Planning and Preventive Maintenance of Water Distribution Systems

Training Course
held in May 1978

INSTITUTE OF
PUBLIC HEALTH ENGINEERING
AND RESEARCH
UNIVERSITY OF ENGINEERING AND TECHNOLOGY LAHORE PAKISTAN

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FOREWORD

The Training Course on Planning and Preventive Maintenance of Water Distribution Systems organized by the Institute from 13 to 18 May 1978 should be considered a timely exercise in view of the large scale expansion of urban water utilities and planned provision of rural water supplies in the country. This is one in a series of training and refresher courses regularly offered by Institute in water and wastewater engineering fields. Others offered earlier in association with WHO personnel included computer application in water distribution network analysis disinfection practices and laboratory techniques for water pollution indicators. The organizations which nominated their engineers to participate in the present Course included public health engineering departments of the provinces of Sind, N. W.F.P. and the Punjab, development authorities of major cities like Karachi, Hyderabad, Islamabad and Lahore and one of the engineering consulting firms of international repute. The nominees were engineers who have had long standing association with public health engineering works in general and public water utilities in particular.

The purpose of the Training Course was to afford an opportunity to the practising engineer to update his technical and managerial knowledge and upgrading of planning, operation and maintenance practices. A complete absence or lack of such practices is already resulting in excessive leakage in the distribution mains, careless waste of water. leakage or non-registration of water due to poor quality or inoperative house meters, unaccounted for or unauthorised house connections, delivery of free water to public agencies and inadequate or unregulated plumbing practices. The end result of the excessive water losses together with minimum water rates that encourage waste; inadequately sized distribution system ; inadequate storage capacities is that very few of our urban water supply systems have water in taps at adequate pressure during 24 hours of the day every day of the year.

The quality of service in the life of the water utility is directly related to planning, operation and maintenance practices. For economic as well as public health reasons this problem is important. Its solution is related to finance, management and design. Keeping in view the needs of the profession this Course was designed and it consisted of lectures, a field visit and laboratory work relating to quality control procedures. The present document is largely a compilation of lectures prepared by the individual speakers using published references on the topics dealt by them. The discussions during the course of lectures provided and excellent opportunity for exchange of views and information among the participants and speakers both of whom have been drawn from various parts of the country. Most of the speakers are persons of eminence in the water supply engineering field. In particular the name of Mr. A. G. Banks Leader of the Management Team from United Kingdom Overseas Development Ministry should be mentioned who has had the opportunity of completing large projects at international level. His cooperation and continued help in planning the Course is gratefully acknowledged. Professor Dr. M Islam Sheikh Vice-Chancellor of this University on the occasion of course inauguration

emphasized the need for use of local technology and materials and Sayyid Hamid pointed to the need of standardization of items used in public health engineering practice. It should be pointed out that the WHO/UNDP assistance programmes aimed at strengthening the facilities of this institute have gone a long way in developing the necessary competence for offering such training courses as the present one. Mr. Waris Ali, Assistant Professor rendered substantial assistance in organising the Course. Thanks are also due to Syed Mohsin Raza Ali Assistant Professor and Mr. Shahid Farooq Chaudhry Lecturer who made excellent arrangements in connection with miscellaneous services like accommodation and transport for participants and other related matters. The financial assistance rendered by Azam Instruments Limited Karachi in connection with programme arrangements is gratefully acknowledged. The excellent typing work done by Mr. N. A. Najmi in preparation of the material for the printer need to be appreciated.

Lahore
10 December 1978

DR. MOHAMMAD NAWAZ TARIQ
Director and Professor of
Public Health Engineering.

COURSE PROGRAMME

Saturday 13 May 1978	0800	Registration
	0930	Inauguration
	1100	Refreshments
	1130	Guidelines for Determining Water Requirements
	1300	Water Distribution System Capacity
Sunday 14 May 1978	0800	Hydraulics and Pumps for Water Distribution Systems
	0930	Water Distribution System Planning
	1100	Refreshments
	1130	Economic Considerations in Water Distribution System Planning
	1300	Leak Detection Surveys Their Role
Monday 15 May 1978	0800	Leak Detection Survey Field Visit
	1100	Refreshments
	1130	Water Supply Bye-Laws
	1300	Drinking Water Quality
Tuesday 16 May 1978	0800	Records in Water Distribution Systems
	0930	Laboratory Procedures
	1100	Refreshments
	1130	Laboratory Procedures
Wednesday 17 May 1978	0800	Metering the Water Distribution System
	0930	Operation and Maintenance of Water Distribution Systems
	1100	Refreshments
	1130	Water Rates
	1300	Manpower Utilization
Thursday 18 May 1978	0930	Management Needs
	1100	Refreshments
	1130	Course Review
	1300	Certificates Presentation

CONTENTS

GUIDELINES FOR DETERMINING WATER REQUIREMENTS

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Quantity of Water Required

One of the early steps in the design of a waterworks system is the estimation of water required. The estimate must of necessity be an approximation since it applies at some future date. However, a thorough understanding of all factors involved will greatly aid the estimator in arriving at a reliable answer.

The water consumption of a community usually is reported in gallons per capita per day. This quantity does not apply to any single day. It is the yearly average and is based upon the total population of community. A collection of data on per capita water requirements in various countries is presented below for general reference.

Country	Year	City or Community	Water Consumption
Australia	1970	Melbourne	105 gpcd
	1970	Sydney	125 gpcd
	Future	...	170 gpcs
Colombia	1960	Bogota	234 lpcd
	1960	Cartagena	132 lpcd
Ethiopia	1970	Urban Area	20—100 lpcd.
Japan	1965	Osaka	111 gpcd
	1965	Tokyo	87 gpcd
	1965	Yokohama	81 gpcd
	Future	...	120 gpcd
Kuwait	...	...	40 gpcd

Country	Year	City or Community	Water Consumption
Pakistan	1976	Lahore :	
		High density (151 persons/acre)	35 gpcd
		High density design	67 gpcd
		Medium density (76—150 persons/acre).	53 gpcd
		Medium density design	75 gpcd
		Low density (75 persons/acre)	125 gpcd
		Low density design	94 gpcd
		Multan, Hyderabad and Gujranwala Design ...	90 gpcd
		Public standpost	12 gpcd
Pakistan	1976	Department of Public Health Engineering:	
		Villages : Population : 5,000—10,000	
		No drainage	12 gpcd
		With surface drains	18 gpcd
		Towns Population : 10,000—25,000	24 gpcd
		Cities : Population : 25,000—100,000	35 gpcd
		Population : Over 100,000 (domestic and public uses only).	36—48 gpcd
Syria	1970	Rural	30—75 lpcd
		Rural design	75 lpcd
Thailand	1975	Bangkok	115 gpcd
	2000	Bangkok	140 gpcd

Notice that some values are for design purposes. For Lahore the design values for low density areas is less than the present use based on the assumption of reduced wastage.

The normal values of daily per capita water consumption for different uses in USA are given below. For a given project a detailed investigation is often necessary to predict the future per capita water consumption. In cases where uses other than that for domestic purposes are high such as towns built around a big industrial complex it is probably more realistic to estimate the future demand separately rather than to lump all uses together in terms of per capita water consumption.

Uses	Water Consumption gpcd	
	Range	Average
Domestic	15—70	50
Commercial & Industrial	10—100	65
Public	5—20	10
Uncounted for	10—40	25

Uses of Water

The ultimate users of the water produced by a public water supply system are commonly classified as residential or domestic, commercial, industrial and municipal or public.

Domestic Use

It includes all water used in and around residences. The amount of domestic consumption varies considerably according to climatic conditions, habits, customs of the population, economic level and the existence of the sewers.

The gpcd (US) for domestic and institutional uses in some cities in the USA is given below :

Description	Water Use
Homes	35—90
Offices	27—45
Hospitals	125—225
Grade Schools	5—10
High Schools	15—20

The breakdown of domestic water consumption for various uses in terms of percentages of total use is given below. The fact that a large percentage is for washing and toilet flushing purposes has prompted environmental engineers to advocate dual supply systems for conserving high quality water sources. It was suggested that the relatively low toilet use in Melbourne could be due to the fact that the size of the flush tank used in the city is 3 gal as compared with the 5 to 6 gal size tanks normally used in the USA.

Category	U.S.A.		U.K.	Australia Melbourne
	Town 1	Town 2		
Toilet flushing	41	45	41	31
Washing and bathing	37	30	37	38
Kitchen use	6	...	11	9
Household cleaning	3	20	...	...
Clothes washing	4	...	11	16
Drinking and cooking	5	5	...	6
Sundries	4	...	...	...
Total	100	100	100	100

Lawn watering in many cases represents a large item of water use. Lawn watering is normally the first to be restricted in case of water shortage. In cities where open space is limited watering requirements will be much smaller. However with the expectation of better living conditions even in developing countries water use for lawns in urban areas should not be overlooked too lightly.

In Pakistan, a figure of 50 imperial gpcd has been suggested for estimating urban domestic consumption. In this estimate 80 per cent of the population is assumed to have been provided with water carriage system. This is based on a detailed analysis presented below. The minimum recommended design value is 35 imperial gpcd for high density areas as shown in previous table.

Use	Requirement
Drinking	0.45
Bathing	18
Cooking	0.83
Water Closets	8
Ablution	4.16
Kitchen & Household Cleaning	4.67
Washing Clothes	3.75
Watering Lawn	2.74
Cleaning Bathroom	1.8
Guest	5.6
Total	50 igpcd

Commercial use

It includes water used in commercial districts by persons who are not residents of the districts. Commercial use and light manufacturing use cannot be stated conviently in terms of floor areas of the buildings in the district. Commercial consumption is variable reaching a maximum of 900 cubic metres per day per hectare in centres with a high commercial density. This daily consumption may lead to a maximum of 200 to 300 cubic metre per hour per hectare.

Industrial use

Industrial use bears no relation to the population of an industrial district. It is difficult to define industrial consumption in terms of gallons per person per day. Nevertheless, it is advisable to express it in terms called "equivalent population". The large variety of industries existing all over the world and the new ones which are emerging as civilization advances make it very difficult to produce a long term forecast of the amount of water that should be reserved for industrial consumption. Due to this difficulty large industries are compelled sometimes to develop their own industrial supply and use the urban water system only for the limited consumption of the employees and workers. The lack of industrial connection to the metropolitan water system has an adverse repercussion on the financing of the urban water programme and its development.

When designing a metropolitan water system we should bear in mind that industry, in general, is a good customer of water supply systems and in some cases produces considerable revenues that make it possible to improve and extend the existing services.

Even when it is difficult to establish the "equivalent population" for industries, it is convenient to predict some percentage figures compared with total onsumption. In U.S.A. the industries use from 15 to 65% of the total urban consumption. Industrial expansion and development and the advent of new industries, processes and uses can have an important effect and produce real problems in a distribution system. It is impossible to guide any design for the future without adequate operating records of the past and the present.

Unaccounted For Water

The water processed in the production works does not all reach the ultimate users. Some of it is used up in processing as for for example, in flushing the basin, washing filters, and producing steam for pumping. Even in a fully metered system, that is, one in which the water going to each individual customer is constantly measured, there are uses and losses that cannot be fully accounted for such as water discharged from hydrants for flushing streets and sewers, public water-fountains, leakage. A portion is lost in leakage from joints in pipe lines, from broken pipes, faulty valve and hydrant packing, and the like. Even customer meters cannot accurately measure all water passing through them, particularly at very low flow rates. This in the aggregate, can amount to an appreciable under registration of system uses.

The difference between the net plant output and the sum of metered or estimated flow through customer services is generally referred to as "unaccounted for water." The losses and uses in such unaccounted for water must be recognized in estimates of actual consumption. The volume of wastage of water could reach 5 to 20 per cent of the total amount of water delivered to the city.

Fire Demand

The fire flow may be defined as the rate of flow needed for fire-fighting purposes to confine a major fire to the building within a block or other group complex.

Fire demand is quite often not considered in designing a water supply system in Pakistan. This may conceivably change in the future. The major items of costs in providing extra system capacities for fire protection are those for larger laterals and for storage for fire flow. Fire hydrants are normally provided even in systems not adequately designed for fire flow.

In the U.S.A. the following equation is used for estimating fire flow for central congested areas with population less than 200,000.

$$Q\ (\text{gpm}) = 1020\sqrt{P} \quad (10.01\sqrt{P})$$

where P=Population in thousand.

For areas with population more than 200,000 a flow of 12,000 gpm for the first fire and an additional flow of 2,000 to 8,000 gpm for a second fire are provided. For residential low risk districts four fire streams of 175 gpm each are considered adequate.

The extra storage capacity for fire flow is for a fire of 5 hours duration for small communities (population less than 2,500) and 10 hours duration for large cities.

Revised Fire Demand in the U.S.A.

The municipal fire flow requirements used to be based on the population of the area as indicated by the method adopted by the National Board of Fire Underwriters in U.S.A. The formula was developed by Metcalf, Kuichling and Hawley using data collected from April 1906 to March 1911 in more than 90 cities. The function of the municipal fire protection survey and grading work was taken over by Insurance Services Office (I.S.O) in October 1971.

With the changing living pattern in urban areas especially in developed countries it is now felt that the populatior growth may not reflect the growth of the downtown district as it used to be and that the development of outlying areas is such that high level fire protection may be needed in these areas even though the population is small. A new method based mainly on floor areas is therefore developed for estimating the fire fighting needs. The method is contained in an I.S.O. publication of 1 June 1972 entitled "Guide for Determination of Required Flow".

I.S.O. Method—General Criteria

The main equation is as follows :

$$F = 18\,C\sqrt{A}$$

where F is the required fire flow in gpm. The values of C for different types of construction are :

1.5 for wood frame construction,

1.0 for ordinary construction (in U.S.A.)

0.8 for noncombustible construction.

0.6 for fire resistive construction.

A is the floor area in sq ft including all storeys except basement. For fire-resistive buildings, consider the six largest successive floors if the vertical openings are not protected. If the vertical openings are protected, consider only the three largest successive floors.

The fire-demand for the above should not however exceed 8,000 gpm for wood frame or ordinary construction and 6,000 gpm for noncombustible or fire-resistive construction as well as one-storey buildings of all types of construction. The fire flow shall not be less than 500 gpm in any case.

A resume of all the percentage variations in city water consumption is :

Domestic use	... 30 to 70 per cent.
Commercial and Industrial use	... 6 to 65 per cent.
Public use	... 5 to 10 per cent.
Wastage	... 5 to 20 per cent.

Variation in Demand

Water consumption varies from hour to hour, day to day, and season to season. The range of variation depends on a number of factors notably the size of the community. Knowledge of the relation that the monthly, daily and hourly peak loads on a water supply system bear to the average load is of vital importance. A large reservoir used for providing water through periods of draught should have its capacity predicted upon the monthly average variation. Mains carrying water to the consumers need to be large enough to meet the maximum demands. Therefore, their sizes should be based upon the hourly variation. Pumps supplying direct pressure likewise must meet the maximum hourly demand, but if elevated storage is provided their size could be reduced to the requirements of the daily variation. In general, a small town usually has a much wider range of variation than large cities. For average design conditions in U.S.A. the following are normally used :

Ratio	Range	Average
Max. day : Ave day	(1.2 to 2) : 1	1.5 : 1
Peak hr : Ave hr.	(2 to 3) : 1	2.5 : 1

Based on limited measured data on water consumption in Peshawar, Pakistan, in 1972, the ratio between maximum and average day flow was estimated to be 1,26. The average day flow was 2.05 mgd.

The following have been suggested for Lahore, Pakistan :

Ratio	Average
Max. day : Ave. day	1.5 : 1
Peak hr. : Max. hr.	1.5 : 1

The Public Health Engineering Department design standards recommend the following for towns and villages in the Province of the Punjab, Pakistan.

Ratio	Average
Max. day : Ave day	1.25 : 1
Peak hr. : Ave hr	2.5 : 1

Population Estimates

Most of the design standards indicate that engineering services are performed in three steps : an engineering report ; preparation of construction plans, specifications and contract documents and construction inspection, administration and acceptance. The standards amplify what is expected and the first item is the development of the predicted population. Unfortunately no exact methods or formulations have been developed to predict future changes in population with a high degree of accuracy. With the development of more and more reliable statistical reporting programmes there have been an appreciable decline in the use of purely mathematical methods.

There are many cases on record where engineers have made large errors in population estimates. However the engineer is still required to come up with a population prediction over the design period of the proposed facility and the ultimate population at some additional point more distant in the future.

