Committee on Education and Training and for ease of reference have been included in the Task Schedule.

3.3 The Working Group will welcome any general comment or observation on the analysis of work as illustrated on the centre page of this paper and in particular, will be pleased to receive suggestions for the inclusion or addition of clearly definable jobs or tasks which may appear to have been omitted. All such helpful comment should be addressed in the first instance to Mr. H. W. Barker, the Secretary of the Standing Committee on Education and Training.

4 Using the Task Schedule

4.1 There are a variety of ways in which it is hoped the Task Schedule may be used either by a water authority within a developed country or within a developing country:

(a) Assuming the Schedule to be sufficiently comprehensive in terms of the breadth and depth of the first stage job analysis that it sets out ultimately to achieve, then it could be useful as a means of measuring the extent to which current training provisions in an Authority match the jobs to be done.

(b) The progressively increasing knowledge and skill which is reflected in the way in which the tasks are ranked within the respective levels of work may usefully provide a basis for staff development programmes and grading structures.

(c) A complete group of tasks is defined within one particular function and at, say, the Technician level of work could form the nucleus of a scheme of training leading to the certification of Technicians or Higher Technicians.

(d) Setting out what will hopefully become a complete spectrum of the work done by waterworks personnel at a given level could pave the way to more flexible use of the labour available thus reducing demarcation or specialisation and creating increased versatility and job satisfaction.

(e) On the basis that a considerable variety of training programmes are currently available for those tasks described at the operator level of work it is hoped that by drawing attention to these through the dissemination of the Task Schedule, it will be brought home to some water managers both in developed and developing countries that waterworks operators, particularly within the distribution function, require training, even ad-hoc training, as much as anybody else.

4.2 Whether or not this on-going project of the IWSA Training and Education Committee has sufficient impact to influence water managers in these ways it is hoped that, as a minimum, it will prompt some water managers to ask for training information or a validated training programme in, for example:

Timbering of excavations	—Operators
Training responsibilities	—Supervisors
Disinfection of mains and services	—Technicians
Selection and use of Pumping Machinery	—Professional Engineer
Manpower Planning	—Managers

Such requests would be met at least with the advice of the Committee through its Working Group on where the required piece of training can be obtained if only through guided and supervised planned practical experience in a member country which accepts trainees for short periods

INTERNATIONAL WATER

STANDING COMMITTEE ON

GUIDELINES FOR THE DEVELOPMENT OF TRAINING

MANAGER

PLANNING

UNITS:
1. Establishing Objectives
2. Identifying Constraints
3. Economics of Water Use
4. Influence of Technological Change
5. Interdependence of all departments
6. Water Law
7. Manpower Planning

ORG.

UNITS:
1. Establishing the Op
2. Defining and Delega
3. Organising Manpow
4. Selecting, Motivatin
5. Providing for Safety
6. Establishing Good I

PROFESSIONAL

WATER PLANNING

UNITS:
1. Water Quality Standards
2. Principles of Effluent Treatment
3. Hydrological Concepts
4. Principles of Water Supply and Treatment
5. Use of Data
6. Licensing
7. Statistical Techniques
8. Market Research and Demand Analysis
9. Flood Forecasting
10. Economic Concepts and Charging Policies

WATER PRODUCTION & DELIVERY

UNITS:
1. Economics of Water Use
2. Design Concepts of Water Supply & Treatment Works
3. Project Appraisal
4. Selection and Use of Pumping Machinery
5. New Works Construction:—
   Administrative Aspects
   Legal Aspects
6. Water Treatment Processes
7. Small Reservoir Location and Design
8. Organisation of Work

WATER D

UNITS:
1. Hydraulic Analysis
2. Design of Mains (C
3. Organisation of Was
4. Automatic Control a
5. Pumps and Pumpin
6. Water Treatment an
7. Water Finance and
8. Water Law

HIGHER TECHNICIAN

WATER PLANNING

UNITS:
1. Surface Water Hydrology
2. Groundwater Hydrology
3. Principles of Water Quality
4. Principles of Chemical Analysis
5. Principles of Biological Analysis
6. Data Assembly
7. Licences
8. Operation of River Regulating Works

WATER PRODUCTION & DELIVERY

UNITS:
1. Construction of Wells and Boreholes
2. Pumping Tests
3. Selection and Use of Construction Plant
4. Selection and Use of Pumping Machinery
5. New Works Construction:—
   Administrative Aspects
   Legal Aspects
6. Water Treatment Processes
7. Small Reservoir Location and Design
8. Organisation of Work

WATER D

UNITS:
1. Organisation of Was
2. Corrosion of Mains
3. Control of Animals i
4. Hydraulic Analysis c
5. Design of Mains (Si
6. Organisation of Dep
7. Organisation of Veh
8. Pumping Machinery

