MINISTRY OF LOCAL GOVERNMENT, HOUSING AND CONSTRUCTION

NATIONAL WATER SUPPLY AND DRAINAGE BOARD

SRI LANKA

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DESIGN MANUAL D2

URBAN WATER SUPPLY AND SANITATION

MARCH 1989

WATER SUPPLY AND SANITATION SECTOR PROJECT

(USAID SRI LANKA PROJECT 383-0088)

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URBAN WATER SUPPLY AND DISTRIBUTION

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1. INTRODUCTION

This design manual is intended to assist NWSDB Engineers in the design of urban piped water supply schemes, both for new construction and rehabilitation of existing works. It is a development of the Design Manual on Small Community Water Supplies, prepared for NWSDB with WHO assistance in 1982 (Ref.1),* and includes all relevant material from this earlier manual, which in most cases has been reviewed and updated.

This new manual is substantially more comprehensive than the earlier manual and includes additional sections on hydrology, flow measurement, small dams and intakes, pumping stations and more thorough coverage of pipelines and distribution design, and service reservoirs. It should be used in conjunction with other manuals in the same series, particularly:

- D3 Water Quality and Treatment
- D5 Mechanical/Electrical/Instrumentation Design and
- D1 Rural Water Supply Design, which may have some overlaps concerning minor rural piped schemes.

The manual has been prepared by G. A. Bridger, Environmental/Sanitary Engineer, USAID Project with the assistance of NWSDB Engineers, using material from a variety of sources including the earlier manual. Subject items to include in the manual were established through use of questionnaires to all Engineers and a Design Manual Committee. Principal sources are listed in the references, most of which are available in the NWSDB Library.

In a similar way to the earlier WHO manual, the intent of this new manual is to develop standard practices in the use of design criteria and preparation of engineering designs and drawings. It should be emphasized, however, that the design principles presented in this manual should in no way limit the professional judgement of individual Engineers. Deviations from accepted design criteria may be appropriate provided they are technically sound and fully discussed as part of the design process.

In NWSDB there has until recently been no systematic design review procedures involving experienced Construction and O&M personnel, nor have there been systematic post construction evaluations to provide feed back to the design process. Such reviews and evaluations are now being implemented with the assistance of the USAID Project (e.g. Technical Coordination Guidelines, Project Evaluation Procedures, Commissioning Guidelines (Ref. Procedure Manuals P4, P6, and P3 respectively).

If these processes work, there should be a need to periodically reevaluate design principles and criteria and to introduce revisions or additions to the manual as necessary on a regular basis. It is recommended that this be done annually by a Design Manual Committee under the supervision of the DGM (P&D), with representation from Construction and O&M departments included.

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* A list of specific references and a general bibliography are given after text.

2. BASIC DESIGN CONSIDERATIONS

2.1 Appropriate Technology

In the design of water supply and sanitation facilities an appropriate technological approach to design should be adopted, which simply means designing facilities that will be suitable for the job, accepted and used by the community which they serve, and straightforward to operate and maintain with the level of operator expertise available. If these criteria are met then it is more than likely that the facilities will work well for a long period of time.

In addition, the facilities should be cost-effective and straightfoward to design and build. The use of the term appropriate technology implies simplicity, reliability, efficiency in energy use and generally a less sophisticated approach to design. It does not, however, imply an inferior solution or reduced level of service to consumers - on the contrary it should provide a better, more reliable, more energy efficient service at a cost affordable by the community.
(Reference 2)

2.2 General Principles of Design

The objectives of any water supply scheme are to supply safe water, in adequate quantity, conveniently located, and at reasonable cost to the user. The basic considerations for sound engineering decisions are the area and population to be served, the design period, the water demand, the selection of the water source and the nature and location of transmission and distribution facilities to be provided.

The source of water and the distribution are the two main factors affecting the cost of the project. The nearer the raw water source of good quality and adequate quantity, the lesser the cost and the greater the reliability of supply. It is important to ensure the best design which would provide the community with a reliable, safe and adequate water supply at least cost, both capital as well as recurrent. Affordability and willingness to pay are other factors that determine the viability and social equity in service.

In applying the concept of appropriate technology, some basic principles should be followed:

- Energy conservation: design systems for less use of energy and better energy efficiency. Minimise unnecessary pumping.

- Suitability: design systems that will do the job required, no more, no less; in other words do not over-design or under-design.

- Standardization: do not introduce new, untried methodologies or equipment when existing methodologies or equipment are working well. Conversely, do not repeat design mistakes in the interests of standardization.

- o Community acceptance: an important aspect often overlooked in urban scheme design. The design should meet the needs of the community it serves in terms of affordability, quantity, quality, period of service, access to standposts, etc.

- o Ease of O&M: design from the standpoint of how the scheme will be operated and maintained. Consider O&M tasks and provide necessary access and facilities. Visit similar existing facilities to assess potential problems and consult with O&M regarding staffing, operating shifts, level of expertise and training. Provide for essential spare parts.

- o Equipment and Materials: utilise locally available facilities where feasible and consistent with maintaining quality and ease of O&M.

  Attention to details is important: common problems or items overlooked are as follows:

  - o various details in bill of quanitities;
  - o poor architectural finishes - ceilings, walls, floors, external appearance;
  - o inadequate bar charts;
  - o landscaping;
  - o highways/railways/local authority building approvals;
  - o congestion, lack of space in pump station;
  - o low headroom, small manholes;
  - o communications systems;
  - o problems of plant installation.

2.3 Site Planning

This is an important aspect of design which needs to be considered at an early stage, to ensure that the facilities are located on site in the most efficient manner, which can make a big difference in capital costs and ease of O&M. The following is a check list of points to be considered:

- o location;
- o elevation
- o flood levels;
- o area required for future extensions;
- o cost of clearing, earthworks, embankments;

- site drainage and wastewater disposal;

- foundation conditions;

- security of site;

- access and roads, particularly access to intake site which should preferably be close by to allow ease of monitoring and supervision;

- cost of utilities;

- O&M facilities.

In particular:

- Location and elevation in relation to other elements of the water supply scheme: for instance, the closer the supply is to the treatment plant and/or storage and distribution, the lower the costs of constructing conveyance systems and operating the scheme.

- Drainage: careful design of site drainage should be included to prevent flooding and possible contamination of the water supply. Drainage should be designed so that it effectively removes excess water without causing soil erosion and damage to adjacent property.

- Expansion possibility: the availability of usable land for future additions or expansion to the original installation needs to be considered. The terrain and shape of the land should be suitable for expansion without entailing expensive construction.

- Physical characteristics: in evaluating a site, the amount of site preparation for the proposed structure should be checked. The existing topography may be used to advantage minimising the costs of clearing, grading, excavation and drainage.

- Foundation conditions: carry out adequate soil investigations for foundation design, e.g. trial pits, boreholes.

- Security of site: adequate provision should be made for fencing and security, including a guard house and security lighting.

- Access to the site: ease of access to the site during construction and later for operation and maintenance of the system should be considered. Site preparation may be significant both in cost and time, where improvements such as clearing, access roads and bridges are required. Difficult access often results in poor operation and maintenance due to the reluctance of operators to visit the site. Where floods are expected, access should be assured to provide uninterrupted operation.

- o <u>Availability of electric power</u>: where electricity is required at the site, the power supply should be readily available and of sufficient capacity to operate plant equipment. Where power is unavailable, the cost of installing power should be compared to the cost of using a generator. In that event, the supply of fuel would have to be assured.

- o <u>Availability of site</u>: the site must be readily available for the project, otherwise considerable delays in land acquisition may result in high implementation costs, especially if an alternate distant site has to be acquired subsequently.

- o <u>Staff quarters and stores</u>: adequate space should be provided where staff quarters and materials are to be stored. Quarters should be located adjacent to the plant premises and at sufficient distance away from plant equipment to provide privacy to the occupants. Stores should be located in an area accessible to vehicles and sheltered from public view.

2.4 <u>Design Period</u>

Traditionally, water supply projects have been designed to meet the requirements over a 20-year or 30-year period, (including the design and construction period), but in recent years, financial constraints and economic factors have led to more frequent use of 10 or 15 year horizons, particularly for smaller projects in communities where the growth rate is uncertain. The ongoing ADB rehabilitation project uses the planning horizon of 1995, which means that some projects may have a useful life of 5 years or less.

The best approach is to prepare the future demand curve and from this, evaluate what staging of works may be possible. As a general principle it is recommended that future urban schemes be designed for a 20-year planning horizon in two 10-year stages. That is, for facilities which are suited to staging such as pump sets, tanks and reservoirs, and distribution, only the 1st stage would be provided, whereas facilities which are not suited to staging, such as intakes, transmission mains and dams would be provided for a 20 year period. Minor urban and rural schemes should be designed for a 10 year planning horizon. Staging of such small schemes is not likely to be practicable or economic.

In the case of a long transmission main where the yield of the source is assured to be greater than the design capacity, it may be more economical to initially lay a larger main to carry the full yield rather than to lay a parallel main at a later date. Although an economic evaluation may be justified for mains larger than 100 mm accessibility of the site and ease of construction may be deciding factors in constructing a larger main. If, for instance, the terrain is difficult and leaves little room for future expansion, it would be better to initially lay a larger main to carry the assured yield. (See Section 7.3.2 for details)

When a project is suitable for staging, it is often convenient to meet the requirements of a selected priority area in the first stage, and subsequently serve more areas in the second phase, as development proceeds and as more funding becomes available. In some cases, when more than one source is required to meet the long term demand, the first stage could be tailored to the expected yield of the most economical source.

For larger projects, a master plan detailing the various stages should be developed at feasibility study stage and the implementation programme adjusted to the availability of funds. Individual project components normally should be designed to meet the requirements of the following periods:

	Item	Design Period	Remarks
o	Major dams for impounding reservoirs	50	Height of dam may be considered in two stages
o	Minor dams, weirs	20	Not suited to staging
o	Raw water intakes, infiltration galleries	20	If practicable may be considered in 2 stages
o	Pumpsets, water treatment plant	10	Replacement period
o	Pumphouses	20	Not suited to staging
o	Water treatment structures	20	If economical may be considered in two stages
o	Transmission mains	20	If economical may be considered in two stages
o	Ground service reservoirs	20	If economical may be considered in two stages
o	Water towers	20	Not suited to staging
o	Distribution systems	20	Should be staged
o	Feeder mains to unserved areas	20	Should be staged

3. FORECASTING POPULATION AND DEMAND

3.1 Population Forecasting

This is normally done at feasibility study stage. In order to produce any meaningful forecast of population it is necessary to know the existing population in the project area with some certainty, and this is often a difficult task. It is pointless to present comprehensive projections by mathematical methods with any reliability from a doubtful base population.

Therefore, initially concentrate on collection of data in the project area, for the following: (Ref: Manual P1)

- o population by villages/towns;
- o past trends in population growth/decline;
- o family composition, persons per household;
- o seasonal migrations or other effects;
- o future projections of population growth.

The best sources of information are:

- o The Census of Population and Housing 1981, published by the Department of Census & Statistics, Colombo. This gives data by District and by AGA Division. Data by village or Grama Sevaka Division are not available.
- o The Department of Census and Statistics also publishes Annual Population Reports based on a 10% sample of the population. These Reports would be available in the office of the Government Agent of the District
- o District Reports of the Census of Population and Housing 1981 are available in NWSDB Library for all Districts in the Island. These reports give in the introduction, the average annual growth rate for the District. This rate is important in making population projections and estimating the future requirements for water and sanitation.
- o Records available with the Grama Sevaka/AGA. Data on population of a particular village/hamlet may be obtained from the Grama Sevaka and the AGA office. Again the data would be based on Enumeration of Householders and data collected for Electoral Registers. Even if data is not directly available, the Grama Sevaka may be able to provide adequate data by village/hamlet.
- o Population Survey, see Annex C, Table C.1 for details.