There are a number of methods for projecting population estimates into the future. These methods were discussed in detail in the lecture.

System Capacity

Because a water distribution system must provide adequate fire flow and satisfactory supply of water to consumers at the same time, the question is what system capacity should be designed for ? The usual practice is to design all the raw water transmission and treatment facilities for the maximum day flow. The capacity of the distribution system is sized for either the peak hour flow or the sum of the maximum day flow plus fire flow whichever is greater.

The figures generated in the above discussion are only general estimates based on past experience. They should be used with caution in forecasting future requirements for many variables influence their applicability to any one system. Some of these variables are local climatic conditions, the character of community served, the extent of air conditioning and lawn sprinkling use, the relative amount of commercial and industrial development, and the percentage of customers metered. For these reasons the past records of water use in a particular city or community are more valuable, in forecasting future requirements, than comparisons with cities or reference to general averages. Each water utility should set up and maintain records of how much water goes to the various classes of consumers so that in estimation of future demands the trend of development of the community and its effects upon water use can be better determined.

WATER DISTRIBUTION SYSTEM CAPACITY

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The adequacy of a distribution system is determined by the pressures that exist at various points under the conditions of operations. The factors which cause pressure loss include size of pipe, rate of flow of water and friction of the pipe material.

Since the permanent municipal distribution system is to be used over a long period of years, it becomes necessary for the engineer to anticipate the future and design the system for the conditions that will exist near the end of the time. Number of years in the future for which excess capacity is provided is called the design period which should be decided by keeping the following factors in mind (a) life of the facility (b) ease or difficulty in extension (c) lead time (d) rate of interest (e) rate of population growth.

To illustrate the importance of some of these factors Manne's capacity expansion model is briefly presented herein. The model assumes that the demand increases linearly with time 't' into the indefinite future with the annual rate of increase 'D' with units of mgd/year no excess capacity at the beginning of the planning horizon and economic parameters remain constant over time.

If it is desired that capacity be never less than demand the first expansion will be needed when $t=0$. Assuming an excess capacity period (design period) of X years the expansion will have capacity of XD mgd. At time $t=x$, excess capacity of the first expansion will be exhausted. By then demand will have grown equal to capacity. The conditions at $t=x$ are identical to those at $t=0$. Therefore another expansion of scale XD will be required.

In the field of water engineering, the cost is a concave power function of the

$$C\ (y) = Ky^{a} \qquad \text{—(1)}$$

form when 'y' is the amount of expansion needed and 'K' and 'a' are constants. Here 'K' is the cost of a unit capacity and 'a' is called the economy of scale factor and indicates the percentage change in cost per percent change in scale or equivalently, the ratio of marginal to average cost.

If the above cost function is used then the cost of the expansion XD can be written as :

$$C\ (XD) = K\ (XD)^{a} \qquad \text{—(2)}$$

Repeating the cost pattern for each point of zero excess capacity, the following expression of total present value construction cost for the infinite planning horizon will be

$$C_T = K\ (XD)^a/1-\exp\ (-rx) \qquad -(3)$$

'*r*' is annual discount rate expressed as decimal. The optional design period will be obtained by setting the derivative of above total cost equal to zero which results in optimality condition as

$$a = rx/\exp(rx-1) \qquad -(4)$$

Equation (4) shows that the optimal design period depends on the economy of scale factor and the annual discount rate *r* under the assumption of the above model. The equation (4) has been solved for varying values of '*a*' and '*r*' to obtain the corresponding design periods. *The results are reported below*

Comparison of Design Periods

Discount Rate r	Economy of Scale Factor a	Design Period from Equation (4)
0.05	0.5	25
	0.6	20
	0.7	13
	0.8	8
	0.9	4
0.10	0.5	12
	0.6	9
	0.7	7
	0.8	4
	0.9	2
0.15	0.5	8
	0.6	6
	0.7	5
	0.8	3
	0.9	1
0.20	0.5	6
	0.6	5
	0.7	3
	0.8	2
	0.9	1

The table indicates that for higher economy of scale factor and discount rate shorter design periods should be adopted. However, the design period should always be less than the life of the facility and greater than the lead time.

The ideal system capacity of a distribution system at the time under consideration would be to provide peak consumption and fire flow simultaneously at all points of the system. Since such a system will be very expensive, normal practice is to examine the adequacy of the system for the following (a) Peak consumption (b) fire flow plus maximum or average flow (c) minimum flow together with the filling of the service reservoin

Since the flow and pressure at different points in the distribution system are interdependant, the pressures in the distribution system vary with fluctuations of water concumption. The maximum system pressure will occur when the consumption is at a minimum. The minimum pressures may occur during peak demand or a fire. To maintain a satisfactory service pressure throughout the system at all time is one of the most important criteria in distribution system design. The service pressure is necessary to maintain satisfactory supply to the consumers, to deliver adequate fire flow whenever needed and to prevent the contamination of the system due to back pressure. On the other hand excessive high pressures result in high leakage losses, water wastage, and rapid wearing of fixtures. The upper limit of the pressure is generally adopted as 90 psi. The minimum acceptable pressure is 30 psi. However the minimum pressure should not drop below 20 psi in any case.

Study of the Capacity of Existing Distribution Systems

The field studies in terms of pressure surveys and hydrant-flow tests are most commonly done to establish available pressures and flows and existing deficiencies. Those can then be made the basis of hydraulic calculations for extensions reinforcements and new layouts.

Pressure Surveys

Pressure surveys yield the elementary information about networks. If they are conducted both at night (minimum flow) and during the day (normal demand) they will indicate the hydraulic efficiency of the system in meeting common requirements. However they are not able to establish the probable behaviour of the system under stress.

Hydrant Flow Tests

Hydrant flow tests include the observation of the pressure at a centrally situated hydrant during the conduct of the test and measurement of the combined flows from a group of neighbouring hydrants. Velocity heads in the jets issuing from the hydrants are usually measured by hydrant Pitot tubes. Most commonly used practice is the following :

1. Read the presure when the first hydrant is closed.

2. Open the fire hydrant, read the pressure and calculate fire flow from hydrant Pitot tube reading.

The flow which the system can provide during a fire such that the pressure in the system does not fall below 20 psi is computed by using the following relationship :

$$Q_F = Q_H \left(\frac{P_1 - 20}{P_1 - P_2} \right)^{0.54} \qquad \text{—(5)}$$

where Q_F is the flow which the system is capable of providing during the fire, Q_H is the hydrant flow during the test. p_1 is the pressure when the hydrant is closed and p_2 the pressure when the hydrant is open.

Table below illustrates how the calculations are made.

Hydrant Flow Test

Test No.	Type of District	Hydrant Location	Q_H gpm	Pressure psi when Hydrant is Closed P_1	Open P_1	Q_F Equation (5)
1	Residential 3 ...	ST—4	360	73	54	630
2	Commercial ...	ST—2	1590	69	58	3560
3	Residential 1 ...	ST—3	850	135	125	3180
4	Institutional ...	M—2	1480	87	78	4380
5	Residential 2 ...	R—1	990	67	53	1900
6	Residential 1 ...	B—4	1330	134	54	1610
7	Residential 2 ...	ST—5	1060	66	51	1940

HYDRAULICS AND PUMPS FOR WATER DISTRIBUTION SYSTEMS

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The Fundamental Equations of Fluid Flow

Hydraulic analysis of water distribution systems requires a basic foundation in mathematics and engineering. Some of the fundamental principles of hydraulics should be understood by all those who are engaged in the planning,design or operation of water distribution systems. The three most important equations in fluid mechanics are the continuity equation, the momentum equation and the energy equation. For steady, incompressible, one-dimensional flow the continuity equation is simply obtained by equating the flow rate at any section to the flow rate at another section along the stream tube. By 'steady flow' is meant that there is no variation in velocity at any point with time. 'One-dimensional' flow implies that the flow is along a stream tube and there is no lateral flow across the boundaries of stream tubes. It also implies that the flow is irrotational.

The momentum equation stems from Newton's basic law of motion and states that the change in momentum flux between two sections equals the sum of the forces on the fluid causing the change. For steady, one-dimensional flow this is

$$\Delta F_x = P\,Q\,\Delta V_x$$

where F is the force, P is the fluid mass density Q is the volumetric flow rate, V is velocity and subscript x refers to the 'x' direction.

The basic energy equation is derived by equating the work done on any element of fluid by gravitational and pressure forces to the change in energy. Mechanical and heat energy transfer are excluded from the equation. In most systems there is energy loss due to friction and turbulence and a term is included in the equation to account for this. The resulting equation for steady flow of incompressible fluids is termed the Bernoulli equation and is conveniently written as :

$$(V_1^2/2g)+(P_1/w)+Z_1=(V_2^2/2g)+(P_2/w)+Z_2+h.$$

Here V is mean velocity at a section, $(V^2/2g)$ is velocity head (units of length), P is pressure, (P/w) is pressure head (units of length), w is unit weight of fluid, Z is elevation above an arbitrary datumn and h is head loss due to friction or turbulence between sections 1 and 2. The sum of the velocity head plus pressure head plus elevation is termed the total head.

Strictly the velocity head should be multiplied by a coefficient to account for the variation in velocity across the section of the conduit. The average value of the coefficient for turbulent flow is 1.06 and for laminar flow it is 2.0. Flow through a conduit is termed either uniform or non-uniform depending on whether or not there is a variation in the cross-sectional velocity distribution along the conduit.

For Bernoulli equation to apply the flow should be steady, i.e. there should be no change in velocity at any point with time. The flow is assumed to be one dimensional and irrotational. The fluid should be incompressible, although the equation may be applied to gases with reservations. For most practical cases the velocity head is small compared with the other heads and it may be neglected.

Flow Head Loss Relationships

Conventional Flow Formulae

The throughput or capacity of a pipe of fixed dimensions depends on the total head difference between the ends. This head is consumed by friction and other (minor) losses.

The first friction head loss/flow relationships were derived from field observations. These empirical relationships are still popular in water works practice although more rational formulae have been developed. The head loss/flow formulae established thus are termed conventional formulae and are usually in an exponential form of the type

$$V = K\ R^x\ S^y \text{ or}$$

$$S = K'\ Q^n/D^m$$

Here V is the mean velocity of flow, K and K' are coefficients, R is the hydraulic radius (cross sectional area of flow divided by the wetted perimeter, and for a circular pipe flowing full, equals one quarter of the diameter) and S is the hydraulic gradient (in *m* head loss per *m* length of pipe). Some of the formulae more commonly applied are listed below :

Formula	SI units	fps units
Hazen—Williams	$V = 0.949\ CR^{0.63}\ S^{0.54}$	$V = 1.318CR^{0.63}\ S^{0.54}$
Chezy	$V = 0.552C_z\ R^{0.5}\ S^{0.5}$	$V = C_z\ R^{0.5}\ S^{0.5}$
Manning	$V = \frac{1}{n}\ R^{2/3}\ S^{1/2}$	$V = \frac{1.49}{n} R^{2/3} S^{1/2}$
Darcy	$V = \left(\frac{8g}{f}\right)^{1/2} R^{1/2}\ S^{1/2}$	$V = \left(\frac{8g}{f}\right)^{1/2} R^{1/2}\ S^{1/2}$

Except for the Darcy formula the above equations are not universal and the form of the equation depends on the units. It should be borne in mind that the formulae were derived for normal waterworks practice and take no account of variations in gravity, temperature or type of liquid. They are for turbulent flow in pipes over 50 mm diameter. The friction coefficients vary with pipe diameter, type of finish and age of pipe.

The conventional formulae are comparatively simple to use as they do not involve fluid viscosity. Solution of the formulae for velocity, diameter or friction head gradient is simple with the aid of a slide rule, calculator, computer, nomorgaph or graphs plotted on logpaper. The equations are of particular use for analysing flows in pipe networks where the flow/head loss equations have to be iteratively solved many times.

The most popular flow formula in water works practice is the Hazen Williams formula. Friction coefficients for use in this equation are tabulated below. If the formula is to be used frequently, solution with the aid of a chart is the most efficient way. Many waterworks organizations use graphs of head loss gradient plotted against flow for various pipe diameters, and various C values. As the value of C decreases with age type of pipe and properties of water, field tests are desirable for an accurate assessment of C.

Hazen Williams Friction Coefficient C

Type of Pipe	Condition: New	25 Years old	50 Years old	Badly Corroded
PVC ...	150	140	140	130
Smooth concrete AC ...	150	130	120	100
Steel bitumen lined galvanized ...	150	130	100	60
Cast Iron ...	130	110	90	50

For diameters less than 1000 mm, subtract $0.1\left(1 - \frac{\text{Dmm}}{1000}\right)C$

Rational Flow Formulae

Although the conventional flow formulae are likely to remain in use for many years, more rational formulae are gradually gaining acceptance amongst engineers. The new formulae have a sound scientific basis backed by numerous measurements and they are universally applicable. Any consistent units of measurements may be used and liquids of various viscosities and temperatures conform to the proposed formulae.

The rational flow formulae for flow in pipes are similar to those for flow past bodies or over flat plates. The original research was on small bore pipes with artificial roughness. Lack of data on roughness for large pipes has been one deterrent to the use of the relationships in water works practice.

The velocity in a full pipe varies from zero on the boundary to a maximum in the centre. Shear forces on the walls oppose the flow and a boundary layer is established with annulus of fluid imparting a shear force into an inner neighbouring concentric annulus. The resistance to relative motion of the fluid is termed kinematic viscosity, and in turbulent flow it is imparted by turbulent mixing with transfer of particles of different momentum between one layer and the next.

A boundary layer is established at the entrace to a conduit and this layer gradually expands until it reaches the centre. Beyond this point the flow becomes uniform. The length of pipe required for fully established flow is given by $X/D=0.7\ Re^{1/4}$ for turbulent flow.

The Reynolds number $Re=VD/v$ is a dimensionless number incorporating the fluid viscosity 'v' which is absent in the conventional flow formulae. Flow in a pipe is laminar for low Re (less than 2 000) and become turbulent for higher Re (normally the case in practice). The basic head loss equation is derived by setting the boundary shear force over a length of pipe equal to the loss in pressure multiplied by the area $t\ (22/7)\ DL = wh_f\ (22/7)\ D^2/4$. Therefore

$$h_f=f\ (L/D)\ (V^2/2g)$$

Here $f=(4t/w)/V^2/2g)$ the Dracy friction factor 't' is the shear stress, 'D' is the pipe diameter and 'h_f' is the friction head loss over a length 'L'. 'f' is a function of Re and the relative roughness e/d. The various rational formulae for 'f' were plotted on a single graph by Moody. However, Moody diagram is not very amenable to direct solution. The Hydraulics Research Station at Willingford re-arranged the variables in the Cole brook—White equation to produce simple explicit flow head loss graph in terms of V, D and S.

One method of expressing head loss through fittings and changes in section is the equivalent length method, often used when the conventional friction loss formulae are used. Modern practice is to express losses through fittings in terms of the velocity head i.e. $h_1=KV^2/2g$ where K is the loss coefficient. Valve manufacturers may provide supplementary data and loss coefficient K which will vary with gate opening. The velocity V to use is normally the mean through the full bore of the pipe or fitting.

Network Analysis

The flows through a system of interlinked pipes or networks are controlled by the difference between the pressure heads at the input points and the residual pressure heads at the drawoff points. A steady state flow pattern will be established in a network such that the following two criteria are satisfied.

(1) The net flow towards any junction or node is zero, i.e. inflow must equal outflow, and

(2) The net head loss around any closed loop is zero, i.e. only one head can exist at any point at any time.

The line head losses are usually the only significant head losses and most methods of analysis are based on this assumption. Head loss relationships for pipes are usually assumed to be of the form $h = K\ lQ^n/D^m$. Here h is the head loss, l is the pipe length, M the flow and D the internal diameter of the pipe. The calculations are simplied if the friction factor K can be assumed the same for all pipes in the network.

It is often useful to know the equivalent pipe which would give the same head loss and flow as a number of interconnected pipes in series or parallel. The equivalent pipe may be used in place of the compound pipes to perform further flow calculations.

The equivalent diameter of a compound pipe composed of sections of different diameters and lengths in series may be calculated by equating the total head loss for any flow to the head loss through the equivalent pipe of length equal to the length of compound pipe. The equivalent diameter can also be derived using a flow head loss chart.