TECHNICIAN

WATER PLANNING

UNITS:
1. Measurement of Abstractions
2. Flow Measurement
3. Surface Water Measurement
4. Groundwater Measurement
5. In-Situ Quality Sampling
6. Data Sheet Compilation
7. Site Maintenance of Recorders

WATER PRODUCTION & DELIVERY

UNITS:
1. Drawing Office Equipment and Techniques
2. Detailing for Water Supply and Treatment Works
3. Principles of New Works Construction
4. Surveying
5. Elementary Hydraulics

WATER DI

UNITS:
1. Improvement of Pip
2. Elementary Hydrauli
3. Provision of New Su
4. Surveying Practice
5. Scraping and Lining
6. Disinfection of Main

SUPERVISOR

THE MAN IN

ALL FU

UNITS:
1. General Principles
2. Organisation of Wor
3. Communication
4. Job Safety (By Func
5. Industrial Relations
6. Training Responsibi
7. Financial Controls

CRAFTSMAN

PLUMBING

ENGINEERING

OPERATOR

CONSTRUCTION : MAINTENANCE : REPAIR

UNITS:
1. Laying and Jointing Pipes
2. Timbering of Excavations
3. Service Connections and Mains Ferruling
4. Fixing Water Meters
5. Exchanging Water Meters
6. Excavations, Backfilling, Reinstatement
7. Underpressure Mains Branching

OPERATION : CONTROL : INSPECTION

UNITS:
1. Control of Mains Supplies
2. Monitoring Mains Supplies
3. Inspecting / Detecting / Preventing / Measuring —Waste
4. Reading Water Meters
5. Inspecting Water Installations (Byelaws)

SUPPLY ASSOCIATION

EDUCATION AND TRAINING

PROGRAMMES – A TASK ORIENTATED APPROACH

NISING

rational Structure
ing Powers and Duties
r and Materials
, Training and Paying Staff
Hygiene and Welfare
dustrial Relations

CONTROLLING

UNITS: 1. Use of Physical Controls
2. Use of Financial Controls
3. Selection and Use of Management Techniques

MANAGER

TRIBUTION

mplex)
e Control
d Telemetry

Examination (Principles)
dministration (Principles)

WATER EXAMINATION

UNITS: 1. Trace Metal Analysis by:—
(a) Atomic Absorption Spectrophometry
(b) Electromechanical Techniques
(c) Polarographic Techniques
2. Auto-Analysis Systems
3. Continuous Monitoring Systems
4. Laboratory Management
5. Legal Aspects
6. Mathematical Models
7. Pilot Plant Studies
8. Evaluation of Plant Performance

ADMINISTRATION & FINANCE

UNITS: 1. Investment Appraisal
2. Project Appraisal
3. Economics of Water Use
4. Data Processing
5. Stores Accounting and Stock Control
6. Water Law

PROFESSIONAL

TRIBUTION

e Control

Water Mains
Water Supply Networks
ıple)
ts, Workshops, Stores
cles and Mobile Plant

WATER EXAMINATION

UNITS: 1. Water Quality
2. Industrial Pollutants: Toxic Substances
3. Water Biology and Botany
4. Animals in Distribution Systems
5. Chemical Dosing Floc Action
6. Filtration Systems
7. Biological Treatment Processes
8. Sludge Conditioning and Disposal
9. Interpretation of Data

ADMINISTRATION & FINANCE

UNITS: 1. Charges for Water Services
2. Income Accounting
3. Expenditure Accounting
4. Preparation of the Payroll
5. Capital Financing and Expenditure
6. Financial Controls
7. Stores, Transport, Plant
8. Statistical Techniques and Records
9. Insurance
10. Water Law

HIGHER TECHNICIAN

TRIBUTION

work Systems
s
oplies

of Mains
and Services

WATER EXAMINATION

UNITS: 1. Laboratory Safety and Maintenance
2. Sampling
3. Gravimetric Analysis
4. Volumetric Analysis
5. Colorimetric Analysis
6. Bacteriological Examination of Potable Waters
7. Trace Metal Analysis
8. Routine Measurement Physical/Chemical Parameters
9. Analysis for Potable Supply Works

ADMINISTRATION & FINANCE

UNITS: 1. General Office Operations
2. Principles of Administration
3. Principles of Finance
4. Data Processing
5. Stores Accounting and Stock Control
6. Water Law

TECHNICIAN

TIONS

(By Function)

on)

ies

THE MIDDLE

SUPERVISOR

ELECTRICAL

VEHICLE

CRAFTSMAN

SUPPLY & TREATMENT

UNITS: 1. Cleaning/Attending/Operating
(a) Softening Plant
(b) Hardening Plant
(c) Chlorinators
(d) Rapid Gravity Filters
(e) Pressure Filters
(f) Slow Sand Filters
(g) Sedimentation Tanks
2. Chemical Dosing
3. Flow Gauging