In addition, Table C.4 presents the official Census Department projected annual population growth rates by District for the period 1991-2011 and Table C.5 gives the population growth in Municipal Councils, Urban Councils and former Town Councils over the period 1971-1981.

When assessing the existing or future population to be served in an area, it is important to assess the number or proportion of residents who use and will continue to use non-piped alternative sources. In many areas, particularly rural or semi-urban, this proportion of population may be substantial.

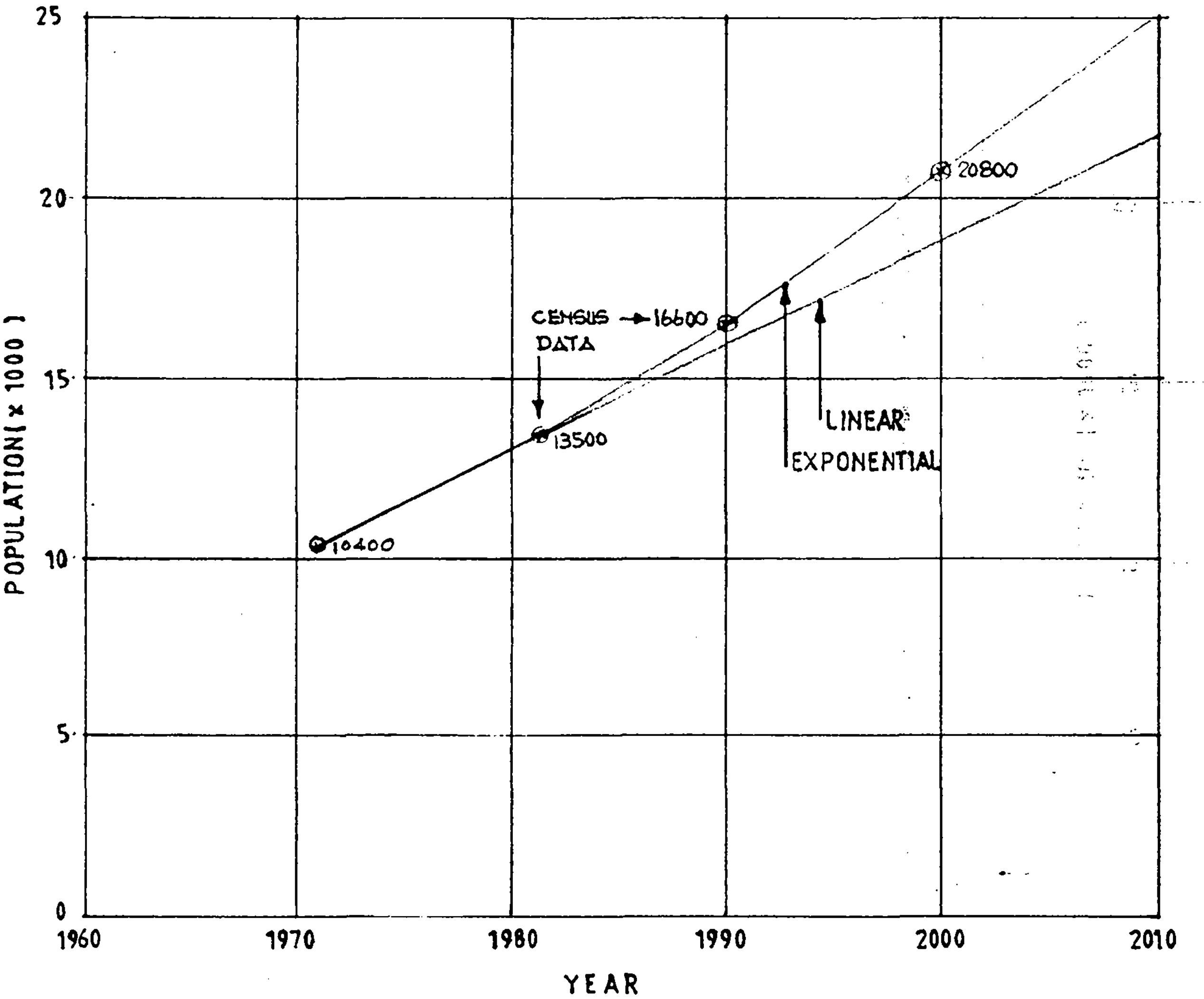
The preferred method for projecting population is by graphical extrapolation of actual population data. Usually, for urban areas, the existing population is known for the past census years 1971 and 1981 (see Table C.5) and earlier if required and these are plotted on natural graph paper.

From the actual known data, the future population can be extrapolated either by linear or exponential projection. The linear projection would be a suitable continuation representing the best-fit straight line extrapolation, and the exponential projection would be based on a suitable growth rate or range of possible growth rates (see Figure 3.1). The use of natural graph paper permits an undistorted view of the trend of the projection which is not possible when log paper is used.

In deciding whether to consider the upper range represented by the exponential projection, or the lower range represented by the linear projection, it is necessary to take into account factors such as development of the urban area (economic, industrial, commercial, social) and migration patterns.

For smaller urban and rural communities, where past growth is unknown, it will usually be adequate to assume the average growth rate for the District, from Table C.4, for an exponential growth projection.

The density and distribution of the future population within the service zones or districts will have to be determined judiciously according to existing and expected plans for development. The availability of master plans by local authorities, NHDA, Town & Country Planning, Mahaweli Authority etc. should be ascertained and, if available, should be incorporated in planning the water supply schemes. This subject is dealt with in more detail in Section 8.13 on distribution system analysis.



For Exponential Projection:

$P_n = P(1 + r)^n$

P = Base Population

P_n = Population at n^{th} year

r = growth rate (annual 2.3 %)

n = no of years

EXAMPLE OF POPULATION PROJECTION

FIGURE 3.1

3.2 Demand Forecasting

This, as for population forecasting, would also normally be done at feasibility study stage and checked or reviewed at final design stage.

A procedure for calculating water demands in each section of distribution, suitable for small communities, is given in Annex C, Tables C.1 to C.3

3.2.1 Domestic Demands

The earlier WHO manual set out guidelines for normal domestic use at house connections and standposts as follows:

(Note that the proportions of population on house connections and standposts have been revised from the earlier manual.

Community	House Connection % Population	House Connection Rate lpcd	Standpost % Population	Standpost Rate lpcd
1. Medium rural (population 1000 - 1500 people)	10-20	-	80-90	45
2. Large rural (population 1500 - 5000 people)	20-40	140	60-80	45
3. Small urban (population 5000 - 10,000 people)	30-50	185	50-70	45
4. Medium urban (population 10,000 - 20,000)	30-60	185	40-70	45
5. Large urban (population over 20,000)	Assess	values	individually	

In general, these values should be used when other more specific data is unavailable. Until recently, reliable data on production, metering and billing was not readily available but with improvements in bulk metering and improved computerization of the billing system, new data is becoming available, which should be reviewed, particularly in the case of rehabilitation projects on existing schemes.

In reviewing actual production/consumption data take into account factors which may restrict current demand such as limited supply hours, poor quality water or use of alternative sources. If possible, check the actual pattern of consumption which will affect the Average Billing Rate for domestic consumption, and relate this to theoretical values based on the standard consumption rates (see example, Annex D).

The standard consumption rates, if used, should be used with caution, taking into account the following:

- o Standpost consumption: the rate of 45 l/d/cap is based on the amount consumers will carry to their nearby residence. Commonly, however, standposts are used for washing and bathing under a running tap, with consequent increased consumption. In most urban schemes, recent standpost meter records should be available for review.

- o Ability to pay: the standard rates of 140 and 185 l/d/cap will result in theoretical average monthly billings of Rs.36 and Rs.74 respectively (see Annex D, Figure D.1) which may well exceed the average consumers' ability to pay, particularly in smaller urban and rural schems. In such cases it must be decided whether these standard per capita demands should be retained as a design "safety factor" or whether they should be reduced in line with peoples' ability to pay (which would also bring down the capital costs of the scheme).

3.2.2 Proportion of House Connection and Standpost Users

This is an important factor because it has such an impact on water demands and revenues. Even when the current proportion is known, for an existing scheme, there is the problem of predicting how the proportion will change in the future, as the supply improves and with it, hopefully the basic standard of living of the community. As far as possible, the percentage of population that will be served by house connections should be decided after careful study in consultation with the local authorities and with due regard to income levels. It is unrealistic to expect that consumers will be able to pay water charges in excess of about 5% of their income.

It is also unrealistic to expect that the poorer section of the community, receiving food stamps, will be able to afford a house connection. When there is no better data available, the tentative values given in the Table above should be used, with extreme caution.

3.2.3 Non-Domestic Demands

According to the standard NWSDB tariff, these are categorised into:

Category

- o 60 Government (government offices, institutions, hospitals)
- o 70 Commercial (private offices, shops, small industries)
- o 71 Tourist hotels
- o 72 Shipping (major ports only)
- o 73 Industries (factories)
- o 80 Schools
- o 81 Religious Institutions and Government approved charities.

For small urban and rural schemes the non-domestic demand is usually negligible or may be taken as a small percentage of the total domestic demand, say 10%, unless specific large consumers can be identified.

However, for medium and larger urban communities it is preferable to assess each category from the basis of existing meter records or on-site surveys.

The table below may be used as a guide:

Non Domestic Use	Litres/day
Hospitals (per bed)	220 - 300
Hotels (per bed)	180 - 700
Boarding schools (per resident)	90 - 140
Restaurants (per seat)	60 - 90
Bus/Railway stations (per user)	15 - 20
Day schools (per pupil	1[illegible] - 30
Offices (per person)	25 - 40
Factories (per person)	20 - 30
Cinemas (per seat)	10 - 15

Where specific data is available for a particular type of use, this of course should be substituted for the values above. In the case of specific factories or other institutions (such as tourist hotels) which are large users of water, special surveys should be undertaken. Other requirements not mentioned above should also be considered individually (such as sewer flushing, street cleaning, public garden watering, animal watering, bowser filling, etc.).

3.2.4 Unaccounted-For Water (UFW)

Note that the above domestic and non domestic demands do not include an allowance for unaccounted-for water, which should be individually assessed for each project. (Note that this differs from the earlier WHO manual which stated on page 6 that unaccounted-for water, wastage and leakage were included - the reason for this exclusion is the potential variation in UFW from 10% to 50% or maybe even higher, depending on system conditions.)

Unaccounted-for water is defined as the difference between treated water production and billed water; it therefore consists of:

- Transmission main leakage;
- Unauthorized tapping of transmission mains;
- Reservoir leakage and overflow;
- Pump station leakage;
- Distribution leakage;
- Unbilled authorized use (eg. at hydrants, filling bowsers, unmetered public standposts, etc.);
- Unauthorized use at illegal connections;
- Service connection leakage;
- Meter under registration; and
- Billing errors.

Note that the waste of water on consumer's premises, after the meter, is not included in UFW.

For a new, tight system, UFW may be as low as 10–15%, otherwise 20–25% should be used for rehabilitated systems or more for specific systems if justified by production and billing data.

Remember that it is in the SWSPB's interests to reduce UFW as much as possible and for systems recording high values, leak detection surveys should be initiated prior to or as a part of any rehabilitation works. (See Section 8.3). UFW costs the SWSDB money to produce (commonly Rs.2-5/m^3) and it derives no revenue. In a scheme with a water supply shortage it represents water that could be sold to consumers; in some cases major capital projects could be deferred by several years merely by reducing UFW: do not underestimate its effect.

3.2.5 Peaking Factors

The maximum demand for water in a system varies from season to season depending on temperature and rainfall, with the highest demands naturally occurring on hot, dry days. In addition, the system demand varies throughout the day with peak system demands normally in the early morning and late afternoon.