It often speeds network analyses to simplify pipe networks as much as possible using equivalent diameters for minor pipes in series or parallel. Of course the methods of network analyses described below could always be used to analyse flows through compound pipes and this is in fact the preferred method for more complex systems.

The Loop Method for Analysing Networks

The loop method and the node method of analysing pipe networks both involve successive approximations speeded by a mathematical technique developed by Hardy Cross.

The steps in balancing the flows in a network by the loop method are : (1) Draw the pipe network schematically to a clear scale. Indicate all inputs, drawoffs, fixed heads and booster pumps (if present) (2) If there is more than one constant head node, connect pairs of constant head nodes or reservoirs by dummy pipes represented by dashed lines. (3) Imagine the network as a pattern of closed loops in any order. To speed convergence of the solution some of the major pipes may be assumed to form large superimposed loops instead of assuming a series of loops side by side. Use only as many loops as are needed to ensure that each pipe is in at least one loop. (4) Starting with any loop ascribe arbitrary initial flows around the loop, consistent with inputs and drawoffs from nodes. The more accurate the initial flow assumption the speedier will be the solution. Once the flows in one loop are determined, the flows in other loops should follow automatically, since the flow in at least one leg will be known. (5) Calculate the head loss in each pipe using a formula such as $h = K\,lQ^n/D^m$ or use a flow/head loss chart (preferable if the analysis is to be done by hand) (6) Calculate the net head loss around any loop, i.e. proceeding around the loop, add head losses and subtract head gains until arriving at the starting point. If the net head loss around the loop is not zero, correct the flows around the loop by adding the following increment in flow equal to $(\Sigma h/\Sigma(hn/Q)$ in the same direction that head losses were calculated. This is the first order approximation to the differential of the head loss equation. (7) If there is a booster pump in any loop, subtract the generated head from (Σh) before making the flow correction using the above increment. (8) The flow around each loop in turn is thus corrected. The process is repeated until the head around each loop balances to a satisfactory amount.

The Node Method for Analysing Networks

With the node method, instead of assuming initial flows around loops, initial heads are assumed at each node. Heads at nodes are corrected by successive approximation in a similar manner to the way flows were corrected for the loop method. The steps in an analysis are (1) Draw the pipe network schematically to a clear scale. Indicate all inputs, draw-offs, fixed heads and booster pumps. (2) Ascribe initial arbitrary heads to each node (except if the head at that node is fixed). The more accurate the initial assignments, the speedier will be the convergence of the solution. (3) Calculate the flow in each pipe to any node with a variable head using the formula $Q=(hD^m/Kl)^{1/n}$ or using a flow/head loss chart. (4) Calculate the net inflow to the specific node and if this is not zero, correct the head by adding the increment $H=\Sigma Q/\Sigma(Q/nh)$. Flow Q and head loss are considered positive if towards the node. H is the head at the node. Inputs (Positive) and drawoffs (Negative) at the node should be included in ΣQ. (5) Correct the head at each variable head node in similar manner i.e. repeat steps 3 and 4 for each node. (6) Repeat the procedure (steps 3 to 5) until all flows balance to a sufficient degree of accuracy. If the head difference between the ends of a pipe is zero at any stage omit the pipe from the particular balancing operation.

Alternative Methods of Analyses

Both the loop method and the node method of balancing flows in networks can be done manually but a computer is preferred for large networks. If done manually, calculations should be set out well in tables or even on the pipe analysed manually by the node method. There are standard computer programmes available for network analyses, most of which use the loop method.

The main disadvantage of the node method is that more interations are required than for the loop method to achieve the same converagence, especially if the system is very unablanced to start with. It is normally necessary for all pipes to have the same order of head loss.

Optimization Techniques

The previous section described methods for calculating the flows in pipe networks with or without closed loops. For any particular pipe network layout and diameters, the flow pattern corresponding to fixed drawoffs or inputs at various nodes could be calculated. To design a new network to meet certain drawoffs, it would be necessary to compare a number of possibilities. A proposed layout would be analysed and if the corresponding flows were just sufficient to meet demands and pressures were satisfactory, the layout would be acceptable. If not, it would be necessary to try alternative diameters for some or all pipes and to re-analyse the network. The process of adjusting pipe sizes and analysing flows is repeated until a satisfactory solution is at hand. This trial and error process would then be repeated for another possible layout. Each of the final networks so derived would then have to be costed and that network with least cost selected.

A technique of determining the least cost network directly, without recourse to trial and error, would be desirable. No direct and positive technique is possible for general optimization of networks with closed loops. The problem is that the relationships between pipe diameters, flows, head losses and costs is not linear and most routine mathematical optimization techniques require linear relationships. There are a number of situations where mathematical optimization techniques can be used to optimize layouts.

Mathematical optimization techniques are also known as systems analyses techniques (which is an incorrect nomenclature as they are design techniques, not analyses techniques), or operations research techniques (again a name not really descriptive). The name mathematical optimization techniques will be retained here. Such techniques include simulation (of mathematical modelling) coupled with a selection technique such as steepest path ascent or random searching.

The direct optimization methods include dynamic programming, which is useful for optimizing a series of events or things, transportation programming which is useful for allocating sources to demands and linear programming, for optimizing any system which can be described by a set of linear equations or inequalities. Linear programming usually requires the use of a computer but there are standard optimization programmes available.

Balancing Storage

Demands such as those for domestic and industrial water fluctuate with the season, the day of the week and time of day. Peak-day demands are sometimes in excess of twice the mean annual demand whereas peak drawoff from reticulation systems may be six times the mean for a day. It would be uneconomic to provide pipeline capacity to meet the peak draw-off rates, and balancing reservoirs are normally constrcuted at the consumer end (at the head of the reticulation system) to meet these peaks. The storage capacity required varies inversely with the pipeline capacity.

The balancing storage requirement for any known draw-off pattern and pipeline capacity may be determined with a mass flow diagram : Plot cumulative draw-off over a period versus time, and below this curve plot a line with slope equal to the discharge capacity of the pipeline. Move this line up till it just touches the mass draw-off line at one point, which should be at the end of the peak draw-off period. Then the maximum ordinate between the two lines represents the balancing storage required.

An economic comparison is necessary to determine the optimum storage capacity for any particular system. By adding the cost of reservoirs and pipelines and capitalized running costs for different combinations and comparing them, the system with least total cost is selected. It is found that the most economic storage capacity varies from one day's supply based on the mean annual rate for short pipelines to two day's supply for long pipelines (over 60 km). Slightly more storage may be economic for small bore pipelines (less than 450 mm dia.). In addition a certain amount of emergency reserve storage should be provided ; up to 12 hours depending upon the availability of maintenance facilities.

Pumps

In the great majority of cases, water, after it has been collected and processed, must be pumped into the distribution system. In the design of the plant, equipment, and appurtenances to perform the pumping operations adequately, reliably and efficiently it is necessary to determine the best location for the pumping operations the best source of power, the conditions of operation and the capacities that will be needed and the best type of pump and prime mover for the conditions of operation. Pumps commonly used for water service may be classified according to (1) mode of action, whether piston, plunger, centrifugal, rotary, jet or direct pressure ; (2) motive power, whether engines drive, turbine drive, motor drive ; (3) number of cylinders (if applicable), whether simplex, duplex, or triplex ; (4) number of stages or pump elements in series whether single, double, or multistage ; and (5) mode of connection, whether direct, belt, crank or gear.

Piston and plunger pumps were once the most common types of pumps used with steam or internal-combustion engines. Rotary pumps, injector or jet pumps, and direct pressure pumps, such as the pulsometer, ram, and air-lift pumps, were largely used for special purposes. Few, if any, of these types of pumps are now being installed in water systems.

Because it so successfully fills the requirements of water works service, the centrifugal pump in its several variations has almost completely displaced all other types of pumping equipment. It is most commonly driven by an electric motor, but hydraulic and steam turbines and internal combustion engine drives are frequently used.

The centrifugal pump consists essentially of a rotating impeller which draws water into its centre and a stationary casing which guides the water to the discharge outlet. It may have a single or double suction arranged for end, bottom, or side suction piping connections. It may have an open, semiclosed, or closed impeller and be arranged for direct-connected horizontal or vertical drive, the latter being of either the dry-pit or submerged type. One or more single-stage pumps may be set in series with one driver, or the casing may be built to contain two or more impellers in order to pump against higher head. The casing of the pump may be in the form of a spiral or volute of constant cross section or be equipped with a diffuser.

In the true centrifugal pump, the pressure is developed principally by the action of centrifugal force. Two other similar types of pumps used for specific purposes are the mixed-flow pump, in which the head is developed partly by centrifugal force and partly by the lift of the vanes on the water, and the axial-flow pump, sometimes referred to as a propeller pump in which most of the head is developed by the propelling or lifting action of the vanes on the water.

The centrifugal pump is used for high-head pumping in single or multistage arrangement. The mixed-flow and axial-flow pumps are adapted for low head, usually for large-capacity requirements. An adaptation of the normal arrangement of a centrifugal pump and its driver, used in the smaller capacities (3 mgd or less) is the close-coupled pump in which the pump is located on an extension of the motor shaft without independent support.

The principal operating features of a centrifugal pump and its advantages and disadvantages are (1) low first cost (2) compactness (3) absence of valves and pistons ; (4) low maintenance cost and low rates of depreciation ; (5) smooth flow and uniform pressures ; (6) simplicity of design and ease of operation and repair ; (7) freedom from shock injury ; (8) high rotating speed with direct connection to turbines or motors possible ; (9) ability to handle dirty water ; (10) increased output with pressure drops, and vice versa ; (11) low starting torque; and (12) adaptability of automatic and semiautomatic control.

Generally, the main pumping station should be capable of furnishing the peak demand with the largest pumping unit out of service. If there is more than one pumping station, or if the demands can be met from more than one point (by storage, well supplies, or other means) system realiability is thereby increased and the number of additional units or other power sources needed for fstandby service may be some what reduced.

WATER DISTRIBUTION SYSTEM PLANNING

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Introduction

A water distribution system consists of the pipes, valves, hydrants and appurtenances used for distributing the water; the elevated tanks and reservoirs used for fire protection and for equalizing storage and pumps; and the consumer service pipes and meters. The system should be so designed that an adequate supply of water is available to the consumers and for fire protection at all times at minimum of cost. It should be so constructed and operated that the chances for contamination of the water after it has entered the system are reduced to a minimum. Since most distribution system have developed with the growth of the community served, the problem of designing a complete new system seldom arises except for small towns. The principles of planning involved in a complete system may be employed in the extensions and reinforcing of mains with modifications to suit each individual case.

The twofold function of distribution systems planning requires relatively uniform distribution of water throughout the system for ordinary use and concentration at any point in the system of high rates of flow for fire extinguishing.

Type of Water Distribution

Systems for water distribution include the gravity flow type, use of pressure pumps and selection of the method of distribution should be made in accordance with the following provisions : If there is suitable high ground near the service area, a gravity system should be used. If there is no suitable high ground available near the service area, a pumping system should be used. When high ground is available but not capable of furnishing water by gravity to the entire service area, a gravity system should be used for the low lying sections of the service area, and either distribution pumps should be installed at the site of the distribution reservoir or booster pumps, or secondary distribution pumps installed in the middle of distribution area to furnish water to those areas in which pressures using gravity flow would be insufficient.

Water Distribution System Planning

The planned capacity of any distribution system should be sufficient to furnish, for normal operation, the planned hourly maximum flow and, for fire fighting periods, should be sufficient to meet demand for water for fire fighting plus the planned daily maximum consumption.

Pressure and Flow Requirements for Normal Consumption

The pressure required in the mains for normal domestic consumption depends upon the height of the buildings, the maximum instantaneous rate of flow through the hose service pipes, and the friction losses in meters, house services plumbing, and fixture outlets.

The maximum instantaneous rates of flow through a house service pipe depends upon the character and number of plumbing fixtures in the building and the probability of their simultaneous use.

The studies of the rate of flow through house meters has shown that the maximum instantaneous rate of flow for one—, two—, and three—family houses having tank type water closets did not exceed 10 gpm. The peak flow to one—, two—, and three—family houses having water closets equipped with flushmeters is less than 30 gpm. The peak flow to be used in design of the house water piping and for selection of meters is about sixteen times the average daily consumption figured on a floor-area basis for small private houses and about thirty times the average daily consumption for apartment houses, with three to six apartments.

If the pressure loss in the house service pipe and water lines within the building is assumed at 20 psi, the total pressure loss from the main to a residence with tank toilets at peak flow of 10 gpm is about 20 psi. The pressure required at the street level for excellent flow to a three-storey building is therefore about 42 psi. Experience indicates that flow is adequate for residential areas if the pressure is not reduced below 35 psi.

Planned Maximum Hourly Flow

The standard for the planned maximum hourly flow generally followed is 1/24th of the daily maximum flow augmented as follows : (a) 30% for large cities and industrial cities. (b) 50% for middle size cities and (c) 100% for small cities and special districts.

Fire Fighting Demand

The fire fighting demand depends upon the charactderistics of the city, its facilities for fire fighting, density of pipe line and climatic conditions.

The fire demand reservoir capacity of water provided in addition to the basic capacity of the service reservoir was discussed in the first lecture on guide lines for determining water requirements.

Service Reservoir

The height of the reservoir for gravity flow systems should be determined so as to insure 1.5 kg/cm^2 water pressure throughout the distribution system even when water level of the reservoir is at designed low water level.

If ground levels vary unduly within the service area, reservoir should be constructed respectively for such a service area, divided into two services as low and high service, or into three services as high, middle and low service. Alternately, pressure-reducing valves or booster pumps may be installed as required to provide proper water pressure in a given local area.

Reservoir should not be constructed near cliff-shoulders or other sites where constructional failure might possibly occur.

The capacity of service reservoir can be determined on the basis of the following rules : (1) Effective capacity as a standard should be 1/3 to 1/2 of the planned maximum daily flow. The minimum effective capacity should be atleast 1/4th of the planned maximum daily flow. (2) The volume of reserve fire water, should be in addition to the effective capacity. (3) Effective capacity of the reservoir should be determined for each distribution system respectively.

Flow and Pressure Requirements

In districts in which the mains are designed for a normal pressure just sufficient to give adequate domestic flow to homes, it is evident that there will be an insufficient pressure for fire fighting without mobile pumpers. According to the American Water Works Association, it is desirable that a normal pressure of 60 to 75 psi be maintained on a distribution system for the following reasons : (a) It will supply ordinary consumption for buildings up to 10 storeys in height (b) Gives effective sprinkler service in buildings of 4 to 5 storeys. (c) Permits direct hydrant-service for a few hose streams, insuring quicker operation by the fire department. (d) Allows a larger margin of fluctuation in local pressures in meeting sudden drafts, and offsets losses due to partial clogging or excessive length of service pipes.

An additional advantage of such a high pressure is that it will permit the wider use of flushometers and shower mixing valves. The advantages of higher pressures, however, should in every case be weighed against the additional pumping cost.

Four methods used for supplying pressure for fire streams are as follows : (1) The maintenance of sufficient pressure on the mains at all times for direct hydrant service for hose streams. (2)The use of emergency fire pumps to boost the pressure in the distribution system during fires. (3) The use of mobile pumping engines which take suction from the hydrants. (4) The use of a separate high-pressure distribution system for fire protection only. The first method is not ordinarily economical for large communities but is usually the best method for places not provided with full-time fire departments and mobile pumpers. The second method is applicable to places requiring higher pressures for fire fighting than are desirable for normal consumption and in which there are no mobile pumpers. It is more economical but less reliable than the first method. The third method is preferable for all communities large enough to maintain modern and well trained fire departments. The fourth method can be used only in portions of some of the large cities. Separate high pressure systems are usually supplementary to the main distribution system and thus give added fire protection in high value districts. Pressures of 150 to 300 psi are used in highpressure systems.

Velocity of Flow

Main velocity of flow in pipe line should be higher than 1 ft/sec but should not exceed 10 ft/sec for concrete morter type pipe and 20 ft/sec for steel or cast iron pipe.

General Arrangement of Pipe System

The location of the small distributor pipes in a distribution system is controlled by the location of the consumers and by the location of property requiring fire protection.

The pipes are usually laid in the streets at some standardized position between curbs. In the case of very wide streets, however, it is sometimes cheaper to install a main behind the curb on each side of the street because of the saving in service pipes. The system should be gridironed with connecting pipes laid on the cross streets at intervals not exceeding about 600 ft whether or not there are consumers on the cross streets. Dead ends should be avoided in order to minimize troubles from corrosion and from organic growths. Moreover, a pipe fed from both ends has a capacity equivalent to two pipes.