WORKSHOP & SUPPORT OPERATIONS

UNITS: 1. Driving Motor Vehicles
2. Driving and/or Operating Power and Light Plant
3. Repairing Water Meters
4. Testing Water Meters
5. Testing Water Fittings
6. Grounds Maintenance
7. Materials Handling
8. Repair/Maintenance of Hand Pumps

OPERATOR

of training secondment or attachment. At best the request could be met by the provision of a training programme incorporating a detailed training syllabus, course literature where appropriate, a training manual or manuals, some audio-visual media and a statement defining the training objectives which should be met in order to confirm that the trainee has acquired the set of skills and knowledge required to perform adequately the selected task or job.

4.3 Where, after reference to the Task Schedule, a Water Manager wishes to obtain information, advice or a training programme in respect of any specific task he should in the first instance make contact with the Working Group through the Secretary of the Training Committee. If, upon receiving a training package the Water Manager decides that he cannot from within his own resources adequately implement the programme or courses then he may either buy-in local training expertise to assist him in mounting the programme or he may extract from the material with which he has been supplied that which he can adapt or use within the limitations of his own capabilities and resources. At least he will have something to build upon.

5 The need for Training Staff

5.1 This paper advances the proposition that in the absence of formal and permanent arrangements for waterworks training it is better to attempt an ad-hoc training system rather than make no attempt at all. To some extent this pre-supposes the non-availability of waterworks training specialists; where an authority is fortunate to have acquired or has regular access to waterworks training specialists, either full-time or part-time, there would be less dependence if indeed there were any need at all to resort to ad-hoc methods of training. For this reason, i.e. that normally it requires a training specialist to train the trainer, emphasis is placed yet again upon the value of concentrating one's investment in overseas training on buying the expertise and experience of a country which has an established waterworks training organisation. To give some weight to this recommendation there should perhaps be W.H.O. Fellowships in Waterworks Training.

6 Staff Development and Education

6.1 An undoubted weakness of the ad-hoc training approach is that such a system or lack of system will delay the design and implementation of schemes for staff development and education which are commensurate with systematic and comprehensive schemes of industrial training. It is, however, a well-known fact of life that the time scale for expanding or extending national, regional or even local educational or further educational provisions to match the requirements of industry can be inordinately long. Equally the processes of consultation and negotiation which are the inevitable and unavoidable precursors of designing or improving staff development schemes are often complex and protracted. It is therefore inadvisable to delay training until agreement has been reached on a staff development scheme. A more useful and realistic philosophy perhaps could be to "grasp the training nettle" and, using the inherent right of the local manager to make his own decisions, initiate some training, albeit of an ad-hoc nature, and use this as the catalyst in determining and achieving improvements to the development of educational services and career aspirations of waterworks personnel.

7 Conclusion

7.1 The concept presented by this paper may seem to some to be too much akin to a do-it-yourself kit. But surely the water man is no stranger to improvisation and innovation where the tailor-made article is not available. Self-help brings its own kind of reward—so why not self-help in training?

The Working Group of the Standing Committee on Education and Training of Waterworks Personnel look forward to the opportunity to demonstrate and amplify this concept during the Session on Education and Training to be held during the Amsterdam Congress.

Appendix

Definitions — Extracts from the Glossary of terms

Manager

The activities of managers are essentially concerned with setting objectives and deciding priorities, devising and implementing the means to achieve the objectives successfully, and the means to assess the results achieved.

Professional Person

A professional person is competent by virtue of his fundamental education and training to apply scientific method to the analysis and solution of business problems. He is able to assume personal responsibility for the development and application of science and knowledge. His work is predominantly intellectual and varied, and not of a routine mental or physical character. It requires the exercise of original thought and judgement and the ability to supervise the technical and administrative work of others.

Technician

A person who carries out functions of an intermediate grade between the technologist on the one hand and the craftsman on the other. The education and specialist skills of a technician enable him to exercise technical judgement, i.e. an understanding, by reference to general principles, of the reasons for, and the purposes of his work, rather than reliance solely on established practices or accumulated skills.

Supervisor

The supervisor is at the first or second level of the total management structure and is in charge, either directly or indirectly of a particular area of operations. He is primarily concerned with planning and controlling the work of others, whether in the works, on site, or in the office.

Craftsman

A skilled worker in a particular operation, trade or craft who is able to apply a wide range of skills and a high degree of knowledge to non-repetitive work with a minimum of direction or supervision.

Operator

A manual worker who possesses a degree of skill and knowledge of a narrower range than that of a craftsman, and who is capable of a lesser degree of adaptation.