The maximum day peaking factor in Sri Lanka is typically 1.1 (smaller urban < 20,000 population) to 1.25 (larger urban > 20,000 population) times the mean annual daily consumption. (In foreign countries with hot summers and a high use for garden watering the factor may be 2.0 or more). Facilities that should be designed for maximum day demand are:

- o Source works, intakes*
- o Transmission mains
- o Supply system pumping stations*
- o Treatment works*
- o Service reservoirs

ie. the complete supply system up to the service reservoirs.

The maximum hourly peaking factor depends on the hours of service per day, and the proportion of consumers with in-house storage tanks, which have the effect of levelling out peaks. Aim for providing a continuous 24-hour service unless there are particular constraints preventing this (i.e. water source or power restrictions). Intermittent supply is neither desirable from a public health point of view nor is it economical. However, it may become necessary to provide intermittent service when the available supply is inadequate to meet the demand. For a 24-hour system where the majority of consumers have storage tanks, use a peak factor of 2.5 (smaller urban) to 2.0 (larger urban > 20 000) times the mean annual daily consumption. (See also Section 8.2).

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* Note: where treatment is required, facilities handling raw water (i.e. source works, low lift pumps and mains, treatment plants) should be sized with a capacity of 1.05 to 1.1 times maximum day to allow for treatment losses.

4. BASIC HYDROLOGY AND WATER RESOURCES ASSESSMENT

4.1 Hydrological Cycle and Water Balance

A knowledge of basic hydrology is necessary in assessment of water resources. The hydrological cycle is shown in Figure 4.1.

Estimation of catchment yield requires an inventory of water inflow and outflow which is defined by a water balance equation for a specified time interval, Δt:

$$I + P - E - Y - O = \Delta S$$

Where P = precipitation in time Δt (mm)

I = Inflow to catchment in time Δt (mm) (surface water, groundwater, piped or canal tranfers and effluent discharges)

E = Evaporation and transpiration loss in time Δt (mm)

Y = Water yield in time Δt (mm)

O = Outflow from catchment in time Δt (mm) (runoff and other abstractions-surface and groundwater)

ΔS = Change in water storage in time Δt (mm) (soil moisture, aquifer contents, reservoir contents) (see Figure 4.2)

Not all the above factors, however, are readily measured and the equation will usually have to be simplified to some extent with evaporation and groundwater effects estimated. A monthly time interval is usual, although short intervals of 1 day or 5 days may be used if adequate data exists.

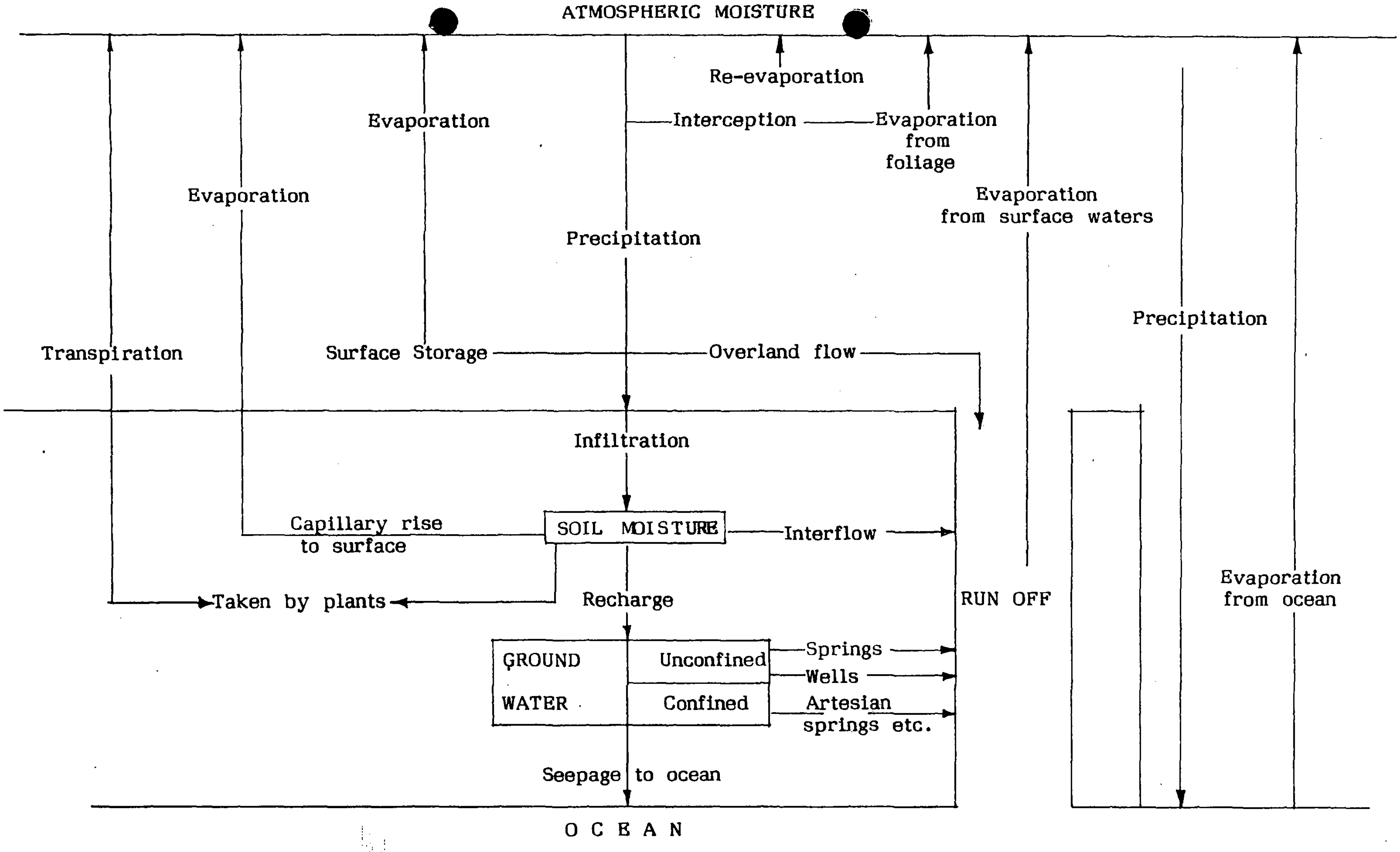
4.2 Catchment Boundaries

These can usually be found accurately from the 1-inch or 1:50,000 scale Survey Department topographic maps and measured by planimeter or counting squares.

Note that the direction of downward flow is always at right angles to the contours, and this defines the catchment boundary (see Figure 4.3)

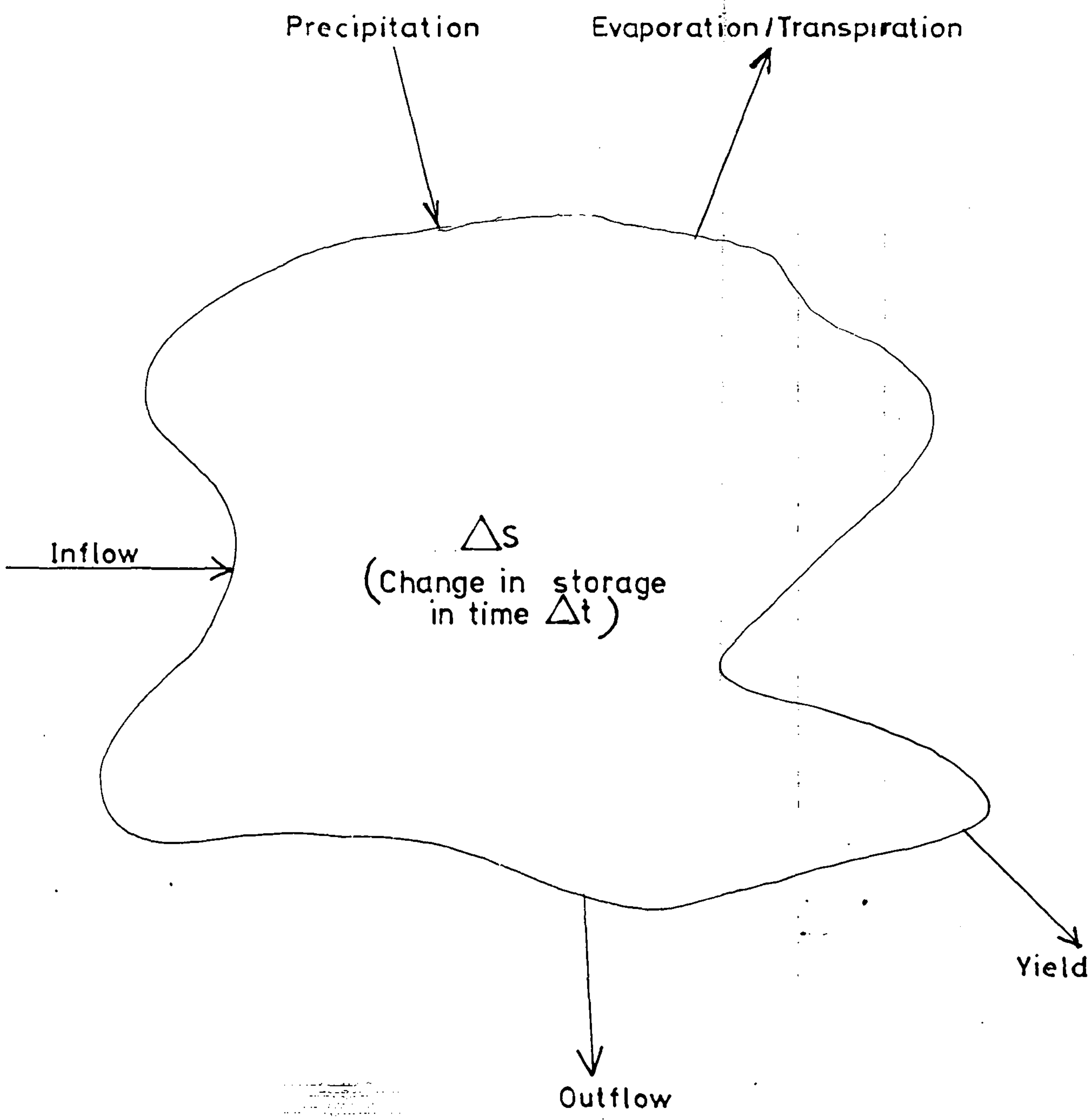
Groundwater catchments are not so readily defined, and will depend on groundwater table contours which may approximate to surface contours, but often do not.

Note that the terms "basin" and "watershed" are also used in place of catchment.