A large system consists of supply mains, arteries, and secondary feeders spaced at intervals of about 3,000 ft in the grid system and preferably looped. The approximate location of the feeders is determined largely by the distribution of the consumers and high-value property.

For fire protection, a minimum size of main of 6 in. for residential areas, and 8 in. for high-value districts if cross-connecting mains are not more than 600 ft apart is desirable. On principal streets and for all long lines not cross-connected at frequent intervals, 12-in. and larger mains are required.

Gate valves should be so located that no single case of breakage in the pipe system, exclusive of arteries, shall require more than 500 ft of pipe to be shut from service in high-value districts, or more than 800 ft in other sections, or shall require the shutting down of an artery. The valves should be located at street intersections in standardized positions so that they can be readily found in case of pipe breakage. All small distributors branching from larger pipes should be equipped with valves, although the larger pipes need not have valves, at each such branch. At intersections of large pipes, a valve in each branch is desirable. Large supply mains should be gated about once a mile and should be provided with air valves at high points and blow-offs at low points. Arteries should be gated so that not more than 1/4 mile within the system will be affected by a break.

Hydrants should be located at street intersections where they are accessible from four directions. They should be so spaced that no hose line need exceed 500 to 600 ft. The spacing will vary from about 150 ft in high-value districts of large cities to about 600 feet in suburban residential districts.

Hydrants are required to have not less than two 2½-in. hose outlets, standard 4½-in. suction outlets where necessary, and to be connected to the main with pipe not smaller than 6 in. and gated. Hydrants should be able to deliver 600 gpm with a loss of not more than 5 psi between the street main and the outlet.

The design of the distribution pipes and the type of the pipe line should be in accordance with the following rules.

Types and Diameters of Pipe

Cast-iron, ductile cast-iron, steel, asbestos-cement, prestressed-concrete centrifugal reinforced-concrete and polyvinyl chloride pipes may be used for appropriate application of service and according to the operating pressure and external load, high pressure, ordinary pressure or low pressure pipes may be used.

The diamter of the pipeline to be used for the distribution system can be determined based on any hydraulic formula.

The diameter of pipeline is determined by the difference in head between lowest water level at up steam terminal and highest water level at down steam terminal of pipeline. In case of cast-iron ductile cast iron or steel pipes coefficient C=100 in Hazen Williams formula is ordinarily recommended beacuse of decrease in conveying capacity over long period of use. For other classes of pipes rules may be excepted and higher C value may be recommended.

The diameter of pipeline through which water will be pumped should be selected to give the lowest cost of construction and operation.

Selection of Sizes

The extreme availability of supply and demand conditions makes it impracticable to use hydraulic analysis for selecting the size of water mains. The following empirical rule of Thumb is suggested : (a) spacing of 12 in. to 10 in. mains 2500 to 3000 feet (b) spacing of 8 in. to 6 in. mains 5000 to 15000 feet (c) spacing of 3 in. to 4 in. mains 3000 ft.

The minimum size recommended for any water mains supplying fire hydrants is 6 in. The minimum size recommended for water mains not supplying fire hydrants 3 in.

Depth of Pipes

The depth to which pipes should be laid is controlled by the cover required for protection against structural failure due to street traffic loads and for protection against freezing. A minimum cover of 24 to 36 in. is usually allowed. For protection against wheel loads of heavy trucks, the amount of cover required increases with the size of pipe and is greater for steel than for cast-iron mains. For important mains, the amount of cover to be used should be determined after an investigation of the stresses produced by wheel loads. Less cover is required for pipes under concrete pavements than for pipes under more resilient pavements and dirt road.

General Procedure in Design

In the design of a distribution system the following general procedure is suggested :

Preliminary Layout

A preliminary layout of all the pipes is prepared on a suitable map of the community. A contour map showing all street and lot lines is preferable. The location of all existing buildings, with heights shown, is helpful. The layout should include the distributing reservoirs and elevated tanks, with their water surface elevations indicated if they are fixed arbitrarily or by the topography. The desired residual pressure for peak flows at critical points in the system stated in terms of the elevation of the pressure table at these points should be shown. A tentative division of the system into two or more presure zones may be made if required. Pipe sizes may next be assumed based on best engineering judgement and computation and economic analysis.

Skeleton System

If practicable, the system is skeletonized for the hydraulic computations by eliminating all the smaller pipes in which the flow is negligible for a particular assumed position of the fire load. The system must be examined for several positions of the fire load. Hence several skeleton frameworks may be required. These systems will be similar with regard to the larger mains but may differ in the inclusion of smaller pipes immediately surrounding the fire. The purpose of the simplified framework is to the hydraulic computations being practicable. In skeletonizing the system, it is desirable to examine the magnitude of the errors caused by the neglect of smaller pipes. When the system involves small pipes framework, it is desirable to use an equivalent pipe for this element which is of sufficient size to take account of the smaller pipes. In some cases, also the skeleton system may be simplified by substituting a single equivalent pipe for several elements.

For many systems, the small differences in the size of the pipes or the difficulty of evaluating the errors due to the neglect of the smaller pipes may make skeletonizing impractical. In such cases, approximate hydraulic analysis may be required since the accurate methods available are too laborious.

Computation of Loads

In order to simplify the computations, the draft from the system for ordinary use is assumed to be concentrated at relatively few take-off points on the network. The loading points are taken at pipe junctions where feasible in order not to increase the number of elements in the system. Fire loads are applied at junctions also. The elevation of the pressure table corresponding to the minimum desired residual pressure for each loading point should be determined. When the loading points are selected, it is necessary to divide the system into districts, one to each loading point, in order to compute the loads. The district boundaries are arbitrarily selected, but some of the bounds may be conveniently located on natural bounds between distritcs of different types or on natural fire breaks. The number of loading points used is also arbitrarily selected, but the accuracy of the hydraulic computations increase with the number of districts selected.

ECONOMIC CONSIDERATION IN WATER DISTRIBUTION SYSTEM PLANNING

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Basics of Economics

Economics is used as a basis for comparing alternative schemes or designs. Different schemes may have different cash flows necessitating some rational form of comparison. The crux of all methods of economic comparison is the discount rate, which may be in the form of the interest rate on loans or redemption funds. National projects may require a discount rate different from the prevailing interest rate, to reflect a time rate of preference, whereas private organisations will be more interested in the actual cash flows, and consequently use the real borrowing interest rate.

The cash flows, i.e. payments and returns, of one scheme may be compared with those of another by bringing them to a common time basis. Thus all cash flows may be discounted to their present value. For instance one rupee received next year is the same as Rs. 1.05 (its present value) this year if it could earn 5% interest if invested this year. It is usual to meet capital expenditure from a loan over a definite period at a certain interest rate. Provision is made for repaying the loan by paying into a sinking fund which also collects interest.

Methods of Analysis

Different engineering schemes required to meet the same objectives may be compared economically in a number of ways. If all payments and incomes associated with a scheme are discounted to their present value for comparison, the analysis is termed a present value or discounted cash flow analysis. On the other hand if annual net incomes of different schemes are comparel, this is termed the rate of return method. The latter is most frequently used by private organisations where tax returns and profits feature prominently. In such cases it is suggested that the assistance of qualified accountants is obtained. Present value comparisons are most common for public utilities.

A form of economic analysis is benefit/cost analysis. An economic benefit is attached to all products of a scheme, for instance a certain economic value is attached to water supplies, although this is difficult to evaluate in the case of domestic supplies. Those schemes with the highest benefit/cost values are attached highest priority. Where schemes are mutually exclusive such as is usually the case with public utilities the scheme with the largest present value of net benefit is adopted. If the total water requirements of a town for instance were fixed, the least cost supply scheme would be selected for construction.

Local Conditions

In urban and rural water supply schemes majority of the cost of schemes relates to the distribution systems particularly now-a-days when the cost of material has shot up by more than three times. In order to reduce the cost of scheme if the engineer is concentrate on economical planning and design of the schemes, it will give a very healthy effect.

According to present day practice, the design criteria both for urban and rural water supply schemes is :

peak hourly demand : 1.5 times of average demand

seasonal demand : 1.5 times of peak hourly

In this way the pipe lines for peak hour requirement on a hot summer day is designed as 1.5X 1.5=2.25 times the average hourly demand. To be on the safe side this factor is raised to 2.25 times in Public Health Engineering Department Lahore. There is no justification in increasing this factor as 2.25 times is found to be most appropriate factor. In all big towns of Pakistan shortage of water during summers has been experienced invariably. The shortage of water primarily is due to inordinate "domestic and sprinkling needs of water". Besides other factors this shortage is also felt due to inadequate and insufficient source of water supplies. Mostly it has been noticed that only one tube well remains in working order against two to three tube wells which are installed. If the number of tubewells are according to the need of the water supply districts, then the shortage of water will not be felt so badly as is being experienced in hot summer days. Sprinkling of. lawns and pavements during summer, unmetered water connections, innumerable standposts resulting in excessive wastage raise the amount of un-accounted for water. The condition in rural areas is quite otherwise. The number of private house connections is small as compared to urban areas. This leads to the conclusion that the factor of 2.25 for peak demand can be conveniently reduced to 1.25X1.25=1.56 in the design of rural water supply schemes. This factor will result to a very healthy financial effect on the planning and design of distribution systems of rural areas.

The cost of distribution system is also dependent on the type and material of pipes required for distribution network. The comparative cost of pipes made of cast iron, P.V.C. Asbestos Cement and G.I. (for small pipes) is in decending order. The selection of type of pipes plays a very important role in the cost of the scheme. Due to import of cast iron or cast iron pipes; the cost of C. I. pipes is the highest. Although P.V.C. pipes are manufactured in Pakistan yet prices are higher than Asbestos Cement pipes and nominally less than the cast iron pipes.

Although "Asbestos", an important ingredient used in the manufacture of A. C. pipes is an imported item, yet the manufactured cost of A. C. pipes is considerably less than C. I. or P. V. C. pipes. The G. I. pipes having very little life have their own importance for their use of smaller diameters for house connections etc.

It can, therefore, be appreciated that A.C. pipes if selected for use in the distribution system will also make the schemes more economical. Likewise A. C. pipes are as good as C. I. or P. V. C. pipes. Moreover, the use of specials required in the A. C. pipes distribution system is in no way expensive than the specials for C. I. or P. V. C. pipes. The slection of A. C. pipes for distribution system can give a health economical impact on the cost of distribution system and consequently the water supply scheme.

LEAK DETECTION SURVEYS THEIR ROLE

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The following submission which was illustrated was meant as a background to a field visit organised on the following day when a leak detection survey was conducted by the participants of the Training Course in Shadman II Area of the Water and Sanitation Agency Lahore and the use of the equipment was demonstrated.

The Importance of Waste Prevention

Even in the most efficient water undertakings, quite a large proportion of the total quantity delivered into the supply system leaks to waste; and it is every water engineer's aim to reduce this wastage to as low a figure as possile.

The leakage can occur through faulty joints and fittings, bursts etc. in trunk and distribution mains, communication pipes, house services, and distributing pipes and fittings within consumers premises. All these individual leaks, though possibly very small in themselves, add up to a considerable proportion of the total water supplied. Most of this wastage stems from hidden sources. The speedy detection of leaks, their amount and source, and the effecting of the necessary repairs are of extreme importance in obtaining the best service from a water supply system.

Apart from increasing the available supply, a reduction in the amount of wastage represents, for the authority concerned, a considerable financial saving, which can very quickly offset the cost of the special detection apparatus and labour involved in a waste-prevention scheme. There are, however, certain limits beyond which it is not an economic proposition to reduce waste, the cost of the work involved being greater than the value of the water saved. For this reason waste is never totally overcome. A wastage of about 10% of the delivered quantity is usually accepted as normal for a water undertaking. Only the engineer concerned can decide how far to go in this matter, as the effort must be related to the cost of supplying the water.

Loss of water from the sources mentioned above is not only wasteful from the point of view of the cost of its treatment and distribution. It also reduces the pressure available for supplying the whole system at certain times, it may even deprive some customers altogether. This state of affairs, if not checked leads to the possibility of the authority concerned having to restrict the use of water to certain hours, with consequent inconvenience to both industrial and domestic consumers. The reduction of waste by the introduction of a prevention scheme should have the effect of increasing water pressures generally, and possibly of increasing the resources of the undertaking sufficiently to make projected plans for greater storage and pumping facilities unnecessary.

A waste-prevention scheme should therefore be concerned with locating and stopping any leaks in the whole of the reticulation system between the service reservoir and the taps, and other water fittings, on the premises of each individual consumer. It is not necessarily concerned with the so-called, misuse of water by the consumer.

The Development of a Special Waste-Detecting Water Meter

As the charactristics of individual water supply schemes vary so widely, it is difficult to lay down any generally applicable plan for the detection of waste. A carefully considered system of metering is the essential basis, whether the undertaking is concerned with a densely populated city or a sparsely populated rural area. Meters placed at strategic points indicate, by their readings, the occurrence of abnormal flow conditions.

Some small undertakings, which already have their areas split up into several districts, each fed through a conventional totalizing meter, find that they can check wastage by regularly reading the meters and by making investigations in the district concerned when an unusual rise in consumption is indicated.

This is a satisfactory arrangement, provided that the meters can be read every day. Where this is not possible, daily inspection of the bulk meters on the trunk mains supplying the system indicates more serious flow abnormalities, following which a quick run round the district meters shows up the district in which the leak is. Because of the limited flow range of the bulk meter, which is usually differential-pressure operated, it only clearly indicates the very large escapes such as from bursts. Smaller leaks are not necessarily made evident by it.

For this reason and because the trunk and district supply mains of most of the larger undertakings are so numerous, widespread and interconnected as to render the bulk meters useless for waste-detection purposes, the waste detection meter has been evolved. It is essentially a recording-type district meter, designed to show graphically on a chart the flow rate variation in the main supplying the district.

A Scheme for the Detection of Waste Water Meter Districts

The whole of the undertaking's supply area should first be carefully considered and divided into "meter districts", i.e. districts each capable of being isolated by the manipulation of valves, so that, for the purpose of taking surveys, all the water fed into a district passes through one meter.

Each district must be sized so that (a) its consumption is within the capacity of the size of meter to be used (b) a day time survey, by visual and sounding methods, can be carried out in a reasonable time ; and (c) no more than about 20 valves have to be operated for a sectional survey (valve inspection), excluding boundary and circulating valves which are usually shut by the turncock or waterman during the day before the test and opened again the next day. With appropriate valve and by-pass connection arrangements, it is often possible to place a meter so that several meter districts can be fed through it in turn.

It is usually found that existing valves are sufficient to enable each district to be isolated from all the others, but interconnection of trunk and service mains sometimes makes it difficult to select suitable districts. The mains in new systems can, of course, be laid out to provide for the inclusion of waste-detection meters.

It is the general practice to fit the meter on a by-pass with appropriate valves. With this arrangement, it is possible to isolate the meter entirely, if desired—for servicing, say—without having to cut off the supply to the district. This is an additional advantage in the event of supplies being required for fire-fighting purposes, as, with the by-pass open, the flow is not impeded by the resistance of the meter. Also, in these circumstances, the meter does not prevent a reversal of the direction of flow, when necessary.

Having decided on the lay-out of a district and selected the meter location, it is advisale to carry out two preliminary investigations before proceeding to install the meter :

(*a*) Pressure Survey. Isolate the area, as if for a meter run and test the pressures available by fixing gauges or recorders on hydrants at strategic points. This shows whether test conditions can be maintained in the area without causing inconvenience to the consumers.

(*b*) Trial hole. If as is usually the case in a town district, the proposed meter chamber is to be located in a footpath or in the road, the digging of a trial hole at the spot is advisable to ensure that there are no public utility services in the way.

Some undertakings are initially reluctant to install permanently fixed waste meters in all the mains feeding their various meter districts. In such cases, the use of one or two transportable meters has proved quite successful. A pair of hydrants, with an intermediate valve, are provided at each metering point. These are arranged at a distance apart, say at 10 ft centres, so that the installation at every point is exactly the same. The meter, with a short length of G. W. I pipe on each end, is taken from point to point in a van. For setting up, the meter is removed from the van and connected between the two hydrants on a trestle, and the whole unit is enclosed by a canvas cover. The meter is at a convenient height for observation and attention.