La formation des agents d'exploitation à la Compagnie Générale des Eaux

par C. Dyard

Compagnie Générale des Eaux, Paris, France

1 Introduction

En France, la distribution de l'eau potable est assurée par des régies municipales ou confiée à des Sociétés privées. La Compagnie Générale des Eaux est l'une de celles-ci.

Chaque Société privée organise pour son propre compte, la formation de son personnel. La Loi du 16–7–1971 a d'ailleurs fait de la formation professionnelle, une obligation pour toutes les entreprises.

Bien que de nombreux établissements publics et organismes privés offrent une grande diversité de cours et de stages, la spécificité de la profession oblige les distributeurs d'eau à organiser, au sein de l'entreprise, la plupart des actions de formation destinées à leur personnel, surtout au personnel technique.

C'est ce qui a amené la Compagnie Générale des Eaux à créer son propre Centre de formation professionnelle à Romorantin (Loir et Cher).

L'objet de la présente étude a été limité aux agents d'exploitation, c'est à dire à ceux qui assurent la production et la distribution de l'eau potable. Ils ont maintenant également la responsabilité du rejet des eaux usées urbaines (assainissement).

On peut distinguer quatre niveaux d'agents d'exploitation:

1. les ouvriers (plombiers),
2. les responsables d'un "service" ou exploitation (fontainiers),
3. les techniciens,
4. les ingénieurs.

Ces agents sont généralement polyvalents, à l'exception des techniciens qui sont spécialisés (électricité, traitement de l'eau potable, épuration des eaux usées).

2 Détection des besoins

La détection des besoins est une opération préalable indispensable à laquelle il convient de consacrer tout le temps nécessaire. Elle permet notamment de faire la part de la formation dans la solution des problèmes, qui relève parfois d'une réorganisation des services, liée ou non à la formation.

La méthode la plus efficace, parce que très pragmatique, consiste à multiplier les entretiens avec les catégories de personnel visées par les stages et avec la hiérarchie. C'est la méthode la plus généralement utilisée à la Compagnie Générale des Eaux. Elle suppose de nombreux déplacements lorsque l'entreprise est implantée sur l'ensemble du territoire national, mais elle permet de faire la synthèse de l'attente des futurs stagiaires et de leurs chefs directs.

Dans certains cas, notamment pour des stages très spécialisés, une autre méthode a été employée: un programme a été rédigé puis soumis à la critique des futurs utilisateurs (personnel et hiérarchie). Le nouveau texte construit à partir des remarques et suggestions recueillies diffère souvent notablement du programme initial mais l'objectif est atteint.

3 Elaboration des programmes

Le programme est alors rédigé avec la collaboration des moniteurs qui l'animeront. De toutes façons, le premier stage est expérimental et ce sont ses participants qui apporteront (voir plus loin au chapitre "évaluation") les critiques qui permettront aux moniteurs d'apporter les correctifs nécessaires.

Au départ, il faut être conscient du problème que posera nécessairement l'impossibilité d'obtenir, pour chaque session, un groupe parfaitement homogène. Il convient d'élaborer un programme valable pour tous, quel que soit le niveau de base de chacun. Ce niveau est généralement assez faible pour les deux premiers niveaux (plombier et fontainier) bien que l'on constate une amélioration due à l'allongement de la durée de la scolarité obligatoire.

D'autre part, les tâches confiées aux agents ne sont pas toujours rigoureusement identiques. Nous avons délibérément opté, à l'égard des agents du premier niveau, pour le stage polyvalent, qui correspond à la réalité de la majorité de ces agents, notamment de ceux qui ont une responsabilité dans une exploitation isolée. Les agents plus spécialisés, appartenant à des services importants, y trouvent cependant leur compte car le stage leur apporte une ouverture vers d'autres tâches, une connaissance de l'ensemble des fonctions dans leur exploitation, des possiblités de remplacement ou de mutation, donc de meilleurs possibilités de déroulement de carrière.

4 Choix des moniteurs

C'est le problème essentiel, qu'il faut résoudre avant tous les autres.

Le moniteur doit répondre à deux critères indissociables:

(a) c'est un homme compétent et d'une grande expérience,

(b) il a une disposition naturelle au contact humain.

Ces conditions étant remplies, le moniteur devra recevoir une formation aux méthodes pédagogiques d'entreprise, mais cette formation serait inutile si la deuxième condition n'était pas satisfaite, quelle que soit la compétence technique de l'intéressé.

Son niveau doit être immédiatement supérieur à celui des stagiaires. Si possible, il "sortira du rang" et connaîtra ainsi parfaitement les problèmes des catégories de personnel dont il assurera la formation.