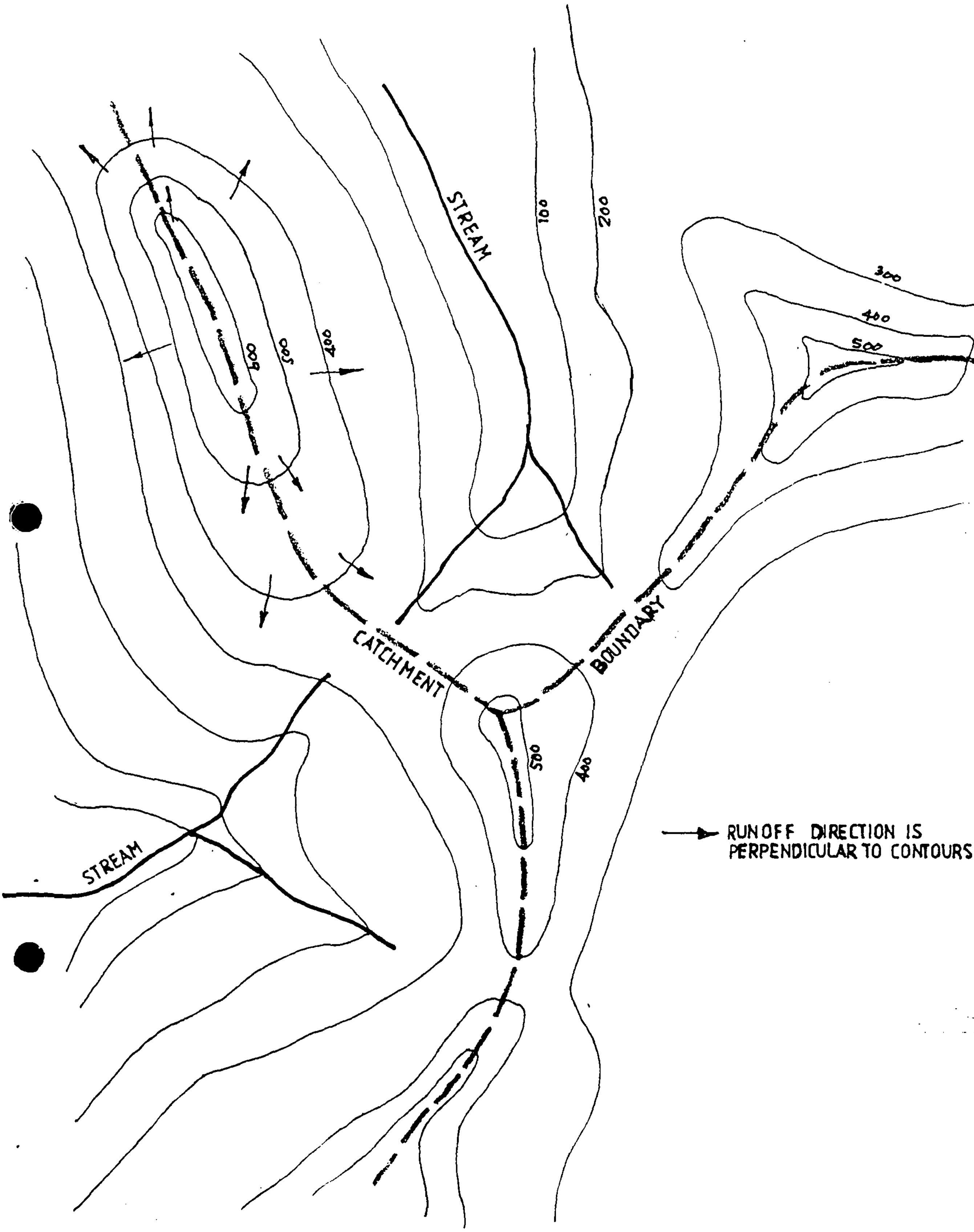
THE HYDROLOGICAL CYCLE

FIGURE 4.1



Catchment Water Balance

FIGURE 4.2



CATCHMENT BOUNDARIES

FIGURE 4.3

4.3 Climatic Zones

The pattern and distribution of rainfall in Sri Lanka is governed by the two monsoons - the South-West monsoon prevailing from about May to September (Yala season) and the North-East monsoon prevailing from about December to February (Maha season). These seasons define the Sri Lankan hydrological year, running from October to September. During the inter-monsoonal periods, rainfall, due to either convective or cyclonic activity, is generally less.

The mean annual rainfall and water deficiency in the three major climatic zones are as follows. (see Figure 4.4)

o	Wet zone:	rainfall 2000 - 5000 mm; no persistent dry session, perennial water surplus.
o	Intermediate zone:	rainfall 1300 - 3500 mm; water deficiency up to 500 mm in February, July - September.
o	Dry zone:	rainfall 1000 - 2000 mm; water deficiency up to 800 mm in February-September; months of zero rainfall common.

(Reference 3,4)

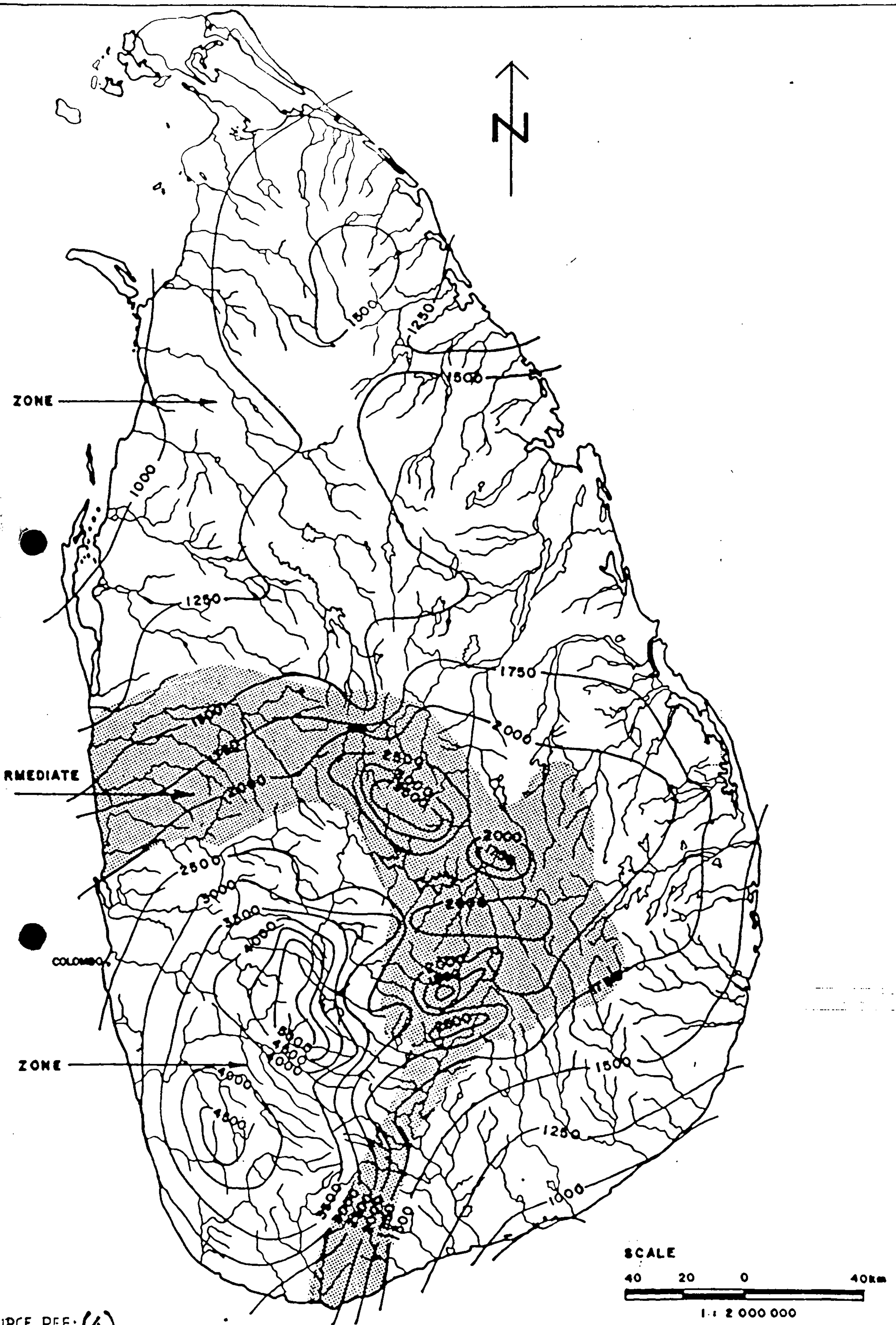
4.4 Availability of Data

The principal data requirements for water resources assessments are for rainfall and streamflow data. The Irrigation Dept. Hydrology Division operates streamgauges and publishes an annual year book. Streamflows are available for 157 locations in 27 out of the 103 river basins. During the hydrological crash programme (1979-84) the hydrometric system was substantially upgraded, with some new stations and improved recording equipment.

Meteorological data is collected by the Dept.of Meteorology at 20 principal stations, and there are a total of about 600 raingauges in existence operated by the Irrigation and Agriculture Departments, estates and municipalities.

An excellent recent compilation of streamflows (m^3/s) ,rainfall (mm) and other hydrometric data has been prepared for the CEB (Ref.4), which includes historic and corrected data up to 1985 for 135 rainfall stations and 157 streamflow stations. Data on evaporation and sediment transport are also given.

Table 4.1 gives basic data on the 103 river basins of Sri Lanka (see Figure 4.5).



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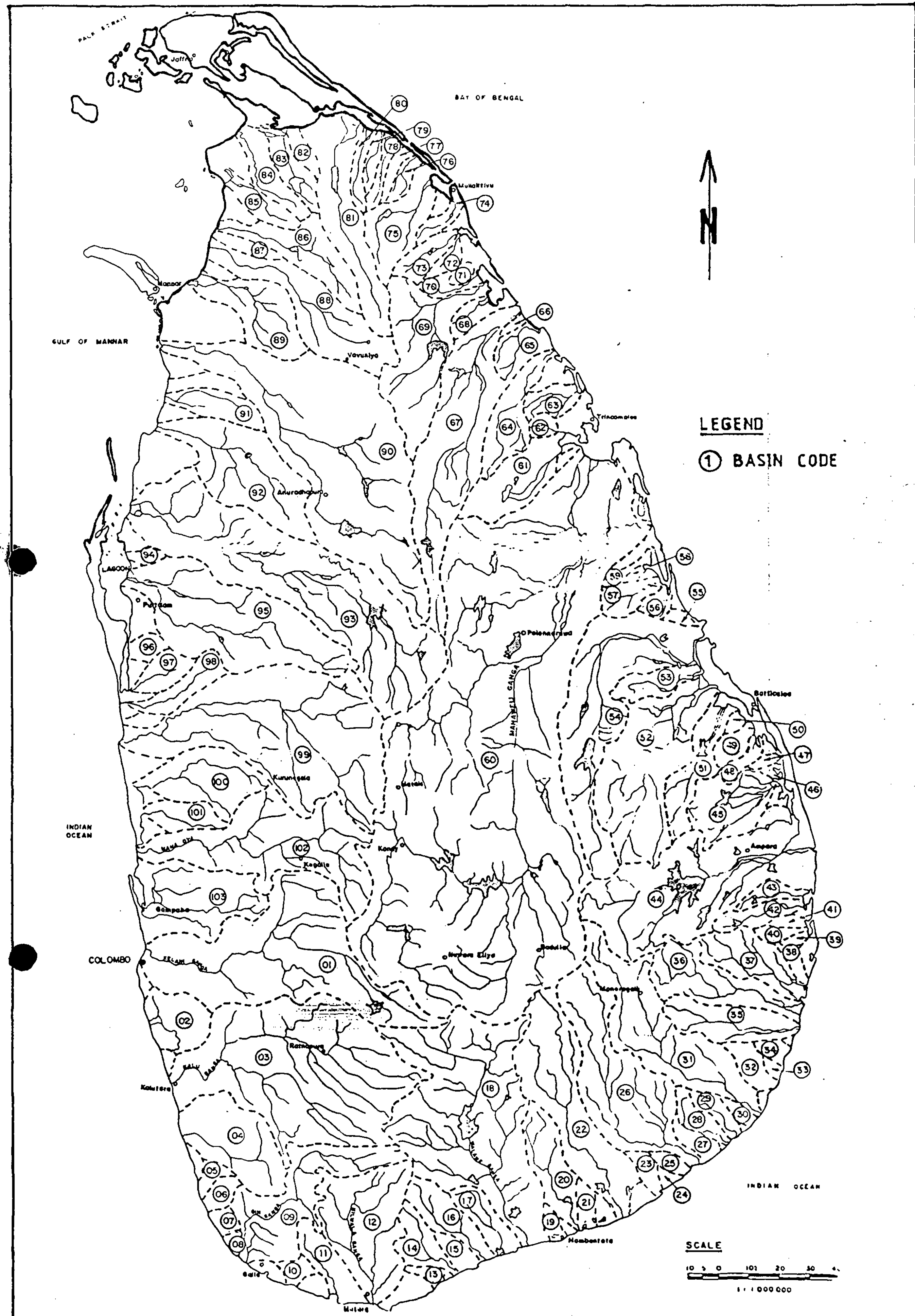
Climatic Zones and Annual Isohyets (mm) FIGURE 4.4