Routine or "Open-Reading" Tests

Having installed the waste-detection meter in a district, it is necessary to establish the minimum night-flow characteristic of that district. This is done by means of a number of "open-reading" or "O.R." runs, using 24-hour charts on the meter drum. Day and night flows are thus incorporated on each single chart. For these runs all the boundary valves are closed, so that the whole of the water consumed by the district passes through the meter.

During the day, in a purely domestic district the total consumption is the sum of many small intermittent draw-offs, resulting in an oscillating or shaded record, i.e. the meter pen continually rises and falls over a small flow range.

In a district where consumption is mostly industrial, the individual draw-offs are usually few but large, and last a long time. Consequently, the chart record is peaked or stepped.

In all cases, however, the loss of water through defective pipes etc. is, for all practical purposes, constant and increases the chart reading by a fixed amount.

From about 11 p.m. onwards, the water consumption in a district decreases rapidly until a steady flow is reached between 1 and 5 a.m. This, on the meter chart, is known as the "night line" and the flow is called the "minimum night flow" or "M.N.F." If industrial consumption occurs at night, the night line is not straight and the lowest recorded flow becomes the minimum night flow.

Some districts contain premises, which are supplied from storage cisterns. The refilling of these may continue for most of the night, especially if the cisterns are large and the service pipes small or the pressure low. Here, the minimum night flow recorded does not give an accurate figure for the total wastage. However, a sudden rise in consumption above what experience has shown to be normal is an indication that waste is occurring somewhere. Comparison of the night flows over a period shows whether the waste is increasing or decreasing.

In an average domestic district containing no large storage cisterns, the night flow settles down at a fairly constant value, which may be attributed to waste. In a district combining domestic and trade consumers, there may be a few legitimate draw-offs during the small hours of the morning, and these are shown on the chart. An experienced waste inspector readily interprets the various types of recording obtained.

Judging the seriousness of waste is not easy in a trade or combined domestic and trade district, as there may be a legitimate consumption of water throughout the night. One has to find out the size of this consumption, by enquiry at the trade premises. This is a simple matter if the establishments are separately metered for revenue purposes. Or, if the trade premises do not use water on certain nights, the wastewater-meter record for those nights is the same as a purely domestic one. If there is a large legitimate use of water throughout the night by a tade consumer, it may be necessary to isolate his premises before the start of, or for a short pre-arranged period during the survey.

One can find out whether the M.N.F. is reasonable by comparing the average M.N.F. of a number of O. R. runs with the average maximum day flow. For a normal domestic/trade district, the ratio is between 1 : 5 and 1 : 8. In a purely domestic district it can be as low as 1 : 12 or 1 : 15 and in a mainly industrial district as high as 1 : 3. If the first survey result is within these flow-ratio limits, the average M.N.F. figure may be taken as a datum on which to base the results of later O.R. tests in the same district.

Where there are many meter districts, it is helpful to keep individual graph records of the M.N.F. obtained on each O. R. test. The average M. N. F. can more easily be assessed in this way, as well as the exact amount of leakage.

O. R. tests must be made at regular intervals. Big undertakings with large numbers of meter districts usually make an O. R. test on each district every one or two months. Some of the smaller authorities, where meter districts are not so interconnected, prefer to run their waste-water meters continually, examining the charts daily for wastage.

In this method of determining the M. N. F. characteristic, each district is considered individually. This is best where both domestic and trade consumers are involved, and where the type of consumer and size of district vary greatly from one district to another. In areas containing many mainly domestic districts of similar size, some authorities calculate a "night-flow factor". This is the average M. N. F. of a number of similar districts, in which, by special inspections and tests, the waste has been reduced to an economic level. This factor is the standard to which they endeavour to reduce the wastage in all districts in the area.

Some authorities go a stage further and calculate, monthly, an "area night-flow factor". This is arrived at by multiplying the average of the M. N. F's recorded on all the O.R. tests during the month, by the total number of meter districts in the area (or by the ultimate number of districts, if the area is still in course of being fitted out with waste meters). Comparison of the area night-flow factors over a period shows whether waste is increasing or decreasing.

Detailed District Survey or Valve Inspection

If waste is large, one must localize the leak or leaks. If a serious burst is indicated, a "special survey" of the district concerned is instituted. This involves "visual" and "sounding" inspections throughout the district by day, and possibly by night, until the escape is found.

Detailed Section Survey or "Section Examination"

This survey usually consists of house to house inspections and stethoscope soundings on stop taps, hydrants etc. in the section shown up by the meter until the source or sources of the leakage are traced.

One can, if necessary, extend the valve-inspection test by leaving the water-wasting section under supply and closing as many as possible of the other sections. Closing the consumers stop cocks, starting at those farthest away from the meter, shows on the meter chart the extent of the waste and the service responsible.

Check "O. R." Test

After a valve inspection or section examination, a further open-reading run is carried out to check that all the waste indicated by the previous high M.N.F. has been found and repaired. In a district with a bad waste history, the M. N. F. may still be high, wastage having started again. A further sectional examination or a valve inspection is then necessary, followed by another check O.R. run.

The introduction of a waste detection scheme represents, perhaps, one of the greatest economies a water authority can make. This is so whether the scheme is large and comprehensive, as in the case of a big city, or comprises a few meters for checking district consumption, as for a scattered rural area. The initial cost of the meters is usually negligible, when compared with the value of the water saved during the course of a few years. Trained staff are of course, necessary to get the best out of a wastewater detection scheme.

An idea as to the saving which can be provided by a waste system may be obtained from the example of a large industrial city which is introducing such a scheme. Preventive action in the area initially covered by meters (about 10,000 acres) reduced the waste in that area by 750,000 gallons a day. Another large authority reported that, in a recent year, their waste-meter scheme helped them to detect and prevent wastage amounting to no less than 21.6 m. g. d. In another case, the experimental installation of one meter by a rural district council enabled their waste inspectors to reduce waste to such an extent that it was decided to fit similar meters on three other supplies. The result was that consumption was practically halved ; restrictions which had been put on consumers in previous years and the purchase of water from an adjoining district were rendered unnecessary, even in a long period of drought.

WATER SUPPLY BYE-LAWS

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1. The need for standardization was recognized by the water industry in the United Kingdom many years ago. The principal means by which this was achieved was by the application of

(1) Bye-laws

(2) Regulations.

(3) British Standard Specifications.

2. Application of such standards ensure that the quality of materials used, the standard of workmanship employed and the safeguards incorporated in a distribution system do not fall below a minimum acceptable standard.

3. Therefore they

(*i*) protect the interests of the consumer by ensuring that his water supply is installed in accordance with sound and accepted practices, the risks of pollution and hazards to health are reduced and the reliability and effective life of his supply are maximised.

(*ii*) protect the interests of the water authority by removing the risk of contaminating the supply, preventing the abuse, waste and undue consumption of water, reducing the costs of maintenance and the inconvenience to the public by minimising repair work.

4. In Pakistan the number of Government Standards relating to water supply materials is very limited. Therefore, there is a tendency to refer to the Standards prepared by other countries e.g. British Standards DIN etc. These standards are very comprehensive.

5. Bye-laws are a legal vehicle which allows a water authority to specify its requirements with regard to the installation and operation of water services and to take such steps as may be necessary to ensure their implementation.

6. Bye-laws requirements include disconnecting supplies and prosecuting offenders. They also authorise officers of the authority to exercise certain powers with regard to the payment for the consumption of water.

7. For the reasons stated above the general title given to the Bye-laws is

"Bye-laws Governing Water Supply for the Prevention of Waste, Undue Consumption, Misuse or Contamination of Water Supplied by the Water and Sanitation Agency".

Bye-laws should be reviewed from time to time and updated at regular intervals.

8. The Bye-laws which have recently been redrafted for Water and Sanitation Agency Lahore are divided into eight parts, the main contents of which are summarized below.

Part I—Interpretation

Contains legal definitions.

Part 2—Application and General Provisions

Lays down procedure for applying for domestic and non-domestic water services ;

Plans and details required ;

Necessary consents ;

Fees payable ;

Specifies stop valves required ;

Fire services ;

Public standposts ;

Impose restrictions.

Part 3—Installation and Maintenance

Consumer responsible for maintenance of water service ;

Restricts persons carrying out work on services to licensed plumbers ;

Pumps not directly connected to main.

Part 4—Specifications for Pipes and Fittings

Route and depth of pipe ;

Cover as protection to pipe ;

Joints and fixing ;

Location of stop valves ;

Drinking troughs for animals ;

Buried cisterns.

Part 5—Specifications of Materials

Materials for pipes, bends, junctions, fittings etc ;

Size of service pipe ;

Pipes under roads and footpaths ;

Stop valves, fittings, joints etc ;

Building materials.

Part 6—Protective Measures

Protection of pipes from damage and contamination ;

Inspection of materials and work.

Part 7—Tests

Tests for pipes and fittings to ensure compliance with specified standards.

Part 8—Additional Information

Defines services and responsibilities of licensed plumbers ;

Reasons for disconnecting supplies ;

Misuse of water services ;

Private supplies and new wells ;

Powers of officers ;

Penalties.

Bye-laws contain schedules which :

(*i*) State the periods of time after which a penalty may be imposed for non-compliance.

(*ii*) Set out the rules and regulations controlling licensed plumbers so that their experience and responsibilities are clearly defined for the benefit of the public.

General rule with respect to the making of Bye-laws is that the Bye-laws must be reasonable. They should be designed to protect the interests of the consumer and the authority.

This can only be achieved by rigid application of the Bye-laws and the conscientious surveillance of services by the officers of the authority.

It should be remembered at all times that the authority has the power to

(*i*) refuse to connect a service that fails to comply with the Bye-laws.

(*ii*) disconnect a service which is found to contravene the Bye-laws. This power can be more effective than taking an offender to court.

It will be seen that the enforcement of Bye-laws by a water authority is the most effective means available for ensuring that it maintains a safe, efficient, economical and satisfactory supply of water to the community it serves.

DRINKING WATER QUALITY

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Many substances may be permitted to exist in a public water supply only to a limited extent. Other substances indicate the possible past history of the water and whether it has been, or is now polluted. Certain properties of the water must also be controlled within specified limits to preserve a wholesome and palatable supply.

Water intended for human consumption must therefore be free from organisms and from concentrations of chemical substances that may be hazard to health. In addition, supplies of drinking water should be as pleasant to drink as the circumstances permit. The situation, construction, operation and supervision of a water supply distribution system must be such as to exclude any possible pollution of the water.

The basic requirement to control the water quality is to select the quality parameters of importance. The quality parameters about which most regulations are concerned can be divided into three main categories: Physical parameters, Chemcial parameters and Biological parameters.

Most of the physical parameters have not been given the same degree of attention as the others in the past. Many water supply practitioners have thought their effects as being primarily aesthetic in nature. Recent events have indicated that this situation is changing rapidly. Although the direct effects of these parameters are still considered to be aesthetic, other indirect effects now are being considered to be of importance. Turbidity is an example of a physical parameter being given an increasing emphasis. It has been shown that particulate matter may interfere with the disinfection process and with maintenance of an adequate disinfectant residual. Furthermore, it has been established that the removal of turbidity can maximize the direct removal of pathogens because most of the pathogens are attached to the particulate matter present in water. Color is another physical parameter which may be limited soon. Just as turbidity is a surrogate measure of particulate materials, color may be a crude measure of the humic substances that seem to be primary precursors for the formation of organohalides.

The chemical compounds of concern consist of a number of metals, some anions of health significance, and few other compounds mostly associated with health effects. Among the most toxic compounds arsenic, selenium and lead are encountered most often. The presence of lead is associated generally with aggressive matters and lead plumbing. As a consequence, excessive concentrations of lead often may be reduced through corrosion control techniques. Although the presence of arsenic or selenium is associated sometimes with man-made sources, they also occur at high levels in some natural waters. Nitrate is appearing with increasing frequency in ground water, principally because of agricultural and municipal discharges.

Among the chemical compounds are also included a number of pesticides and herbicides as well as some general indicators of total organic and detergent content. A great deal of research presently is being conducted on organics in water supplies, and indications are that some additional requirements may be established soon. Environmental surveys recently have been completed for aldrin, chlordane, DDT, dieldrin and heptachlor, and regulations for some of these are under consideration. Also likely are monitoring requirements and a maximum contamination level for the trihalomethanes and possibly for some overall measure of total organic carbon.

Microbiological parameters are most important towards maintaining the quality of water. Present regulations use the coliform organisms as an indicator of sewage contamination. The coliform group includes all organisms that are gram-negative, nonspore-forming rods and that ferment lactose to produce gas within 48 hours when incubated at 35°C. Escherichia Coli is one of the most common members of the coliform group. An indicator organism has been chosen because assaying for specific pathogens is not practical. Such an assay is too costly the methods are too slow, there are too many potential pathogens to be evaluated, and the results would be available only after the fact of disease exposure. The coliform group was chosen as the indicator of use because coliforms have been shown to have resistance similar to most pathogens and because the coliform test is a sensative economical test.

Viruses are also an aspect of microbiological quality that receive a great deal of attention. At the present time there are few mandatory regulations regarding the allowable level of viruses in drinking water. The WHO standards suggest that a water that does not show one plague-forming unit (pfu) in a litre is safe from the standpoint of viral contamination. The meaning of such a standard would depend on the methods employed for concentrating and assaying the viruses present, and no concensus has been developed yet regarding the significance of this area of water quality.

The quality levels for the above parameters are specified in "WHO International Standards for Drinking Water (1971)". They give concentration limits for the physical and chemical parameters. For the ideal bacteriological water quality, all samples taken from the distribution system, including consumers premises should be free from coliform organisms. In practice, this standard is not always attainable, and the WHO Publication lays down bacteriological standard for water quality in the distribution system.

Once the quality parameters of importance have been singled out, the problem that remains is one of developing monitoring requirements to ensure that an adequate description of the water quality is developed. The requirements ordinarily included in a monitoring programme may be divided into three basic categories : (1) Requirements describing how samples should be taken (2) Requirements describing the schedule of sampling (3) A description of acceptable analytical techniques.

Sample Collection

The most important consideration in establishing requirements for sample collection is the effort that is made to make the sample representative of the entire flow. A number of other considerations regarding the techniques for storing, conditioning and transporting individual samples are also important.

Depending on a number of factors, a water body might be represented adequately by a grab sample, composite sampling, or by continuous sampling or monitoring. Where flowing water is to be sampled for dissolved materials, occasional grab samples are usually adequate—where large bodies of water or flows of water are to be sampled, when suspended materials are to be sampled, when suspended materials are of special interest, or when variations in water quality are expected, composite samples should be considered. Where process control is of concern, continuous monitoring is often desirable. With composite samples, there are three dimensions to be considered : time, space, and rate of flow. Quite frequently, composites over time provide a satisfactory description of the quality parameter of interest.

A great deal of care in the collection of samples for bactoriological examination is necessary to avoid accidental contamination of the sample during collection. Where several samples are being collected on the same occasion from the same source, the sample for bacteriological examination should be collected first, in order to avoid the danger of contamination of the sampling point during the collection of the other samples. Sterilized glass bottles provided with a ground glass stopper or a metal screwcap should be used ; the stopper and neck of the bottle at least should be protected by a paper or by thin aluminium foil.

The sampling bottle should be kept unopened until it is required for filling. During sampling, the stopper and neck of the bottle should not be allowed to touch any thing. The bottle should be held near its bottom. The bottle should be filled wihout rinsing, and the stopper should be replaced immediately. If a sample of water main is to be taken from a tap, the tap chosen should supply water from a service pipe directly connected with the main. The tap should be cleaned and then flamed to sterilize it. The water should be allowed to run to waste from the tap for atleast two minutes before the sample is collected. When the samples have been collected they should be clearly labelled and sent to the laboratory without delay accompanied by a note of all the relevant particulars.

Changes occur in the coliform and E—Coli content of water samples on storage, and it is important therefore, that samples should be examined as soon as possible after collection. Examination preferably be started within one hour of the collection of the sample, but the interval between collection of the sample and the beginning of its examination should never be allowed to exceed 24 hours.

The samples for chemical examination should be collected in chemically cleaned bottles. The samples should be collected with same degree of care and attention except that it is not necessary to sterilize taps. Samples should be transported to the laboratory with as little delay as possible and should be kept cool during transport. Chemical analysis should be started as soon as is practicable after the collection of the sample and in any event within 72 hours.