Il arrive qu'un moniteur nous fasse part de son inquiétude: il craint de perdre le contact avec la vie réelle en exploitation. Ce scrupule n'est qu'en partie fondé, car les stagiaires lui apportent leurs problèmes professionnels. Cependant, on peut lui proposer de consacrer périodiquement un certain temps à un retour en exploitation, par exemple pour effectuer un remplacement. Ce risque n'existe pas pour les moniteurs à temps partiel, qui connaissent un autre problème souvent difficile à résoudre: partager leur temps entre leur vie professionnelle d'une part, la préparation et l'animation des stages d'autre part.

5 Les stages d'agents d'exploitation

Nous laisserons de côté la formation des techniciens et des ingénieurs, que la Compagnie Générale des Eaux organise dans les domaines des réseaux, des installations électro-mécaniques, du traitement de l'eau potable, de l'épuration des eaux usées et bientôt des relations humaines.

Rappelons que la formation des agents d'exploitation du premier niveau est polyvalente, même s'ils n'exercent que des fonctions limitées: travaux de pose et d'entretien par exemple.

En fait, cette formation a toujours existé. Elle se faisait "sur le tas" et l'on ne dira jamais assez combien cette formule comporte d'avantages, puisqu'elle se déroule dans les conditions réelles et sur les lieux mêmes du travail. Pourquoi l'avoir donc remplacé par une formation systématique? C'est parce qu'en réalité, l'agent n'est pas vraiment formé "sur le tas", il se forme lui-même au contact de ses collègues. Il risque d'acquérir des méthodes erronées, il ne voit que ce qui existe dans son exploitation et surtout il n'est qu'un simple exécutant qui ignore le "pourquoi" des consignes données. Son chef direct n'a pratiquement pas le temps de lui donner des explications, il n'en a d'ailleurs peut-être pas l'aptitude.

Il était donc nécessaire de résoudre le problème essentiel du temps: dégager l'agent de son travail quotidien pour qu'il se consacre à sa formation, avoir des moniteurs disponibles; il fallait ensuite donner aux stagiaires l'ensemble des connaissances et du savoir-faire nécessaires à leurs fonctions. Ces conditions ne peuvent être réunies que dans un stage organisé, sous réserve que les méthodes utilisées permettent de se rapprocher le plus possible de la formation "sur le tas".

La formule du stage présente en outre l'avantage de rassembler les agents en un groupe au sein duquel ils échangeront problèmes et expériences.

Nous donnons ci-dessous les grandes lignes du stage polyvalent de base pour agents d'exploitation du premier niveau: les participants sont des aides-plombiers (manoeuvres, terrassiers) qui apprennent le métier de plombier. Il comporte deux parties, d'une durée de trois semaines chacune et est suivi de stages d'application dont nous parlerons dans un chapitre spécial.

La première partie est centrée sur l'activité travaux. Dans la seconde partie, certains chapitres font l'objet d'un enseignement complet. D'autres ont pour but de donner une initiation à des techniques qui seront approfondies au cours de stages de deuxième niveau (colonne de droite —E2—). A noter que la seconde partie du stage peut convenir également aux agents déjà anciens qui ont été formés "sur le tas" et pour lesquels elle apporte un perfectionnement.

Programme succinct du stage de premier niveau (E1)

1ère partie (3 semaines)

- —information générale sur l'entreprise et organisation d'une exploitation,
- —les ressources en eau: captages, sources, puits divers,
- —notions succinctes d'hydraulique (utilisation d'une maquette),
- —réseaux maillés, réseaux radiaux, choix des matériaux,
- —technologie des tuyaux, joints, raccords, appareils
- —exercices pratiques en atelier et sur réseau d'entraînement sur types usuels de tuyaux, joints et raccords, tubes de branchement, robinetterie et fontainerie; raccordement avec arrêt d'eau, pose de compteur, réparation des conduites.

Ces travaux comprennent: la préparation du chantier, les approvisionnements, la signalisation, la sécurité, la préparation du fond de fouille, le bardage des tuyaux, la pose, les essais en pression, le rinçage, la stérilisation, l'attachement du travail exécuté et le repérage.

- —les plans: échelles et symboles.

2ème partie (3 semaines)

	Stages du 2e niveau
—les "traités" avec les collectivités locales,	—
—traitement des eaux potables: généralités, visites de stations,	E2—TS
—assainissement: généralités sur le réseau,	—
—épuration des eaux usées: généralités, visites de stations,	E2—AS
—notions d'hydraulique: débit, pression, vitesse, pertes de charges,	—
—réservoirs: fonctions, équipement,	—
—rendement de réseau—détection des fuites: généralités,	E2—DF
—stations de pompage: technologie des visites de stations,	—
—stations de surpression,	—
—installations électro-mécaniques des stations, notions simples d'électricité,	E2—ES
—compteurs d'eau: types usuels, choix, relevés,	—
—service des abonnés: règlement, rapports avec les abonnés.	—

N.B.—Les stages spécialisés du deuxième niveau durent de 5 à 7 jours; la liste ci-dessus est imcomplète, d'autres stages étant en préparation (hydraulique, compteurs, réseau d'assainissement, relations humaines, etc. . . .).