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River Basins of Sri Lanka

FIGURE 4.5

TABLE 4.1 RIVER BASIN CHARACTERISTICS

RIVER BASIN CODE	RIVER BASIN NAME	DRAINAGE AREA (km2)	ANNUAL RAINFALL (mm)	TOTAL RUNOFF (mm)	RUNOFF FACTOR %	UNIT RUNOFF (m3/d/km2)
1	Kelani Ganga	2278	3800	2445	64	6.70
2	Bolgoda Ganga	374	2668	1069	40	2.93
3	Kalu Ganga	2688	4416	2827	64	7.75
4	Bentara Ganga	622	4505	1607	36	4.40
5	Madu Ganga	59	3915	474	12	1.30
6	Madampe Lake	90	2955	477	16	1.31
7	Telwatte Ganga	51	3588	490	14	1.34
8	Ratgama Lake	10	4800	500	10	1.37
9	Gin Ganga	922	3683	2136	58	5.85
10	Koggala Lake	64	2546	437	17	1.20
11	Polwatta Ganga	233	3150	626	20	1.72
12	Nilwala Ganga	960	3207	1468	46	4.02
13	Seenimodera Oya	38	2605	131	5	0.36
14	Kirama Oya	223	2116	192	9	0.53
15	Rekawa Oya	76	2013	144	7	0.39
16	Urubokka Oya	348	1629	241	15	0.66
17	Kachchigala Oya	220	1481	190	13	0.52
18	Walawe Ganga	2442	1973	687	35	1.88
19	Karangan Oya	58	1206	155	13	0.42
20	Malala Oya	399	1305	230	18	0.63
21	Embilikala Oya	59	1254	152	12	0.42
22	Kirindi Oya	1165	1606	261	16	0.72
23	Bambawe Ara	79	1278	139	11	0.38
24	Mahasilawa Oya	13	1692	76	4	0.21
25	Butawa Oya	38	3289	105	3	0.29
26	Menik Ganga	1272	1669	172	10	0.47
27	Katupila Aru	86	1418	116	8	0.32
28	Kuranda Ara	131	1488	122	8	0.33
29	Namadagas Ara	108	1222	111	9	0.30
30	Karambe Ara	46	1130	108	10	0.30
31	Kumbukkan Oya	1218	1736	205	12	0.56
32	Bagura Oya	92	2347	130	6	0.36
33	Girikula Oya	15	2533	66	3	0.18
34	Helawa Ara	51	1431	117	8	0.32
35	Wila Oya	484	1530	221	14	0.61
36	Heda Oya	604	1885	496	26	1.36
37	Karanda Oya	422	1729	386	22	1.06
38	Simena Ara	51	1490	235	16	0.64
39	Tandiadi Aru	22	1818	363	20	0.99
40	Kangikadichi Ara	56	1714	339	20	0.93
41	Rufus Kulam	35	1942	200	10	0.55
42	Pannela Oya	184	1440	288	20	0.79
43	Ambalan Oya	115	1391	286	21	0.78
44	Gal Oya	1792	1768	82	5	0.22
45	Andella Oya	522	1839	408	22	1.12
46	Thumpankeni	9	1555	111	7	0.30
47	Namakada Aru	12	2916	83	3	0.23
48	Mandipattu Aru	100	1410	300	21	0.82
49	Panthanthe Aru	100	1920	300	16	0.82
50	Vett Aru	26	2000	192	10	0.53
51	Unnichchai	346	2303	248	11	0.68

TABLE 4.1 (Contd.) RIVER BASIN CHARACTERISTICS

RIVER BASIN CODE	RIVER BASIN NAME	DRAINAGE AREA (km2)	ANNUAL TOTAL RAINFALL (mm)	ANNUAL TOTAL RUNOFF (mm)	RUNOFF FACTOR %	UNIT RUNOFF (m3/d/km2)
52	Mundeni Aru	1280	2271	284	13	0.78
53	Miyagolla Ela	225	1622	262	16	0.72
54	Maduru Oya	1541	1985	519	26	1.42
55	Pulliyanpota Aru	52	1403	192	14	0.53
56	Kirimechchi Odai	77	1532	207	14	0.57
57	Bodigoda Aru	164	1524	231	15	0.63
58	Mandan Aru	13	3384	76	2	0.21
59	Makarachchi Aru	37	2000	162	8	0.44
60	Mahaweli Ganga	10327	2157	861	40	2.36
61	Kantalai	445	2049	49	2	0.13
62	Palampottu Aru	69	1637	159	10	0.44
63	Panna Oya	143	1601	188	12	0.52
64	Pankulam Aru	382	1780	230	13	0.63
65	Kunchikomban Aru	205	1751	195	11	0.53
66	Palakutti Aru	20	1950	99	5	0.27
67	Yan Oya	1520	1559	271	17	0.74
68	Mee Oya	90	2700	155	6	0.42
69	Ma Oya	1024	1558	36	2	0.10
70	Churian Aru	74	2067	121	6	0.33
71	Chayar Aru	31	2225	64	3	0.18
72	Palladi Aru	61	2098	114	5	0.31
73	Munidel Aru	187	1470	149	10	0.41
74	Kodalikallu Aru	74	1918	135	7	0.37
75	Per Aru	374	1909	181	9	0.50
76	Pali Aru	84	1523	142	9	0.39
77	Maruthapilly Aru	41	2658	146	5	0.40
78	Thoravil Aru	90	1777	166	9	0.45
79	Piramenthal Aru	82	1597	170	11	0.47
80	Nethali Aru	120	1091	183	17	0.50
81	Kanagarayan Aru	896	1493	63	4	0.17
82	Kalawalappu Aru	56	2285	160	7	0.44
83	Akkarayan Aru	192	1963	192	10	0.53
84	Mandekal Aru	297	1346	191	14	0.52
85	Pallavarayan Kad	159	1817	169	9	0.46
86	Pali Aru	451	1789	184	10	0.50
87	Chappi Aru	66	1984	136	7	0.37
88	Parangi Aru	832	1705	168	10	0.46
89	Nay Aru	560	1503	150	10	0.41
90	Aruvi Aru	3246	1515	157	10	0.43
91	Kal Aru	210	1390	109	8	0.30
92	Moderagama Aru	932	1304	218	17	0.60
93	Kala Oya	2772	1461	241	16	0.66
94	Moongil Aru	44	1090	90	8	0.25
95	Mi Oya	1516	1522	110	7	0.30
96	Madurankuli Aru	62	1564	145	9	0.40
97	Kalagamu Oya	151	1668	165	10	0.45
98	Rathambala Oya	215	1627	218	13	0.60
99	Deduru Oya	2616	1648	451	27	1.24
100	Karambala Oya	589	1651	555	34	1.52
101	Ratmal Oya	215	1697	265	16	0.73
102	Maha Oya	1510	2413	834	35	2.28
103	Attanagalu Oya	727	2544	1017	40	2.79

4.5 Measurement of Rainfall

Where rainfall data for a catchment is inadequate it may be necessary to establish a raingauge network. The careful siting of gauges is important to avoid effects of wind (overexposure) or trees (under-exposure) or slope. Standard or recording raingauges may be used.

Refer to any text on hydrology for details of raingauge instruments and siting. Recording raingauges are used for measuring rainfall intensity over a short period, useful for predicting floods.

4.6 Measurement of Flow

4.6.1 Current Meter Gauging

The majority of long period streamflow records are obtained by the manual reading, usually on a daily basis, of a staff gauge set at a control cross-section of the river.

The water level or stage is related to discharge by a stage-discharge curve (or rating curve) for the particular control section which has been previously obtained by a series of current meter gaugings at the site under different flow conditions. (see Figure 4.6).

Over a period of time, or after a major flood, the river bed conditions may change and affect the stage-discharge relationship, which should be recalibrated periodically.

The current meter is usually of propellor or bucket type with a remote counter to measure revolutions over a period of time, which is related to the current velocity.

In shallow water the meter is mounted on a rod and held in position manually, whereas in large rivers, it must be suspended by a system of cables and pulleys, or from a bridge, with the instrument held in position with the help of a tail vane and a heavy weight.

Velocity readings must be taken at a sufficient number of points to compute the average velocity in the stream, yet not take so long as to allow the flow to change significantly. Typically the stream is divided up into about 5-10 vertical sections, and a velocity profile established for each section, from which the overall flow is calculated (see Figure 4.7). It is sometimes sufficient to take only one reading for each section at 0.6 x depth (below surface) or two readings at 0.2 and 0.8 x depth, which should give a good average value for the velocity in the vertical section. The cross section of the stream must be checked during the survey. (See BS 3680 Part 3 Streamflow measurements and Part 3A Velocity - area methods)*.