Sample Scheduling

Depending on the methods used to analyse the data collected, the schedule of sampling may have a very significant impact on the rigor of a given water quality regulation. The degree of frequency which should be required for analysis of a given quality parameter in a given water supply depends on the health significance of the parameter, the degree to which the concentration of the parameter varies over time, the likelihood of events that might affect the presence of the parameter of concern, and the population at risk, among other things.

The health significance of the parameter is obviously a matter of foremost concern. For example, the potential health risks associated with the presence of pathogens in a water supply justify a high monitoring frequency, whereas the data so far developed on the health significance of high sulfate levels is less compelling. Finally the frequency of monitoring required should be developed with due consideration to the saize of the population at risk. The WHO Internation Standards for Drinking Water can be used as a good guide. The above standards also include recommendations for bacteriological sampling frequency.

Complete chemical examination of all supplies used by public should be carried out once a year. Short routine chemical examination should be carried out once a month with supplies serving more than 50,000 inhabitants, or twice a year with supplies serving smaller populations.

Sampling Analysis

No regulatory programme designed to establish standards of water quality will be effective without a reliable water quality data. In this regard two considerations are of importance requiring the use of reliable standard methods and developing programme of laboratory certification. Most widely used analytical techniques are "Standard Methods for the Examination of Water" because this leads to greater understanding of the limitations of the results reported.

Even where analytical methods are standardized, there may be a great deal of variation among the quality of data reported from various laboratories. As a result, laboratory certification programmes are an essential part of any effort to regulate water quality. As a minimum, certification efforts should include a review of manpower training, reference samples, and periodic inspection of the facilities.

LABORATORY PROCEDURES

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The participants were given a resume of the laboratory procedures relating to important drinking water quality parameters before being required to determine, for given water samples, two of them namely residual chlorine and coliform count using membrane filtration technique in the laboratory.

Residual Chlorine

The prime purpose of chlorinating public water supplies is to prevent the spread of water-borne diseases. Beside its disinfectant properties, it also fills several other plant needs. Chlorine is used in the form of free chlorine or as hypochlorite. Two general types of residuals are produced in a water from the chlorination process—free available chlorine (which refer to chlorine, hypechlorous acid, and hypochlorite ion) and combined available residual (which refer to chloroamines). Residual combined chlorine is a less active oxidizing agent and slower in its bactericidal action than free chlorine.

The OTA (orthotoludine—arsenite) method of measuring free and combined chlorine residuals is based upon the fact that free chlorine residual reacts instantaneously with orthotoludine to produce the yellow—holoquinone, whereas chloramines react much more slowly.

Reagents

Orthotoludine reagent, Sodium Arsenite Solution, Permanent Chlorine Standards.

Procedure

Label three 20—ml test tubes A, B, and C. Use 1 ml of orthotoludine reagent. Use the same volume of arsenite solution.

Free Available Chlorine

To TT A, containing orthotoludine reagent, add a measured volume of water sample. Mix quickly, and immediately (within 5 sec.) add arsenite solution. Mix quickly again and compare with the colour standards as rapidly as possible. Record the result (A) as free available chlorine and interfering colours.

Estimatoin of Interference

To TT B, containing arsenite solution, add a measured volume of water sample. Mix quickly and immediately add orthotoludine reagent. Mix quickly again and compare with colour standards as rapidly as possible. Record the result (B_2). The values obtained represent the interfering colours present in the immeaiate reading (B_1) and in 5 minute reading (B_2).

Total Available Chlorine

To TT C, containing orthotoludine reagent, add a measured volume of water sample. Mix quickly and compare with colour standards in exactly 5 minutes Record the result (C) as the total amount of residual chlorine present and the total amount of interfering colours.

Calculations

Total available residual chlorine $C = B_2$, Free available residual chlorine $- A = B_1$, Combined available residual chlorine=total available residual chlorine—free available residual chlorine.

Membrane Filtration Technique

Membrane filtration technique is used to determine the bacteriological quality of potable water supplies. In this method, the membrane filters that are used are flat, highly porous, flexile plastic discs about 0.15 mm in thickness and usually 47—50 mm in diameter, their pore sizes ranging from 10 milli-micron to 8 microns. For most water bacteriological tests the filters having pores with 0.45 ± 0.02 micron diameter are recommended. The medium used is M—Endo Broth MF, which is a selective, differential medium including components which permit coliform bacteria to develop colonies easily recognizable by form, colour and sheen.

All glassware and other apparatus required for analysis should be composed of material free from agents having unfavourable effects on bacterial growth and should be in accordance with the specifications laid down in the latest edition of Standard Methods. All material used in this method should be sterilized.

Procedure

(1) Prepare data sheet. Minimum data required are : sample identification; test performed including media and methods, sample filtration volumes and the bench numbers assigned to individual membrane filters (2) Disinfect the laboratory bench surface : Use a suitable disinfectant solution and allow the surface to dry before proceeding. (3) Set out culture containers in an orderly arrangement. (4) Label the culture containers : Numbers corespond with the filter numbers shown on the data sheet. (5) Place one sterile absorbent pad in each culture container, unless an agar medium is being used : Use sterile forceps for all manipulations of absorbent pads and membrane filters. Forceps sterility is maintained by storting the working tips in aout 1 inch of methanol or ethanol. Because the alcohol deteriorates the filter, dissipate it by burning before using the forceps. Avoid heating the forceps in the burner as hot metal chars the filter. (6) Deliver enough

culture medium to saturate each absorbent pad, using a sterile pipette. Exact quantities cannot be stated because pads vary. Sufficient medium should be applied so that when the culture container is tipped, a good-sized drop of culture medium freely drains out of the absorbent pad. (7) Organize supplies and equipment for convenient sample filtration. The important point in any arrangement is to have all needed equipment and supplies conveniently at hand in such a pattern as to minimize lost time in useless motions. (8) Lay a sterile membrane filter on the filter holder, grid side up, centered over the porous part of the filter support plate. Membrane filters are extremely delicate and easily damaged. For mainipulation, the sterile forceps should always grab the outer part of the filter disk, outside the part of the filter through which the sample passed. (9) Attach the funnel element to the base of the filteration unit. To avoid damage to the membrane filter, the looking element should only be utilized to apply the locking forces. The funnel element never should be turned or twisted while being seated and locked to the lower element of the filter holding unit. Filter holding units featuring a bayonet joint and locking ring to join the upper element to the lower element require special care on the part of the operator. The locking ring should be turned sufficiently to give a snug fit, but should not be tightened excessively. (10) Shake the sample thoroughly. (11) Measure sample into the funnel with vacuum turned off. The primary objectives here are (*i*)accurate measurement of sample and (*ii*) optimum distribution of colnies on the filter after incubation. To meet these objectives methods of measurement and dispensation to the filtration assembly are varied with different sample filtration volumes. (12) Turn on the vacuum. Open the appropriate spring clamp or valve, and filter the sample. After sample filtration a few droplets of sample usually remain adhered to the funnel walls. Unless these droplets are removed the bacteria contained in them will be a source of contamination of water sample. (In laboratory practice the funnel unit is not routinely sterilized between successive filtrations of a series). The purpose of the funnel rinse is to flush all droplets of a sample from the funnel walls to the membrane filter. Extensive tests have shown that with proper rinsing technique, bacterial ,retention on the funnel walls is negligible. (13) Rines the sample through the mlter. After all the sample has passed through the membrane filter, rinse down the sides of the funnel walls with at least 20 ml of sterile dilution water. Repeat the rines twice after all the first rinse has passed through the filter. Cut off suction on the filtration assembly. (14) Remove the funnel element of the filter holding unit. If a ring stand with split ring is used, hang the funnel element on the ring : otherwise, place the inverted funnel element on the inner surface of the wrapping material. This requires care in opening the sterilized package, but it is effective as a protection of the funnel ring from contamination. (15) Take the membrane filter from the filter holder and carefully place it, grid-side up on the medium. Check that no air bubbles have been trapped between the membrane filter and the underlying absorbent pad or agar. Relay the membrane if necessary (16) Place in incubator after finishing filtration series. Invert the containers. The immediate atmosphere of the incubating membrane filter must be at or very near 100 per cent relative humidity. (17) Count colonies which have appeared after incubating for the prescribed time. A stereoscopic microscope magnifying 10—15 times and careful illumination give best counts. For reporting results the computation is :

Bacteria/100 ml=No. colonies counted X100/Sample volume filtered in ml.

Advantages and Limitations

(1) Results are obtained in approximately 24 hours, as compared with 48—96 hours required for the standard fermentation tube method. (2) Much larger, and hence more representative samples of water can be sampled routinely with membrane filters. (3) Numerical results from membrane filters have much greater precision (reproductibility)

than is expected with the fermentation tube method. (4) The equipment and supplies required are not bulky. A great many samples can be examined with minimum requirements for laboratory space, equipment and supplies. (5) Samples having high numbers of non-coliform bacteria capable of growing on Endo type culture media sometimes give difficulty. In such cases a high ratio of these non-coliform bacteria to coliform results in poor sheen production, or even suppression, of the coliform organisms. (6) In samples having low coliform counts and relatively great amounts of suspended solids, bacterial growth sometimes develops in a continuous film on the membrane surface. In such cases the typical coliform sheen sometimes fails to develop. (7) Some samples containing as much as 1 milligram per liter of copper or zinc, or both, show irregular coliform bacterial results. (8) Occasional strains of bacteria growing on membrane filters producing sheen colonies prove, on subsequent testing, to be acid but not gas-producers from lactose. Where this occurs it may give a falsely-high indication of coliform density. The limitations of this technique are not frequent, but they do occur often enough to require consideration. In samples where these difficulties often occur, the best course of action often is to avoid use of membrane filter methods and use the multiple fermentation tube procedures.

RECORDS IN WATER DISTRIBUTION SYSTEM

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The keeping of records in an essential requirement in any water authority without which efficient operation and development of the authority is not possible. The accumulation and compilation of facts and figures can be of the greatest service or it can entail an utter waste of time and effort. Needless information should not be collected and is time consuming. Before embarking on the collection of data and the keeping of records it is essential that consideration is given to the use which is to be made of the information. This may frequently lead to the conclusion that the time and labour involved is altogether disproportionate to the value of the results.

A clear indication of the value of the proposed records can be obtained by considering the following questions : (1) Which of the water authority's many activities are desirable to keep under close review? (2) At what intervals of time should they be reviewed? (3) What information is required for this purpose and to what use can it be put ? (4) How can the information be most conveniently obtained ? (5) In what form should the information be compiled and presented ? The precise answers will depend upon the characteristics of each authority, but the following may be taken as typical of the technical records necessary for efficient management control. (A) Rainfall (duration and intensity) and and estimated population supplied. This information is normally readily available on request from the appropriate Government Offices, but copies of relevant reports should be held by the authority. (B) Ground water levels (static and dynamic) Daily consumption from tubewells and/or reservoirs divided into domestic, trade, truck supplies etc. Average daily consumption per head of population, analysed as above (c)Water analyses. These should include chemical and bacteriological examinations and reports. (D) Waste detection. Details of premises inspected, leakages discovered and remedial work carried out. (E) Burst mains, giving location, time, type of main, joints, cause of fracture, ground conditions, repairs effected etc. (F) Mechanical and electrical plant. Availability, breakdowns and causes of same, times of outages, repairs carried out etc.

All this information may be presented in tabular or graphical form (or combination of both) but items A—C lend themselves to graphical display so that changes are highlighted. Display charts should be in addition to tabular records and not in lieu of same. Tables present information with a higher degree of accuracy and in form convenient for statistical analysis. There are advantages in one staff member being responsible for keeping the records up-to-date and providing them for regular inspection.

To ensure that the correct information is obtained from the various sources it is usual to prepare printed sheets on which the relevant data is entered and then forwarded to head office for preparation of the permanent records. It is useful if the sheets carry a reference code and they should be reviewed at regular intervals in case they require modifying.

The above records are associated with the operation and development of the authority but other records of historical importance are also necessary. (A) Every authority should have record drawings of the works of the undertaking and maps showing the pipeline distribution system. Each structure or piece of apparatus or mechanism on the works should have a set of drawing showing the work as actually carried out. This is particularly important for those parts of the works hidden from site on completion (B) There should be a set of maps to appropriate scale showing the position, size and material of every main with details of the valves and hydrants etc. attached. In densely populated areas the scale of the maps should be 1 : 500. (C) Inspectors and junior engineers should keep similar field books in which the position of all valves, hydrants, washouts, etc. should be recorded by small sketches with the dimensions to permanent land-marks clearly marked. The book should also record the type of fitting, direction of rotation, whether normally open or closed, material and diameter of pipeline etc. plus any other information which may be of value. Both the detail books and the field books should give cross references to the appropriate main drawings or maps. When a drawing record is amended it should have the date of the amendment marked on it. Comprehensive records should be kept of electrical and machinery items in regular use e.g. pumps, motors, gas engines, vehicles, mobile plants etc. The records should include part reference numbers, installation dates, overhaul periods and dates, correct lubricants, major repairs, fuel consumption etc. etc., in fact anything which will indicate the efficiency of performance of the plant and will permit the operation of an effective preventive maintenance programme. A card index system is generally appropriate for this type of record. It is a good fault to have too much rather than too little information and the keeping of records is really a matter of common sense and experience.

METERING THE WATER DISTRIBUTION SYSTEM

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1. Meters are installed in water distribution systems in order to measure the quantity of water passing a given point. They provide information about the distribution of flows which enable the engineer to calculate demands, check the distribution network, estimate wastage, measure domestic, trade and bulk supplies and to assess future demands.

2. As meters are expensive items of equipment, sensitive to rough treatment and require regular servicing and maintenance, they should only be installed where necessary. Where information is infrequently required, it may be possible to instal a meter for a temporary period only, so that one instrument may be used to serve a number of measurement points.

3. It is essential that meters should be installed on every line of supply so that the total quantity of water being produced each day may be accurately measured. This information is vital to any water authority.

   If the supply passes through storage reservoirs the quantity being drawn from them each day must be measured as this indicates the daily demand.

   It is from such information that consumption statistics for the authority can be prepared. These in turn allow accurate forecasts to be prepared with regard to the future requirements of a developing authority.

4. Meters should also be installed on pipelines conveying bulk supplies of water to or from a neighbouring authority so that accurate charges may be calculated.

5. Bulk supplies to commercial and industrial premises should always be metered to permit accurate charges to be applied.

6. Meters on the discharge mains of pumping installations permit the technical performance of the equipment to be periodically checked as well as the overall output of the plant to be analysed.

7. The merits of placing meters on domestic supplies are open to debate and reports of the benefits derived from such installations vary considerably. It is contended that meters on domestic supplies reduce water consumption and waste, as consumers use

water more sparingly when they know that the flow is being measured and they will be charged accordingly. Similarly those who are conservative in the use of water will reap the benefits of their economy.

It is arguable that a householder should have control over the supply to his property and that the quantity concerned should be at his discretion.

On the other hand meters are expensive pieces of equipment, subject to abuse and error, and in need of regular maintenance.

The cost of supplying, reading, maintaining, repairing and replacing meters is considerable.

8. There should be sufficient number of meters permanently installed in every distribution system to permit an accurate assessment to be made of the amount of water being produced and consumed within the area of supply.

OPERATION AND MAINTENANCE OF WATER DISTRIBUTION SYSTEMS

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Water works management, as reflected in poor operation and maintenance is a serious problem in Pakistan. This is a major problem which still plagues the water works industry even in industrialized countries, especially as it affects the smaller communities. Engineers frequently claim that lack of funds is the real problem. More often than not, however, the real cause is the lack of interest and attention to well known operation and maintenance practices.

A very necessary element in the efficient operation of a water distribution system is a good preventive and corrective maintenance programme. Such a programme is necessary to : (1) Prevent failure of facilities. (2) Detect and eliminate weak links in the system. (3) Determine the type and quantity of materials and replacement parts to be stockpiled for repairs. (4) Analyze how distribution facilities stand up in actual operation as a guide for future installations. (5) Maintain good public relations by making needed repairs before damage and interruption to service occur. (6) Detect and eliminate safety hazards. (7) Distribute work load more advantageously. (8) Reduce the cost of maintenance.

System Inspection

Mains

Because water mains are buried and rarely uncovered or exposed, a comprehensive and systematic routine checking procedure cannot ordinarily be carried out. Nevertheless, by keeping leakage and breakage records, and by making pressure and flow surveys, incipient failures or deficiencies in service can often be recognized and corrected in advance.