6 Sélection des stagiaires

Pour profiter au maximum d'un stage, l'agent doit être pleinement motivé. La première condition est, bien entendu, qu'il soit suffisamment informé et consentant sans réserve.

Il ne suffit cependant pas qu'il soit volontaire. L'expérience nous l'a fait découvrir rapidement. On pouvait en effet logiquement penser que le meilleur moment pour former un agent était la période qui précédait immédiatement l'utilisation des connaissances acquises. C'était une erreur comme nous le prouve l'exemple suivant: nous avons accueilli, dans un stage d'épuration des eaux usées, des agents qui étaient sur le point d'avoir la responsabilité d'une telle station. La session ne leur a profité qu'en partie. Par contre, d'autres agents avaient déjà cette responsabilité depuis quelques semaines; arrivant en stage avec leurs nombreux problèmes, leur motivation professionnelle était totale.

Il est donc important, avant l'admission à un stage, que le candidat ait réellement besoin des connaissances que le programme lui apportera, mais surtout qu'il vive déjà suffisamment les problèmes de l'exploitation pour que sa participation soit vraiment motivée.

Enfin, on cherche à regrouper des agents venant de régions aussi diverses que possible. L'expérience a en effet montré qu'ainsi, des méthodes de travail différentes sont comparées et, d'un point de vue humain, il est enrichissant pour les stagiaires de découvrir et d'apprécier la personnalité de collègues venant d'autres horizons.

7 Méthodes pédagogiques

L'objectif du stage n'est pas de faire emmagasiner des connaissances, ce qui ne ferait du stagiaire qu'un simple exécutant, mais bien plus de lui donner les aptitudes et les réflexes intellectuels qui lui permettront ensuite de résoudre ses problèmes de tous les jours, même les plus simples.

Il serait d'ailleurs impossible d'étudier tous les cas qui peuvent se présenter en exploitation. Il n'est pas question de donner des "recettes" mais des bases suffisantes au stagiaire qui devra être capable de faire face aux situations inhabituelles et proposer ou prendre des initiatives.

Après le stage, on pourra lui confier un travail complet. Si, par exemple, il doit réaliser un branchement, il saura le faire depuis l'ouverture de la fouille jusqu'aux essais, à l'attachement et au repérage. Il en retirera une satisfaction personnelle et son chef direct n'aura plus à exécuter une partie du travail de son subalterne.

Ceci suppose que le moniteur évite l'ensignement didactique chaque fois que cela est possible. Il fait observer des phénomènes, ce qui permet aux stagiaires de découvrir des notions théoriques difficilement assimilables par l'abstraction; par exemple l'hydraulique grâce à une maquette puis, en grandeur réelle, sur réseau d'entraînement.

Le travail en groupe facilite les échanges entre stagiaires, que ce soit avec les moniteurs ou en dehors de leur présence.

Ce groupe est limité à huit (dix au maximum), le moniteur étant toujours "avec" les stagiaires et non devant eux comme un professeur. La disposition des locaux doit le permettre.

Les aides pédagogiques les plus utilisées sont les matériaux que l'agent utilise en exploitation: tuyaux, robinets, compteurs, etc.... Des coupes de joints, de vannes, de corps de pompes ... permettent des explications plus détaillées.

Les maquettes de simulation sont largement employées pour les notions théoriques: hydraulique, traitement, électricité.

Les moyens pédagogiques visuels et audio-visuels classiques interviennent selon les besoins: transparents pour rétroprojecteur, diapositives permettent l'intervention des stagiaires; ceux-ci font des exposés au tableau (à craie ou de papier). Des films sont utilisés pour présenter ce qui ne peut être vu sur place: condensé d'un chantier de pose par exemple.

Les séances en salle sont entrecoupées de travaux pratiques en atelier: soudures, collages, prises en charte, démontages de compteurs, etc....

Sur le terrain d'entraînement, les stagiaires réalisent un réseau complet ainsi que les branchements.

Enfin, le Centre de formation ayant été amenagé dans une exploitation importante, les stagiaires peuvent visiter des installations de toutes sortes. Ils profitent également de toute occasion pour travailler "in situ": pose d'appareils, réparations ...

8 Les stages d'application

Nous avons souligné plus haut les avantages incomparables de la formation "sur le tas".

Si les stages ont été conçus de manière qu'ils s'approchent le plus possible des conditions réelles de l'exploitation, il subsiste malgré tout quelques inconvénients:

—l'eneignement est reçu dans des lieux particuliers,

—les stagiaires travaillent en groupe,

—des moniteurs leur consacrent tout leur temps,

—même les travaux pratiques sont réalisés dans des conditions quelque peu "idéales".