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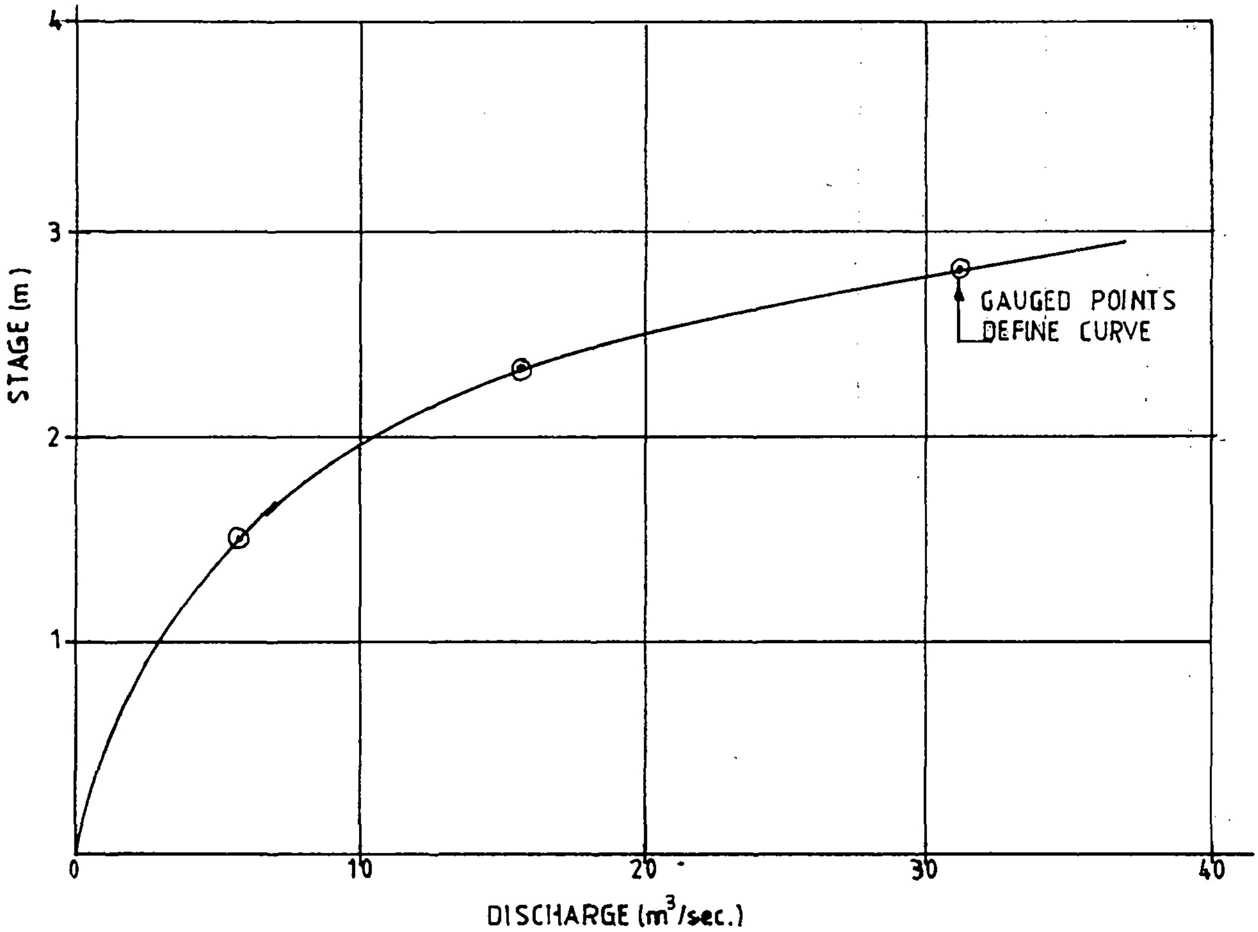
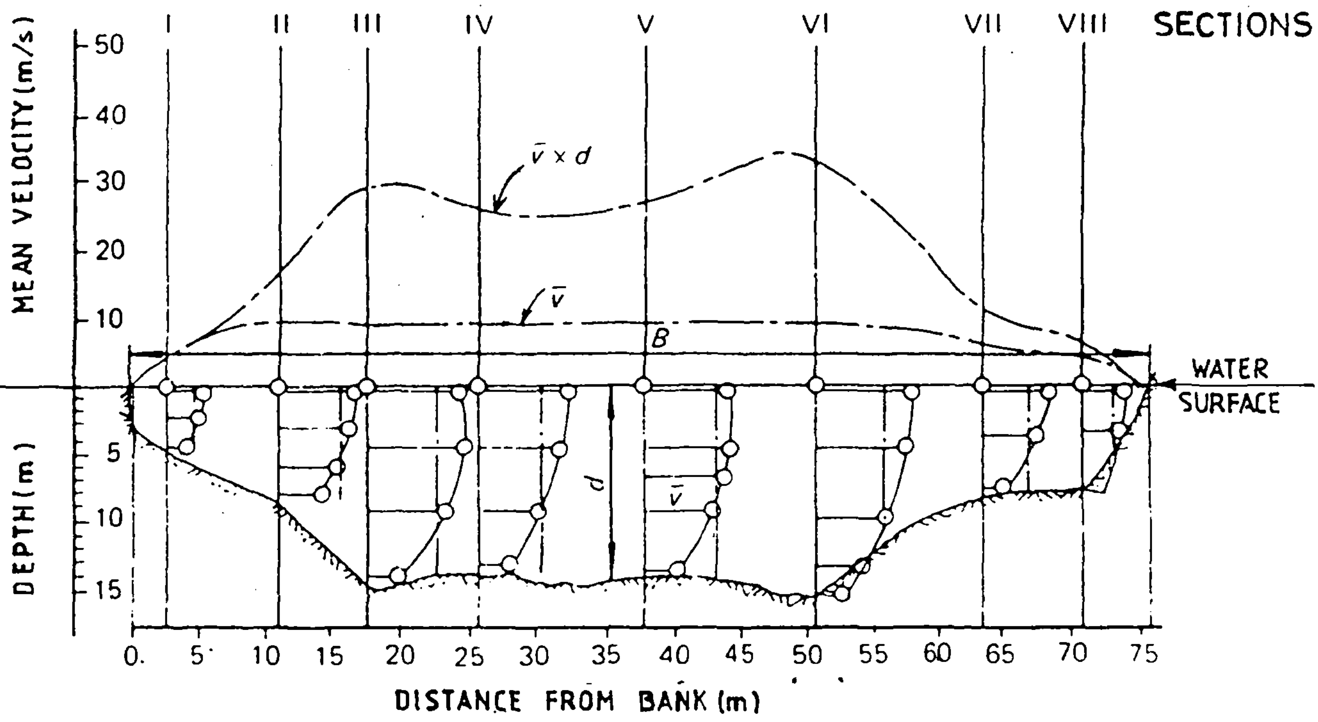
* A list of useful British Standards is given in Annex B



Typical Stage - Discharge Curve

FIGURE 4.6



$\bar{V}$ = mean velocity at each vertical

d = depth at each vertical

Discharge = area under $\bar{v}$d curve

Current Meter Results

SOURCE: BS 3680: PART 3A: 1980

FIGURE 4.7

4.6.2 Slope-Area Method (Manning's Formula)

On a relatively straight reach of channel, Manning's formula may be used by taking readings of the water surface elevation at two points and calculating the surface slope. The cross sections of the channel (giving area and hydraulic radius) must be determined.

Manning's formula is:

$$Q = \frac{1}{n} AR^{2/3} \; s^{1/2} \; (m^3/s)$$

where Q = discharge (m^3/s)
n = roughness coefficient (see Table 4.2)
A = cross sectional area (m^2)
R = hydraulic radius (area/wetted perimeter)
s = slope of energy line (and bed slope for normal depth flow)

Manning's Formula is also commonly used for evaluating normal depth flow in channels of fixed cross-section.

4.6.3 Float Method

Rough measurements of discharge may be made by timing the speed of a floating object. A surface float will travel at about 1.2 times mean velocity, and a float extending to mid depth will travel at about 1.1 times mean velocity.

4.6.4 Sharp Crested Weirs

These may be suitable for flow measurements of small streams or springs, particularly for occasional gaugings where the weir plate can be temporarily fixed in the channel.

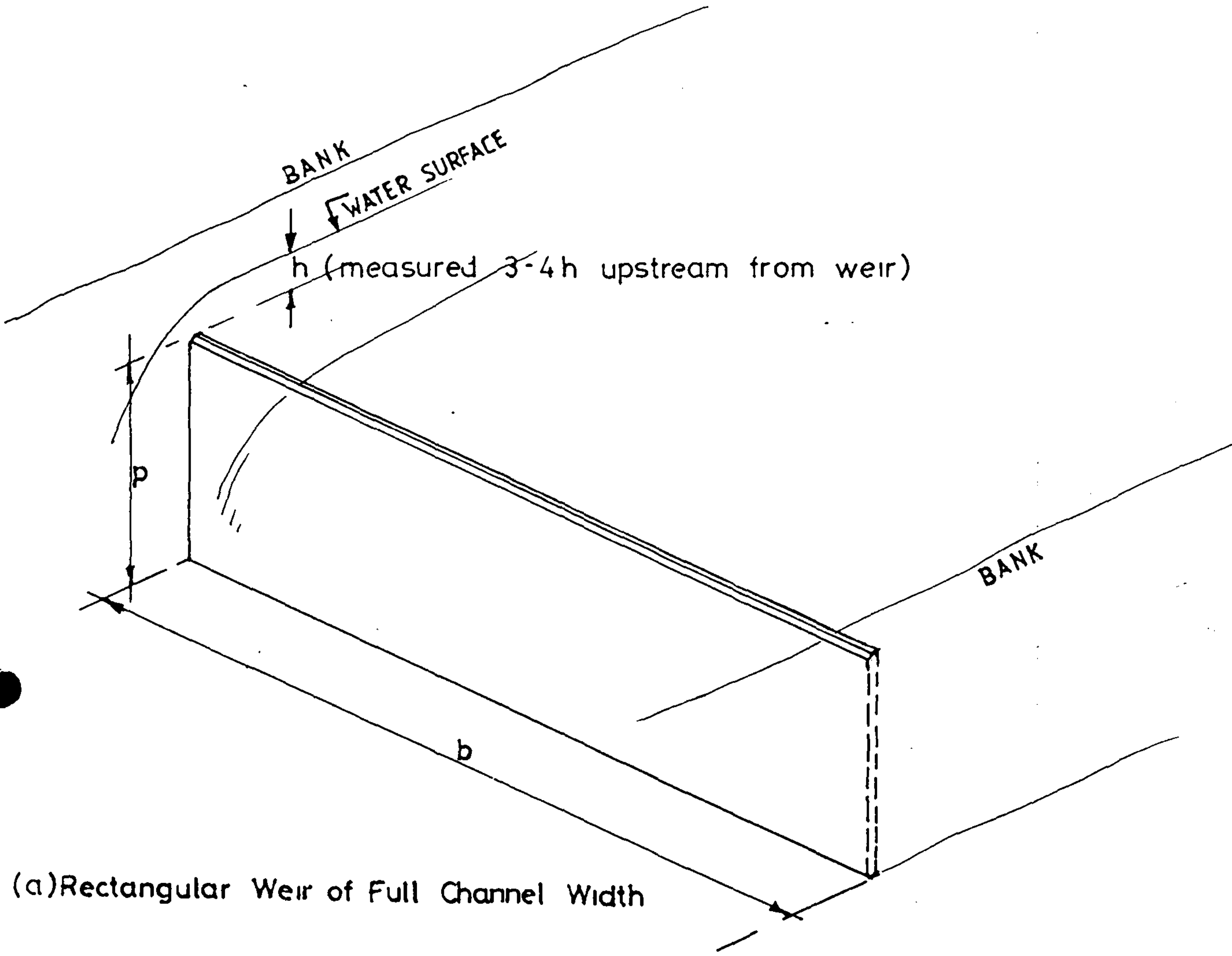
Certain basic criteria should be followed for an accurate measurement (Ref. BS 3680 Part 4A: 1981) since thin plate weir performance is especially sensitive to installation and flow conditions:

- weir should be vertical and at right angles to channel walls;
- intersection between weir plate and channel bed and walls must be watertight and firm;
- weir plate should not distort or bend under maximum flow;
- approach channel flow should be uniform and steady for a distance of at least 10 times width of weir;