Distribution and installation crews, even meter readers and servicemen, should be instructed to note and report any unusual or special conditions which they discover in their routine work. Proper records should be kept of pipe breakage, leak surveys and pressure tests to aid in making decisions relating to maintenance, retirement, and replacement of facilities.

Valves

A well organized valve inspection programme is essential to proper operation. In the larger systems such inspection may well form the responsibility of a special crew doing such work continuously. In smaller systems the distribution crew may have to be assigned for a definite period for a particular type of work.

Valves are provided in a distribution system principally to isolate small areas for emergency maintenance. Thus, most distribution valves suffer from lack of operation rather than from wear. No hard and fast rules can be set up as to how often valves in different parts of the system should be operated for test purposes. The corrosiveness of the water, the rate of deposition of sand or other solids, and the sizes and locations of the valves all have a bearing on the desirable frequency of operation. In a normal system, valves latger than 12 in. should be operated at least once a year. Valves 12 in. and smaller should

be operated at least once every 3 years, and critical valves should probably be operated more often. Valve boxes or vaults located in streets subject to frequent maintenance should be checked annually to see that they are not damaged, filled with earth, or covered over with pavement.

Most systems number the valves consecutively for quick identification and keep a record of the location, type, size make, and time of installation.

Hydrants

Fire hydrants, like valves, are installed essentially for emergency use and should have a regularly scheduled checking programme. Hydrants are particularly vulnerable to damage and failure, because they are exposed. In general, all hydrants should be checked at least once annually. In most systems they are checked and operated twice a year, in the fall and in the spring.

Frequent painting of fire hydrants is an excellent public relations tool, inasmuch as hydrants are usually the only element of the distribution system seen by the general public and thereby able to create a favourable impression.

Customer Services

The checking of services and meter installations is generally a function which the meter readers and servicemen are best able to perform, monthly or quarterly, on their customary rounds. Generally, little can be done in checking the buried portion of a service line. The condition of the meter or curb box should, however, periodically be checked and maintenance should be carried out if required. One of the most frequent sources of suits against a utility is injury caused by tripping over a projecting meter or curb box. Meter readers should be alerted and instructed to report any such displacement for prompt remedy.

Leaking services are another element of distribution reporting which the meter reader can best perform in his regular round of duties.

If the system has some flat-rate customers or inactive metered services, a programme of periodic inspections of them should be established.

The conditions of the customers' meters and the meter installations are other elements of distribution system inspection which the meter reader should perform and report on. An alert meter reader can often spot under-registering meters by a quick comparison with past readings. Meter stoppages should be picked up immediately at the time of meter reading.

Pavement Breaks

Pavement breaks cannot be completely eliminated in the average distribution system. They are an element of operation which probably generates as much ill-will towards the utility as any other single cause. The first requirement of proper repair is speed in effecting it. A stable backfill of the trench is imperative. Temporary pavement surface should immediately be provided upon completion of trench backfill. Generally arrangements are made with responsible paving contractors to effect a permanent pavement replacement as soon as practicali in a large system the repaving may be done effectively and efficiently by the utiity's onw maintenance crew.

Sanitary Checks

One of the most important phases of proper distributionsystem operation is to make certain that water leaving the supply works in acceptable sanitary and aesthetic condition remains so until it is delivered to the ultimate customer. Sampling stations should be estabished at several points in the distribution system, the number depending on the size and extent of the system, and periodic samples of water should be collected and tested to form a continuous monitoring system.

Pressure Tests

Pressure tests in a water distribution system are commonly carried out by attaching a pressure gauge to a hose nozzle of a hydrant, opening the hydrant, and reading the pressure. A pressure record of greater duration can be secured by using recording pressure gauge. It is sometimes advantageous to have several points in the system continuously monitored for pressure so as to evaluate the operation of the system, especially during days of maximum drafts.

Flow Tests

Flow tests are important to determine the efficiency and adequacy of the distribution system in transmitting water, particularly during days of peak demand, and to determine the amount of water available from hydrants for fire fighting. To make such tests it is necessary to induce a flow from one or more hydrants, measure the flow from the hydrants, and note the change in pressures from no flow to full flow at a nearby hydrant.

Pressure Gauges

Most of the tests described above require the use of pressure gauges. Pressure gauges are commonly carried around in service trucks and receive rough treatment. They do not retain their accuracy for long under such treatment. Frequent testing of pressure gauges, whether used in the gauge plant or in distribution testing, is an absolute necessity for accurate results.

Leak Surveys

Leak surveys of the distribution system are sometimes advantageous if accounted for water increases without apparent reason. The amount of unaccounted for water which might be acceptable is dependent largely upon the conditions existing in each system. It might be 25 per cent in one system whereas an amount in excess of 10 per cent in another system might indicate excess leakage or large under registration of meters. Included in such percentages is any water delivered to the system but not metered, or included in a reasonable estimate for flat rate customers.

Equipment And Stores

Careful attention should be given to the selection of the proper tools and equipment for distribution system maintenance. The number and variety of tools and equipment are generally dependent on the size of the utility. In the selection of tools and equipment the following points should be considered : (1) Economic and efficiency value of the equipment, including any reduction of labor cost. (2) Public relations value of the equipment in reducing the duration of service interruptions. (3) Improvement of employee morale and standards of labour. (4) Usefulness of the tools or equipment for construction and installation of facilities as well as maintenance.

For efficient operation and maintenance, an adequate stockpile of commonly used materials, supplies, and replacement parts is essential. A careful analysis of the system requirements should be made to determine what and how much should ordinarily be kept on hand. A perpetual stock inventory should be kept and should include separately new and salvaged materials and equipment. The most carefully planned stock inventory is of little value unless stock is properly classified, described, and identified and can be easily located.

System Records

In order to operate and maintain a water distribution system efficiently, it is essential that adequate records of all details of the system be established and kept up to date. Accurate maps showing the locations of mains, valves, hydrants, and other accessories to the distribution system are important elements of these records.

WATER RATES

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Water is the basic necessity of life. Therefore, the survival of either the individual or the organized community depends on availability of a potable water supply. No matter how limited the supply is, it can at least provide for survival under rigid conditions. As soon as aggregates of people, however, congregate in any one area the problem of potable water supply becomes acute. It requires development of large wells, or lakes, or harnessing of rivers, or storage projects, including gravity mains or pumping through a system of pipelines to the general consumers. Naturally the most convenient source of water supply were tapped in the beginning.

The rising living standards in all parts of the world have tended to increase per capita uses of water. The ever growing population had soon its affect on the convenient sources.

Initially, water rates used to be final cost of quantities of water delivered to the consumer which included interest and amortization on the capital investment, as well as operation and maintenance costs. These latter include, among other things, the necessary labour, chemicals and fuel implicit either in purification or pumping requirements.

However, as communities exhausted convenient sources of water supply and have had to go further afield for additional supplies, and as surface and ground water pollution increased not only the costs of water supply increased but had to include the element of costs for sewage disposal. This necessitates efforts to ensure that ever scacer water resources are not wastefully used. An important means of doing this is to apply such Water Rates that reflect, not the historic costs of utilities operations, but the real resource costs that are incurred as a result of additional consumption. The justification for sinking additional capital investments and incurring operational and maintenance costs will be the willingness on the part of consumers to pay Water Rate reflecting these real resource costs.

Now, we are to examine pricing policies in the light of the relevant objectives and constraints. The role of Water Rate as a means of influencing consumer behaviour is the least familiar aspect. As such emphasis is on the economic aspects of pricing policy.

Any charging system for water supply or sewarage or both must consist of one or more of the following :

(1) A lumpsum payment at the time a consumer connects to the system, determined by one or more of :

- (*i*) The cost of the connection ;
- (*ii*) The size of the connection ;
- (*iii*) Characteristics of the consumer directly relevant to the amount of water to be used or amount and type of sewage generated (such as number of taps);
- (*iv*) Characteristics of the consumer not directly related to water use and sewage generation (such as property value or type of consumer).

(2) A periodic fixed payment determined by one or more of :

- (*i*) Characteristics of the consumer directly related to the amount and type of sewage generated (such as meter inlet size, number of taps, presence of a garden, industrial process) ;
- (*ii*) Characteristics of the consumer not thus directly related (such as type of consumer or property value).

(3) A periodic payment determined by metered water consumption. It may be a single rate or in blocks ; it may vary seasonally, by type of consumer or property value ; it may reflect the strength of sewage and it may differ between areas.

The choice from among these options of the structure of charges and the choice of their appropriate level has to be a compromise between four main aims. These will now be considered one at a time, after which the possible conflicts between them will be discussed.

The first aim is to raise some target level of total revenue. The target needs to be set so that the utility can service its debts and maintain the degree of financial independence necessary for it to be an efficient organization. This requirement is usually the dominant one, since it involves not only covering all current costs and debt service but also making some contribution to future expenditure. How large this contribution should be can only be a matter of judgement, but one relevant factor is the availability of capital from other sources. Where fiscal resources are especially limited, a high degree of self-finance will presumably be desirable.

Since required revenue is a cash-flow concept, the above considerations need to be looked at in cash-flow terms. But this does not mean that the target actually has to be expressed in such terms ; given the depreciation rules and given the book value of assets the gross revenue target can be translated into a target net rate of return on capital. However it is important to realize that such an accounting rate of return on total assets is a very different concept from a calculation of the discounted cash-flow rate of return on a new project.

Given the total target cash-flow of gross revenue, the second aim is to share out this burden fairly between the different users of the system. The question of whether some part of the revenue should be provided by local or central government requires some thought. There may, for example, be cases where subsidy to poor consumers who use public standposts, or costs related to storm water are more fairly provided from general taxation than by charging only the users.

The third aim is that of administrative simplicity and efficiency. The strength of this consideration will naturally vary according to the competence of the utility and the characteristics of the consumers. Not much is said about this aim in what follows, but that is because it is obvious not because it is unimportant.

The fourth aim is the one that is most generally neglected, namely that of influencing consumer behaviour. In the short term this aim is to induce consumers to economize when there is a drought or when capacity is inadequate. More generally, the aim is to reflect the costs of system expansion in charges in such a way that consumers only choose to impose such costs when they are willing to bear them. A poor country like us needs to be very sure that it devotes scarce resources to water and sewerage only when this is at least as good a use of the resources as other kinds of investment.

The costs which are relevant to the aim of influencing consumer behaviour are the value of the resources which are made unavailable for other purposes by being devoted to water supply and sewerage. Sunk costs are thus irrelevent and it is the costs of further system expansions which matter ; engineering cost estimates rather than historical accounting costs are therefore needed. The aim is to reflect these costs in the charges which affect user choices. If there is no conflict with the other three aims this would require, for example :

—low charges when additions to capacity can be provided cheaply ;

—an incentive to reduce the strength of industrial effluents when this would lead to savings in treatment cost or a desirably improved standard of treated effluent from sewage works ;

—a greater incentive to reduce water use in summer than in winter in cases where capacity and hence costs are predominantly summer use related.

For consumers currently lacking piped water or main drainage, the costs which need to be ascertained are those of extending the system to provide them with service. For consumers who already have service but whose use is growing, the relevant costs are those of adding to existing capacity. In either case what has to be estimated are the additional costs resulting from additional use. The basic notion is thus that charges which vary with the use of the system should reflect the rate of change of system costs with respect to volume. This is what is meant by charges which reflect "marginal" costs.

Because new water supply schemes and sewage works are usually large units and because new mains and sewers may combine the purposes of reinforcement, extension and replacement, a refined analysis of marginal costs may not be possible. But this need not deter the planning engineers from deciding what sort of incentive structure would have to be provided by the charging syseem for it to convey a sensible message to suers. Exact calculations are not required ; the point is to reflect the approximate order of magnitude of the costs of system. This notion of simultaneously informing and inducing the consumer to economize most when economy on their part would be do most to save scarce resources is, however, easily confused with the entirely different notion of allocating costs between consumers. An example will make this clear. Suppose domestic water consumption is closely related to property values. Then a fixed charge related to property value would approximately allocate costs between consumers according to consumption. Yet the incentive effects would be zero, since no consumer would save money by using less water or be charged more if he used more. Thus whatever the fairness or unfairness of such charges (a matter of the second aim) they would do nothing to realize the fourth aim. This, to repeat, is to influence user behaviour.

The distinction is so important that another example will be useful. Consider the collective metering of a block of flats. This makes the payment of all the families in the block vary according to their aggregate use, something which may or may not be deemed fair. But whatever subjective judgement is made on this point, and whatever the administrative advantages of collective metering, the incentive effects of the charging system are minimal. The individual family pays scarcely any more if it uses more and scarcely any less if it uses less.

The last two examples illustrate very clearly the point that the aims of revenue raising or fariness, of administrative simplicity and of influencing consumer behaviour can conflict with one another. This is why the choice of a charging system may involve a compromise. No general rules can be laid down about how to weigh up the achievability and the importance of the four main aims. But there are nevertheless three useful approaches to be adopted in seeking to reconcile them.

The first is to recognize that because judgements of fairness are subjective, sometimes reflecting no more than political expediency, they are not unique. Thus to judge one system of charging to be fair does not rule out all other possible systems.

The second is to recognize that while the aim of influencing consumer behaviour relates to the total charges payable by a potential consumer who is deciding for or against connection, things may well be different with existing consumer. Where they are extremely unlikely to seek disconnection, their behaviour will be influenced by the way their charges vary with use but not by their total level. Suppose, for example, that water and sewerage are to be jointly charged for by a semi-annual fixed charge and a charge per thousand gallons of metered water use. The aim of reflecting system expansion costs imposes limitations on the fixed charge only if it is use-related. But if it is determined by some non use-related characteristics chosen in relation to the metered rate as to make the consumer's total bill constitute a fair contribution towards the required revenue.

The third useful approach to reconciling conflicting aims relates to administrative simiplicity versus influencing behaviour. The latter demands metering or ascertaining some use-related magnitude such as appliance ownership and either of these adds to administrative burdens. But most of the disadvantages of administrative complexity can be measured in cost terms ; the more complex a system is, the more it costs to initiate and run it without any increase in fraud. For the important choice between metering and not metering a particular group of consumers for example the minimum reduction in their average annual water consumption which would be required for metering to be preferred can be calculated. It is that reduction which would make the saving in water and sewerage system expansion costs as large as the cost of metering. This requires information about :

(*i*) The capital cost of procuring and installing meters ;

(*ii*) The annual cost of meter reading maintenance and billing ;

(*iii*) The future cost of expanding the water supply and distribution system, plus the corresponding operation and maintenance costs.

(*iv*) The relationship between decrements of water use and the rate of flow of sewage :

(*v*) The future cost of expanding the sewage collection, treatment and disposal system plus the corresponding operation and maintenance costs.

Item (*iv*) and (*v*) are relevant only when reduced water usage will lower sewage collection, treatment and disposal costs. They are, of course, the same costs as are relevant to fixing the metered rate.

It would not require refined and accurate calculation to determine a net cost saving.

In case, water consumption is likely to be reduced more than enough to secure a net cost saving by a charge which is not excessive, only then metering is probably worthwhile.

In case expansion plans show there to be significant differences between seasons or between areas in the costs of adding to operating capacity, the reconciliation of the third and fourth aims requires similar calculations.

Metering may turn out to be of dubious value in the case of poor consumers. Even if metering is on balance cheaper than unrestricted supply, the installation of some flow-limiting device may be preferable. It costs less than a meter and the consumers can pay a single simple fixed periodic charge. Possibilities of this sort merit examination when water supply is being extended to poor urban areas.

In large community there will be aggregate of consumers having varying living standards and localities having different distances and elevations. The per capita consumption will therefore vary according to the standard of living. Providing water supply to meet such demands will cost more than a uniform low per capita consumption. It presents some justification for subsidizing low income groups.

Therefore, the issue of whether or not to subsidize particular groups of consumers often arises. It can best be looked at in terms of the aims of pricing policy since this enables one to distinguish three quite separate reasons for a subsidy. Even though more than one of these reasons may apply in any particular case, clear thinking demands that they be separated. They are :

(1) The willingness to pay for water supply and/or sewerage under-states the strength of the case for providing it at subsidized rates either because the consumers are poorer than is considered desirable or because there is not only a benefit to them but also to their neighbours in terms of amenity and health.

(2) A reduction in charges for water and/or sewerage will constitute a transfer of income to a deserving group of consumers.

(3) The cost of charging for water from standposts or for communal waste facilities such as public latrines outweighs the benefit.