Le retour en exploitation serait trop brutal et le stage perdrait de son efficacité si un palier n'était ménagé entre la période privilégiée du stage et la réalité des tâches quotidiennes.

Ce palier est constitué par le stage d'application, dont les caractéristiques principales sont les suivantes:

—le stagiaire y est envoyé individuellement,

—il est intégré dans l'équipe d'une exploitation réelle.

Il ne faut cependant pas que le stagiaire soit livré à lui-même. D'autre part, s'agissant d'une application du stage en groupe, il importe si non de suivre un programme rigide, du moins de viser un objectif, qui est de varier les tâches de manière à faciliter cette mise en application; dans le même but, on facilite au stagiaire la visite des installations de l'exploitation.

On voit immédiatement qu'il est nécessaire d'opérer une sélection judicieuse des exploitations qui reçoivent des agents en stage d'application. Le premier critère de choix est que l'exploitation ait des activités variées (eau et assainissement) et des installations intéressantes (on évitera l'adduction uniquement gravitaire, qui tend d'ailleurs à disparaître). Le deuxième critère rejoint celui du choix des moniteurs, bien que dans une moindre mesure: le responsable de l'exploitation, sans être un moniteur, doit cependant avoir l'esprit formateur. Il consacre une partie de son temps au stagiaire, s'intéresse à ses activités, répond à ses questions, fait la critique de son travail.

Les exploitations qui remplissent ces conditions ne sont pas faciles à trouver, nous en avons fait l'expérience lorsque nous avons créé les stages de premier niveau: parmi 8 exploitations sélectionnées, 5 ont été conservées par la suite; deux autres ne répondaient qu'en partie aux besoins; quant à la dernière, nous nous sommes aperçus qu'on y utilisait le stagiaire comme un personnel venu en renfort. Maintenant, le processus est bien au point et donne d'autant plus satisfaction que les exploitations choisies ont pris l'habitude de recevoir des stagiaires. Il est d'ailleurs certain que cette nouvelle vocation a des effets bénéfiques sur le personnel permanent de ces exploitations.

Notons d'autre part que, parmi la dizaine d'exploitations sélectionnées, un choix est fait au moment de l'envoi d'un stagiaire en fonction de l'origine géographique de celui-ci, qui est envoyé dans une exploitation extérieure à sa Région mais distante au maximum de 200 Km.

Quant à la durée du stage d'application, elle est variable suivant le programme.

Après un stage de premier niveau (polyvalent), l'agent qui quitte le Centre de formation participe à deux stages d'application, chacun d'une durée de 4 semaines. Le premier se déroule, comme indiqué plus haut, dans une Région différente de la sienne. Le second a lieu dans sa propre Région afin de la familiariser avec les règles locales, notamment dans le domaine technico-administratif (service des abonnés).

Après des stages spécialisés, l'application est faite dans une installation appropriée: station de pompage, de traitement, d'épuration, etc. ... Elle dure quelques semaines et consiste à mettre le stagiaire "en doublure" avec l'agent responsable en titre de cette installation.

9 Suivi de la formation — Evaluation

Le dernier jour de la session, les stagiaires participent à une réunion animée par un cadre du Service Formation. Les moniteurs sont absents de cette réunion, dont ils ont informé les stagiaires au début de la session.

Il est demandé aux participants de faire la critique du stage et de faire toutes les suggestions concernant le programme, les méthodes et les moyens pédagogiques. Cette évaluation est très profitable, surtout s'il s'agit d'une des premières sessions d'un nouveau stage ou si ce stage est animé par un nouveau moniteur.

Parmi ces critiques, l'une d'elles est particulièrement positive: certains stagiaires nous disent qu'ils ont le sentiment de "rester sur leur faim". Ils montrent par là qu'ils ont découvert que leur formation ne s'arrêtait pas le dernier jour de la session mais qu'ils devaient poursuivre par eux-mêmes cette formation. Là aussi, on voit l'importance du stage d'application, période de transition avant le retour en exploitation.

Un autre objectif de la réunion d'évaluation est de pouvoir mettre en garde les stagiaires contre le découragement qui risque d'accompagner ce retour. En effet, la réalité quotidienne est différente de l'organisation idéale vue en stage et le responsable local n'est pas aussi disponible qu'un moniteur. Le stagiaire devra faire preuve de persévérance pour mettre à profit ce qu'il a reçu.