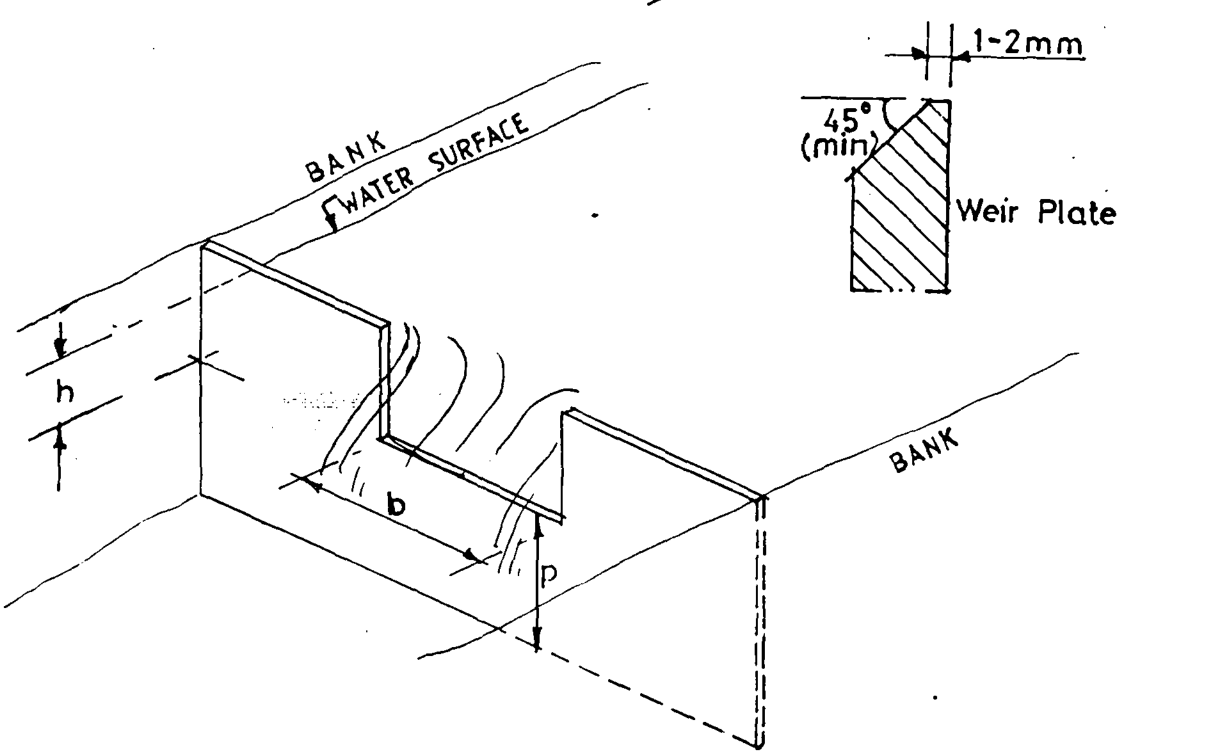
Table 4.2

Values of Manning's Roughness Coefficient

Channel Condition	n
Plastic glass, drawn tubing	0.009
Smooth metal	0.010
Planted timber, asbestos pipe	0.011
Large welded steel pipe with coal tar lining	0.011
Wrought iron, welded steel, canvas	0.012
Ordinary concrete, asphalted cast iron	0.013
Unplaned timber, vitrified clay, glazed brick	0.014
Cast-iron pipe, concrete pipe	0.015
Riveted steel, brick, dressed stone	0.016
Rubble masonry	0.017
Smooth earth	0.018
Firm gravel	0.020
Corrugated metal pipe and flumes	0.023
Natural channels:	
Clean, straight, full stage, no pools	0.029
As above with weeds and stones	0.035
Winding, pools and shallows, clean	0.039
As above at low stages	0.047
Winding, pools and shallows, weeds and stones	0.042
As above, shallow stages, large stones	0.052
Sluggish, weedy, with deep pools	0.065
Very weedy and sluggish	0.112



(a) Rectangular Weir of Full Channel Width



(b) Rectangular Weir with End Contractions

Thin Plate Rectangular Weirs

FIGURE 4.8

- downstream channel shape and size are not important, providing downstream water level is low enough to allow completely free discharge over the weir (fully ventilated nappe or underside);

- head measurement section should be at a distance of 4-5 times maximum head, upstream of weir;

- gauge zero (datum) must be accurately known;

- head on the weir is preferably measured in a gauge well to avoid water level variations due to waves, turbulance or vibration.

The following weir formulae may be used:

<u>Rectangular weir of full channel width (Figure 4.8 (a))</u>

$$Q\ (m^3/s) = \frac{2}{3} . \sqrt{2g} .\ C_D\ b.\ h_e^{3/2}$$

where $C_D = 0.602 + 0.083\ h/P$

$h_e = h + 0.0012$ (m)

P = height (m) of cill above channel floor

b = breadth (m) of weir

g = 9.78 m/s^2 (in Sri Lanka)

<u>Rectangular weir with end contractions (Figure 4.8 (b)</u>

$$Q\ (m/^3s) = 2/3\ \ 2g.\ \ C_D.\ b.\ h^{3/2}$$

where $C_D = 0.616\ (1-0.1h/P)$

(Approximately, $Q = 1.91\ b\ h^{3/2}$)

<u>90% V-notch weir (Figure 4.9)</u>

$$Q\ (m^3/s = \frac{8}{15}\ \ 2g.\ \ C_D.\ h^{5/2}$$

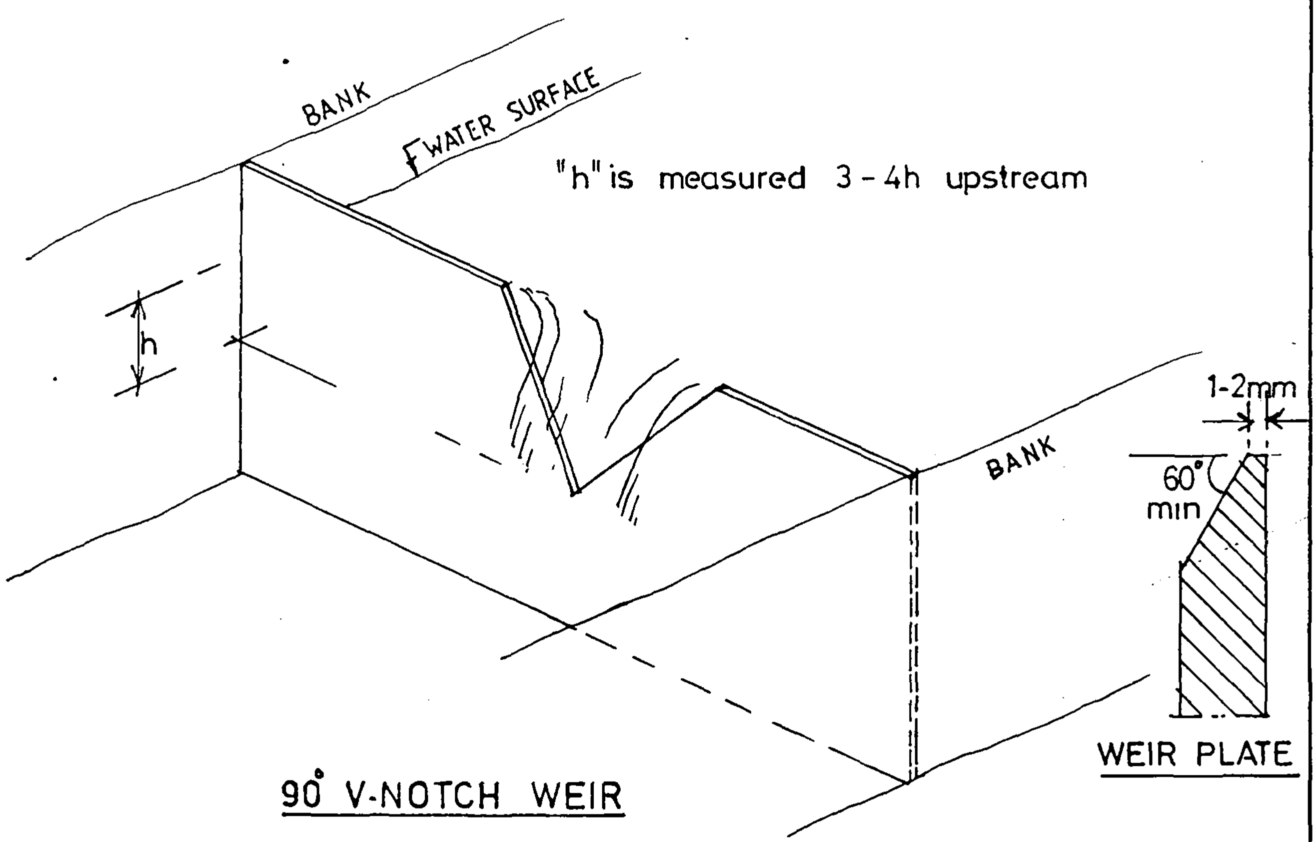
where C_D is found from Figure 4.9

(Approximately $Q = 1.4\ h^{5/2}$)

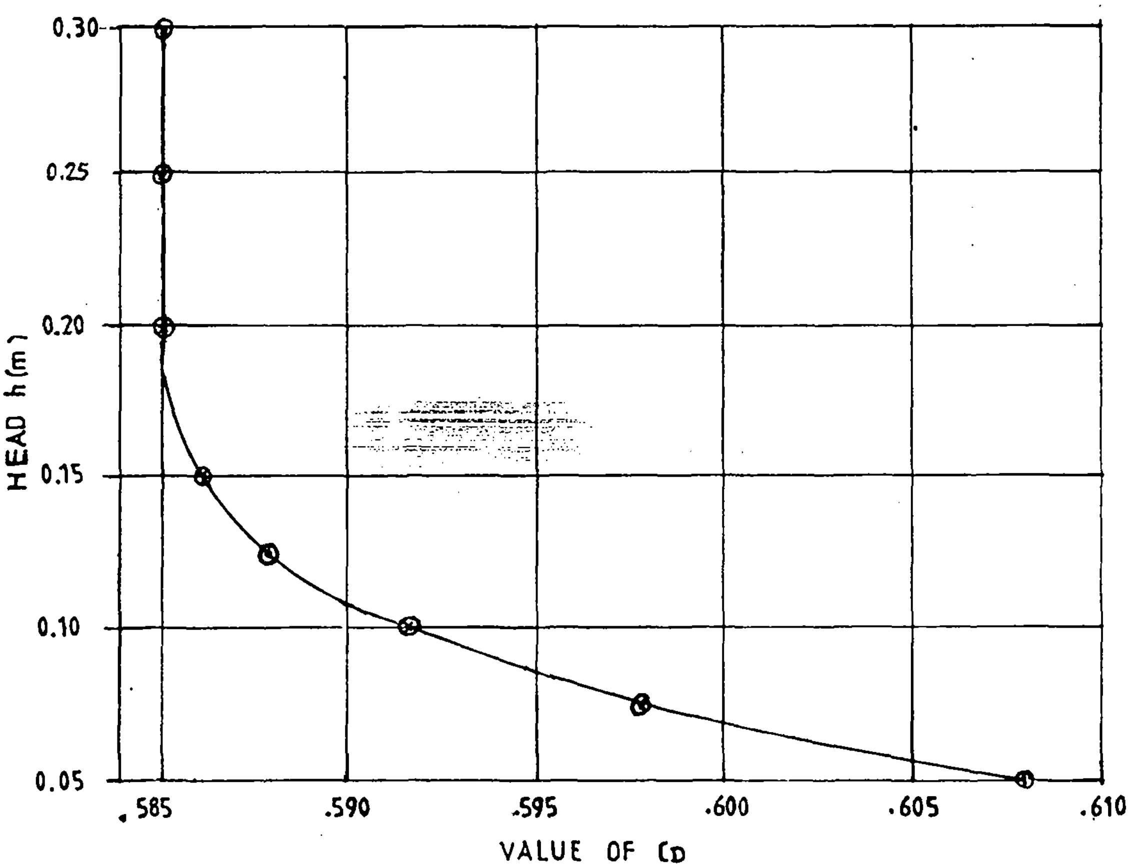
The height of the notch above the channel floor must not be less than 2.5 h and the width of the approach channel must not be less than 5h.

<u>60% V-notch weir</u>

Approximately Q, $(m^3/s) = 0.84\ h^{5/2}$



90° V-NOTCH WEIR



V-NOTCH WEIR

FIGURE 4.9

4.6.5 Broad Crested Weirs (Figure 4.10)

These are suitable for measuring large flows. The formula is:

$$Q\ (m^3/s) = 1.705\ b.\ H^{3/2}$$

where H = head (m) upstream of weir

b = width (m)

(Note that the head over the weir crest will be the critical depth which equals 2/3. H, H being the total energy head upstream of weir).

The downstream conditions also require a free fall away from the weir, but this type of weir may be used with smaller differences in water level than required for thin plate weirs.
(See BS 3680 Part 4E and 4G)

4.6.6 Flumes

Also suitable for measuring large flows, a flume is a construction in a channel such that critical flow is reached in the channel throat. As for a broad-crested weir:

$$Q\ (m^3/s) = 1.705.\ b\ H^{3/2}$$ where H is the total energy height upstream of the throat.