The important feature of these is that (1) relates to the aim of influencing behaviour, (2) relates to the aim of fairness and (3) to the aim of administrative simplicity. Thus in (1) the purpose of subsidy is to encourage use of the service, in (2) the aim is to leave existing consumers with more money to spend on other things, while in (3) the aim is to save administrative costs.

It is not sufficient to examine the case for a subsidy solely in these terms. The subsidy must come either from other users of the system or from the general tax payer.

The effects on their behaviour, the fairness of making them pay for the subsidy and any extra administrative complications in raising the money from them all need to be considered. Thus subsidy of a group of poor consumers for reasons (1) and (2) might on balance be a bad idea if it were to be financed by extra taxation of some items predominantly consumed by the poor, including those who lack piped water.

A lumpsum installation charge upon connections can have some effect upon the number of connections. If they do, then their level has to be looked at in relation to other parts of the tariff structure and not in isolation. For consumers not judged to need a subsidy, it is usually the total charges which will influence their choice for or against connection. Thus it needs to be reflected in the addition to system costs caused by their connection. Where the choice lies between a slum outside the limits without water or sewerage, or a dwelling within the limits with higher charges for the services than people are willing to pay, then development is unduly handicapped.

A metered rate is at the opposite extreme. Whether or not its proceeds are used for sewerage as well as for water supply if it affects water consumption it will usually affect the amount of waste water to be handled by the sewerage system as well. Hence the costs of both must be brought in, as was suggested earlier. These costs, as a rough order of magnitude need to be compared with the effective incremental rate paid by consumers. The sort of thing to look for is :

(*i*) Rates which are the same in summers as in the winter season, even though the risk of shortage or the need for investing in more capacity results exclusively from summer season conditions.

(*ii*) Rates which fail to reflect significant differences in the pumping or capital costs of supplying different areas.

(*iii*) Rate differences reflecting no cost differences, a result which can easily be produced by some consumer class differentials or by block tariffs.

Complicated block tariffs are fairly common. They often result in differences in the effective marginal cost of water as borne by consumers which reflect no corresponding differences in the marginal cost of supply. In such cases it is important to consider whether the implicit, subsidy to those consumers effectively paying less is justified and whether the burden of the cross-subsidization is appropriately distributed. A cheap first block may suffice as a simple way of making minimum water requirement available to all consumers at a low price. If nearly all consumers take more than this first block it needs to be looked at together with the fixed charge.

The main point is that as the level and structure of charges can affect consumer behaviour and hence system costs they should reflect the way those costs vary with the use of the system. The relevant system include sewers and sewage disposal whether or not these are financially separate. Revenue-raising, fairness, and administrative simplicity are the other objectives of a charging system. Compromise between all four objectives will be necessary.

A revision of water rates normally raises severe political and administrative problems, so that all that is possible initially may be to improve it rather than to move over all at once to a well conceived system. The importance of tariff on the lines discussed would invoke :

—Analysis of system cost structure in forward looking terms ;

—Considering where metering or flow limitation devices are appropriate ;

—Formulating cost-reflecting tariffs ;

—Articulating the revenue and fairness objectives ;

—Modifying the tentative cost-reflecting tariffs in the light of these objectives ;

—Examining necessary institutional and administrative improvements.

A well conceived system can be devised only through joint efforts of multi-discipline experts. However, aspects discussed herein will provide the engineers concerned with planning and maintenance of water supply system the understanding of the problems relating to Water Rates.

MANPOWER UTILIZATION

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Introduction

Today we will be discussing the manpower utilization with specific reference to planning and preventive maintenance of water distribution systems. In-Service Training Courses like the present one for professionals are by definition conferences which make extensive use of participative methods, including group discussions. In-service training is a grossly ignored subject generally, and particularly in technical settings, such as the water works industry, where management tends, possibly justifiably, to concern itself disproportionately with the mechanics of operations to the detriment of administration as used in the broadest sense.

It should be emphasized at the outset that training is a definite function of management. The primary responsibility of management is, of course, to carry out the principal functions of an agency as well as it possibly can, but whatever training is necessary to reach this primary objective should be provided by management. Needless to say, training of some kind necessarily takes place in most job situations, but it is the responsibility of management to see to it that this training is systematic, not haphazard ; that it is efficient, not wasteful ; and that it is effective, not useless.

In evaluating the merits of any formalized training programme, one consideration should be held uppermost in mind ; the success of any training programme is dependent, in the last analysis, on the attitudes of employees and on their desire to learn. Employees can be required to sit in conference after conference, given reams of material to study, lectured incessantly and indefinitely, and generally go through all the motions yet no training will take place unless the employees actively participate in the training process. And this, quite simply, can be brought about only by making the learning process interesting.

Within the frame work of a civil service system the training of a new employee should be one of the noncomitants of the probationary period, but whether this is recognised or not, there is a definite, undeniable need to make sure that the new employee has a clear understanding of his duties and how he is to perform them ; to whom he is responsible, and for what? The author recalls an actual administrative situation in which an extremely well qualified individual was asked to accept the position of comptroller. For the first six months after accepting the position he did virtually nothing but watch and listen; for the most part he merely asked questions. And of all the questions that he asked the more than 300 employees in the office, the most revealing were ; "Who is responsible to you" and "To whom are you responsible "? Why were these questions revealing ? Because a surprisingly large number did not really know to whom they were responsible, and a lesser (but still surprisingly large) number contended that a number of employees were responsible to them, which in fact, was not the case. In other words, there were more bosses than bossed. This is a situation that is much more likely to occur in a large organization, but there is a moral for management in this little anecdote, and that is : make sure that your employees know to whom and for what they are responsible and who is responsible and who is responsible to them.

Beyond this, there is need for the new employee to learn certain information about the organization as a whole. A programme to satisfy this need is commonly known as a "general orientation training programme", and its scope and content depend, in large measure, on the educational level of the employees at whom it is aimed.

There are four primary aspects or objectives to achieve the success of an in-service training programme. The first of these is the need to increase the effectiveness of the employee in his present job. The second objective of training present employees is to prepare a certain employee or group of employees for promotion. Of the several different types of training possible, this type is most generally neglected, yet it becomes of paramount importance when the employee is being offered a carere in the public service. Regardless of promotion policies, however, the need for training employees for this prupose becomes acute when the work is specialized perhaps to the point of being peculiar to a single agency and requires long experience to master. In cases of this kind and the water works industry affors a prime example---the agency has not alternative other than to develop within its own staff the experts it cannot precruit from outside. The third aspect of training present employees is retraining, briefly, retraining is necessary whenever an increased range of versatility is required of any specific employee or group of employees, and it should not be overlooked that such employee versatility creates a personnel reservoir upon which management can draw during emergencies, for greater flexibility and continuity of operations. Retraining is also sometimes necessary when a competent employee is dislocated from a particular division of operations because of a significant change in the work load, and must be transferred to a job which is radically different from that which he previously held. The fourth primary goal of training present employees is the development of what, in personnel circles, is sometimes referred to as "organization fitness". This might be defined as an understanding and appreciation, on the part of the employees, of the organization's objectives. It is characterized by breadth of view, openmindedness, a readiness to learn, recognition and abhorrence of inefficiency, and a sincere desire to serve the public. In fact, the "organization fitness" is a frame of mind, a particular habit of thought, and as such, not easily achieved. If it is ever to be achieved, however, it will be only through the utilization of systematic, efficient, and effective training procedures. Other and more precise comments are that ; (a) Training can represent in financial terms an investment inhuman resources greater than is normally made in physical resources e.g. water systems. The water manager is properly concerned and vigilent about his return on investment in physical assests and should equally recognize and accept that a training investment in people will increase the yield from this more valuable resource. (b) Skill and knowledge about a job or work are best acquired through an appropriate blend of organized training and supervised experience. If the acquisition of skill and knowledge is left solely to unsupervised or inadequately supervised experience on-the-job (exposure training) employee performance will develop slowly and may never reach an acceptable level. Organised and systematic training will assist water works staff, at all levels, to reach the standard of an experienced person in the shortest possible time. This will reduce unit costs. One of the problems with us in addition to setting up the training scheme, is to have established standards of performance at which to aim. (c)

Waterworks employees have followed an inevitable pattern and are becoming, perhaps more quickly than anticipated, organized into unions and associations. The functions of collective bargaining and negotiation are likely to increase, as well the demand for joint consultation. The primary reason for providing good training is not, nor should it ever be, to equip the manager with a bargaining too. Sooner or later however, the manpower and employment considerations that arise in almost any industrial forum for negotiation and consultation will begin to exert pressure upon the need and demand for training of one sort or another. It becomes a matter of enlightened self-interest for the waterworks manager to develop training awareness and to demonstrate this in practical terms. In so doing this will help him to retain the initiative when negotiating or bargaining with employee or staff representatives. A policy statement on training may seem to offer a simple answer except that writing such a statement and making it meaningful, is easier said than done, as many developed countries have discovered. (d) Closely allied to this trend is the growing recognition among the lower echelons of the waterworks labour force that job aspirations and career expectations should be no less available to them than they are to other levels of employment. The message appears to be that preoccupation with training the professionals and technolgists, important though this is, has gone on long enough. In this sense the reason for training is one of maintaining the right balance of skill, knowledge, performance and motivation at all levels of the organization pyramid. (e) The rate of urbanization and the consequential overload on water supply and sanitation services, already under strain, will inevitably create additional hazards to the health and hygiene of the people. A reason for training here is to make the people aware of how best to use and not misuse or abuse the previous public and domestic water installations which exist. Suffice to call this public relations and the waterworks man who neglects this does so at this peril.

The participants of the course understandably should know better than anyone the extent, nature and variety of their manpower and training problems and where, in this spectrum, exist the priority gaps to be filled. Let these therefore emerge in the discussion stage of this programme so that those of us who have advice or, hopefully, direct assistance to offer can take careful note for future activity. Perhaps the present discussion can help the Institute to arrange further training course accordingly.

The nature of waterworks operations is such that the labour force is scattered thinly across the area of supply. There is a problem of time and space ; operational control is impeded particularly for the supervisor. In such an industrial situation there is a strong argument for the workforce to be versatile and multi-skilled. But with such dispersion, how can this be achieved by training on-the-job ? How can there be enough trainers to go round and what training use can be made of supervisors and technicians in this situation ? The problem is to identify the appropriate mix of centralised and decentralised training, particularly at the operator and craft levels of employment.

The managers and supervisors have, as an integral part of their duties by definition, a clear responsibility to train, tutor and develop subordinate staff. The paradox is that it is the operational managers and supervisors who have the greatest accumulation of knowledge and experience to impart. However continuing preference appears to be given to the training and career development of professional persons and technologists, particularly in relation to the limited amount of overseas training which is available. With present priorities as they are the greatest pay-off in seconding staff into other countries for training programme is likely to be achieved through sending carefully selected potential training staff to a country which is operating a comprehensive training scheme of the kind now being recommended for developing countries.

There is a tendency to over-man the public service enterprise. This probably arises from un-employment and political pressures. The corollary to this is usually under utilization of waterworks personnel, employment inflexibility and a reluctance to learn new and improved job methods. Manpower planning is as important, if more difficult when obliged to carry a surplus of manpower as it is when manning levels are based upon work measurement or when the labour market is tight. The problem remains, how to quantity the amount and quality of training required and this could be regarded as a high priority. Water Byelaws and their enforcement are part of the answer and a training gap could well exist in this connection. A well illustrated training manual covering the interpretation and application of a set of model byelaws is the answer.

At what level to start the training ? There can be no standard answer to this question. It depends upon the problems and priorities. A widely held view is that any scheme of training must start with the managers. If the initial training effort is to be adhoc there is considerable merit in starting at the level of the supervisor and working outwards from this level.

It is more difficult, because of dispersion, to train water distribution personnel on-the-job than any other category of waterworks staff. Fortunately it is not difficult to stimulate water distribution operations and short concentrated period of centralised off-the-job training is recommended followed up by a set programme of supervised practical experience.

Training water treatment staff, particularly the operators, does not lend itself to classroom or centralised training. The plant is difficult and expensive to simulate and is widely dissimilar. Experience has shown that the best results are obtained by training the plant managers and supervisors off-the-job to train, in turn, the remainder on-the-job.

MANAGEMENT NEEDS

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Water supply undertaking requires sound management practices because its success depends primarily on the quality of the management. To be successful a water supply system should provide an ample supply of potable water to the people in its jurisdiction while at the same time maintain fiscal soundness. For this purpose it is necessary for the management to know how to communicate with and develop the talent of the younger generation of engineers, to be flexible enough to change and to be able enough to provide the right kind of leadership. Modern times require that water utility managers use new and sometimes sophisticated techniques. Special programmes and institutes devoted to management can aid managers to accept changes and adapt new ideas to their needs. Through such programmes, managers can be given a new insight into communications up and down the heirarchy, utility staffing, management systems, utility computer applications, manpower planning and many other techniques.

In our discussions today we will briefly consider management quality, organization, fiscal policies, sources of revenue including water rates, billing, accounting and auditing procedure insofar as they affect the management of a water supply system.

The organization headed by an energetic and capable engineer seems to make a faster progress and to have a better chance of success. Managers interested in their work, dedicated, capable and enthusiastic are able to make an impact on the work quality of the entire set-up because this spirit seems to permeate the organization of the undertaking they are managing. This increases the prospect for successful operation. The manager must be capable, qualified and experienced. The remuneration and conditions of employment must be sufficient to attract and hold such a man. Besides being an engineer he will need a good working knowledge of financial matters. Administrative ability is an essential requirement.

Management of water supply systems in our country is lodged generally in a provincial agency or a local authority with complete responsibility for planning, financing, designing, constructing, operating and maintaining the facilities. The water utility can probably operate more successfully if it is atleast semi-autonomous. Regardless of its organizational level within the government structure there should be no doubt about its ability to levy and collect the revenue necessary for its successful operation.

Methods of financing watersupply works can range from complete subsidy to fully self-supporting systems. Of course, even under a complete subsidy, funds required are raised from some taxing source. Water utility should be as nearly self-supporting as the local economy can justify. As far as possible, consumers should bear their share of the cost of the water supply, preferably by direct charge. Such a policy helps to decrease the amount of wasted water. Three basic cost elements need to be considered for determining the funds needed for successful operation of a water utility. They are current operation and maintenance costs, capital cost, and reserve funds ; the latter being set aside to meet future demands for replacement and extensions.

The most difficult and desirable method of obtaining the income required for maintaining an adequate water utility is some arrangement for collecting revenue from the water customers themselves. Water rates were discussed with you in an earlier lecture. A brief reference to it at this stage is considered in place. Water rates can be classified into five categories as flat (uniform) price, unmetered variable price, metered with decreasing unit price as volume increases, metered with constant unit price and metered with increasing price as volume increases. Nearly all rate schedules for metered water service include a minimum charge that, to a degree, covers the cost of the utility's readiness to serve. Normally, this minimum charge permits the use of a stated minimum quantity of water per billing period beyond which quantity the next rate step takes effect. Other sources of revenue for the support of water supply operation and maintenance include frontage assessment or special property improvement taxes, business operating license fees, plumbing fees; connection charges and customer deposits when service becomes available. There are examples of water utility being prevailed upon to provide free water for such public service as fire protection, public hydrants and public institutions. This is not good practice and can be the difference between a financially self-sufficient water utility and the one that is not.

The use of water meters is far from universal. Certainly the frequent experience of finding a substantial proportion of installed meters not working together with a meter repair shop incapable of handling more than a small fraction of the indicated meter maintenance load, weighs against the principle of complete metering. Nevertheless careful consideration must be given to metering even in the face of such difficulties. No other means even approaches the same degree of management control in terms of accounting for water uses, which may eventually be reflected in very large capital savings for supply, treatment , and distribution facilities not actually needed if waste and careless losses are kept under reasonable control.

From the standpoint of good management the water authority should be empowered to cut off the water services for nonpayment of bills. Without such authority management cannot maintain a viable system. Having this authority and making use of it will keep delinquent accounts to a minimum and thus reduce the book-keeping workload. The importance of regular and prompt billing should be emphasized. Good accounting and auditing procedures not only for capital improvement projects, but also for operation and maintenance are necessary. Annual external audit should also be carried out at all levels. Internal audits are a proper management tool, but there is no substitute for periodic external audits to safeguard public funds. It should be realized that good accounting and auditing procedures are important in evaluating the overall operation of the water system.