Cependant, si cette réunion est très utile, elle ne permet pas de juger l'efficacité du stage, qui ne pourra être connue qu'après qu'un temps assez long se sera écoulé. L'expérience semble montrer que cette durée est d'environ six mois. Ce n'est qu'à ce moment que l'on peut juger l'effet du stage sur l'amélioration de la qualification et du comportement de l'agent. Il serait souhaitable, alors, de pouvoir s'entretenir avec lui d'une part, avec son supérieur direct d'autre part, afin d'apprendre dans quelle mesure le stage a été un facteur de changement. Mais une telle possibilité est limitée.

Nous avons utilisé un procédé qui, s'il est moins bon que l'entretien, permet cependant de toucher tous les anciens stagiaires. Il consiste à adresser à chacun d'eux un questionnaire ouvert qui leur demande de porter un jugement sur le stage et d'indiquer les difficultés qu'ils ont pû rencontrer pour le mettre en pratique. Un autre questionnaire est envoyé à leur supérieur direct qui donne son appréciation sur le même stage à travers le comportement de l'agent. La synthèse des deux catégories de réponses est plus satisfaisante qu'il n'était prévu. Elle va permettre d'une part d'améliorer les programmes en comblant certaines lacunes, d'autre part de conseiller les exploitations afin qu'elles assurent le suivi de la formation dans de meilleurs conditions.

10 Conclusion

On aura sans doute remarqué, tout au long de cet exposé, l'absence de méthode a priori dans l'élaboration d'une action de formation. C'est que celle-ci doit être essentiellement pragmatique. Le formateur n'est jamais sûr d'être dans la bonne voie. Il expérimente, il corrige ses erreurs en se faisant aider par tous et tout d'abord par les principaux intéressés: les stagiaires. Il doit être habité par une "saine inquiétude". Mais surtout, il doit avoir la foi. . . .

Summary

1 Introduction

Water distribution has its specific training needs which can scarcely be met by general training consultants.

The training of distribution personnel includes 4 levels: operator — supervisor — craftsman — engineer, generally multi-skilled, except craftsmen who are specialized.

2 Needs identification

The most efficient method consists in interviewing the particular category of personnel and the management.

For specialized sessions, a programme is written then criticized by the users to come.

3 Programme elaboration

It is impossible to have an exactly homogeneous group of trainees, yet the programme must be valid for any basic level. On the other hand, the tasks are not always similar. We have opted for a multi-skilled programme, which suits the majority of operators.

4 Instructor selection

An instructor must be fully competent and experienced and have a natural bent for human contact. His own level in the hierarchy is immediately above the trainees. If possible, he is selected from the workforce and thus knows the problems of the personnel to be trained. He will be given an appropriate education as a trainer.

5 Sessions for Distribution operators

In the past, this category of personnel received "on the job" training. We replaced it by systematic training because no operator is really trained on the job but trains himself, sometimes in wrong methods, and has only a sketchy view of water distribution.

In a session, the trainee leaves his daily tasks to meet available and skilled instructors. He has an opportunity to open his knowledge and aptitudes to tasks he would not do in his own work. He takes part in a group in which he may exchange experiences.

The first level session is divided in two parts:

1st part (3 weeks)

- —general information on water distribution
- —notions of water resources, hydraulics, networks
- —technology of materials
- —practical work of laying, jointing, testing main and service pipes, including safety.

2nd part (3 weeks)

- —notions of water treatment, waste treatment, leak-detection, electrical installations
- —water-tanks, network-output, hydraulics, pumps, pumping-stations, water-meters, administrative rules.
  (Each item of the 2nd part is developed in a 1 week-session for supervisors.)

6 Trainee selection

They must be well informed and agree with being sent to a session. They must also be sufficiently motivated, professionally speaking. We only accept trainees who have at least two or three months experience, so that they come with their own problems.

In one group, we try to have people from different provinces, for it enriches them to discover their colleagues' personalities.

7 Instructional methods

The trainee must be given aptitudes and mental reflexes to enable him to solve his own daily problems.

With this object, the instructors use lectures as little as possible but let the trainees look at phenomena and discover the theory for themselves.

The group is limited to 8 (max. 10).

Instructional means are the materials of distribution, sections of them, also models, over-head transparencies, slides and films.

Considerable practical work is carried out in the workshops and on the training network. It is supplemented by visits to installations.

8 Post-course experience

This is complementary to the training session and meets the advantages of training "on the job". The trainee is sent individually to a selected service. The latter must have various activities and interesting installations.

The supervisor in charge of it must have a taste for training and give a part of his time to the trainee.

A first level full session is followed by two periods of post-course experience, of 4 weeks each. Other sessions include 2 weeks post-course experience.

9 Training validation

At the end of a session, the trainees criticize the programme and the methods, but it is too soon to appreciate their efficiency. This cannot be known before about 6 months, by asking the former trainee and his own chief. Through the answers, we can improve the programme and give advice so that follow-up to the training may be to the satisfaction of the hierarchy.