A hydraulic jump is formed downstream of the throat (see Figure 4.11). The formula is adjusted slightly by the geometry of the throat, and whether or not there is a hump in the channel invert. (see BS 3680 Part 4C)

4.6.7 Flow Meters

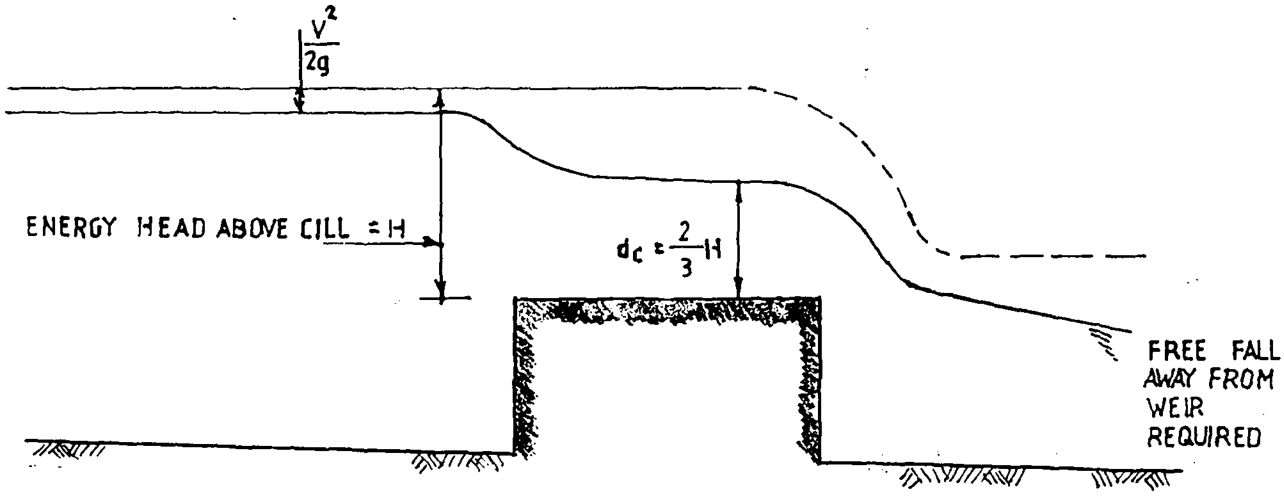
a) Differential Pressure Meters

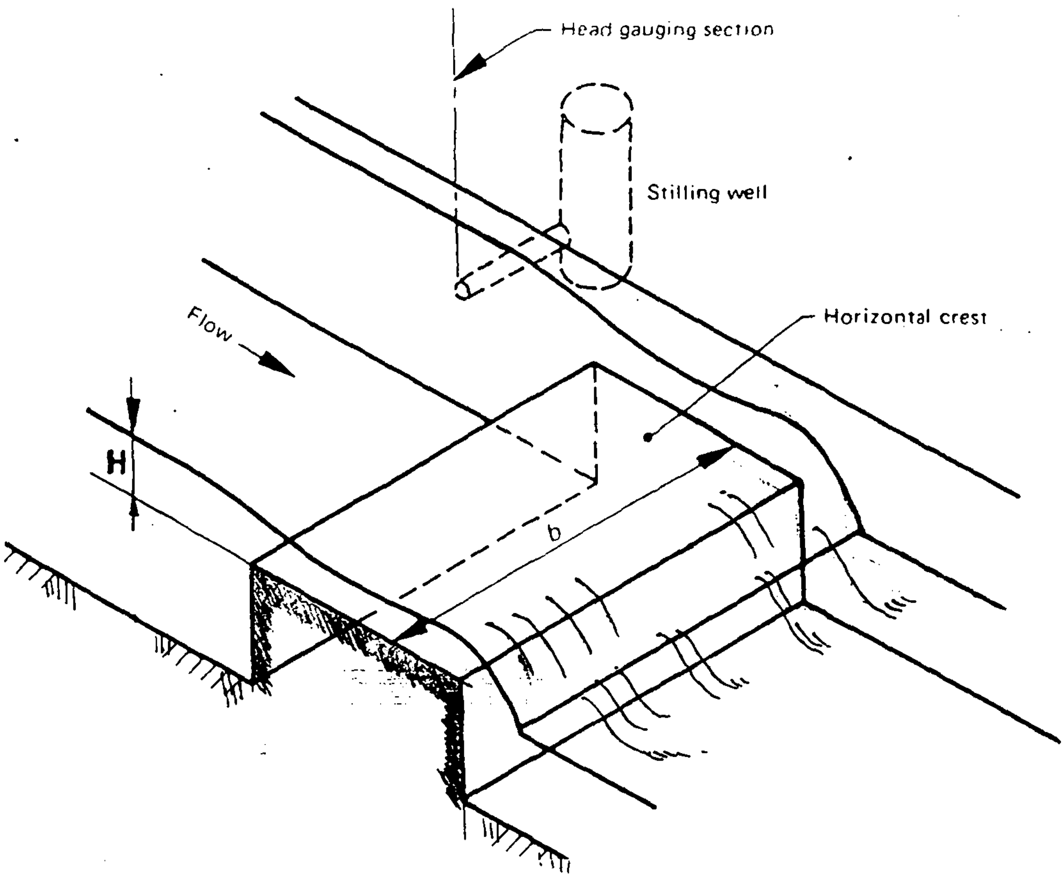
As reported in Manual M5 Mechanical, Electrical and Instrumentation Aspects of Design, it is reported that most Dall tube and Venturi type differential pressure flowmeters in existing plants are not functioning due to various problems.

For this reason it is recommended that flumes or weirs be used instead, except where efficient O&M can be guaranteed.

The main types available are:

- Orifice meter – takes up less space and has a greater head loss than venturi meter, available for medium to large size pipes.

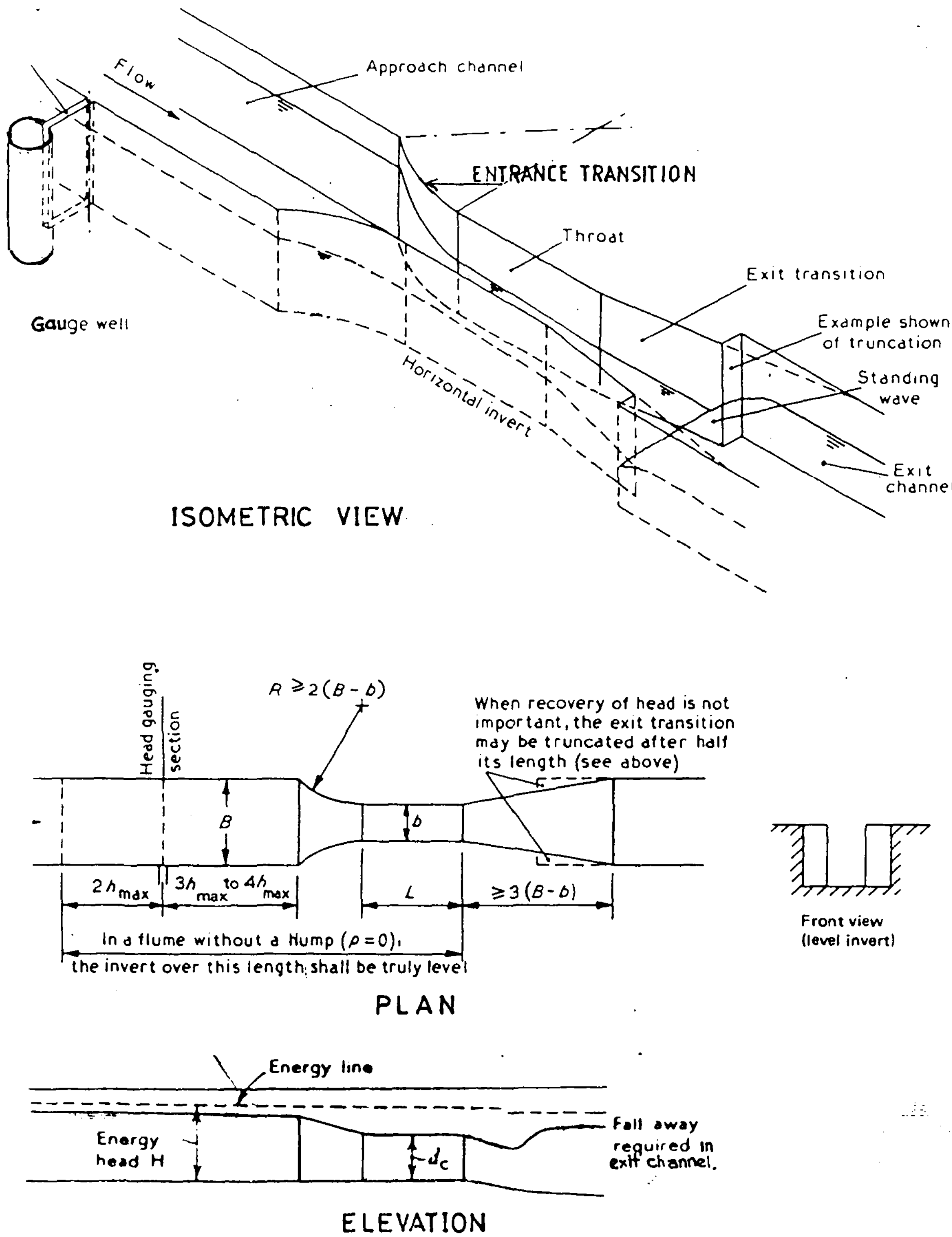




Broad Crested Weir

REF. BS 3680: PART 4E: 1980

FIGURE 4.10



Flumes

REF : B.S 3680 PART 4C

FIGURE 4.11

° Venturi meter - widely used for large pipes.

° Dall tube meter - this is a special compact, low-head-loss version of a venturi meter.

For full details on these types of meter see Ref. 5 .

b) Inferential Meters

Helical vane (propellor) type meters are available up to about 500 mm diameter. For economy, they can be used in a larger pipe size than the meter, providing head loss and maximum flow rate restrictions are not a problem. They should not be used for raw water or where there may be objects in the pipeline which might damage the vanes.

4.7 Droughts

It is important in predicting catchment yield to assess the potential severity and frequency of droughts and the best way to do this is to analyse past rainfall and runoff records. Usually; the available rainfall data will be over a longer period than any flow data, and therefore an analysis of past droughts can conveniently be made from rainfall data.

A rainfall frequency analysis involves compiling an ordered list of annual total rainfalls, R, starting with the dryest year on record, of order m = 1 up to the wettest, of order say m = 75 (if there are N = 75 years of record). The probability of an event equal to or less than R is then:

$$P_R = \frac{m}{N + 1} \times 100\%$$

For the dryest year

$$P_R = \frac{1}{75 + 1} \times 100\% = 1.32\%$$

having a return period of

$$T = \frac{1}{P} \times 100 = 75 \text{ years}$$

The return period is the time, on average, which elapses between two events which equal, exceed, or are less than a particular level.

Figure 4.12 shows a frequency analysis for annual rainfall at Diyatalawa. This may be done for different periods (i. 2 consecutive years or 6 consecutive months) and a set of curves drawn to show the frequency of drought events of each period. From the curve it may be seen that a 1 in 20 year drought would occur with an annual rainfall of 1160 mm (compared with a long term average of 1653.4 mm). A sample of Gumbel probability paper, useful for this purpose, is given in Annex E.

Sometimes it is helpful to try and correlate rainfall with runoff, if, for example, a long period rainfall station is located in an adjacent catchment to a shorter period runoff station where it is required to predict drought flows. Plot annual or monthly rainfall totals against average flows for the same periods. Sometimes a correlation will exist, sometimes not. Figure 4.13 shows a reasonable degree of correlation.

The frequency analysis described above for rainfall may also be carried out for flow data. Estimation of yield is covered in Section 4.11.

4.8 Floods

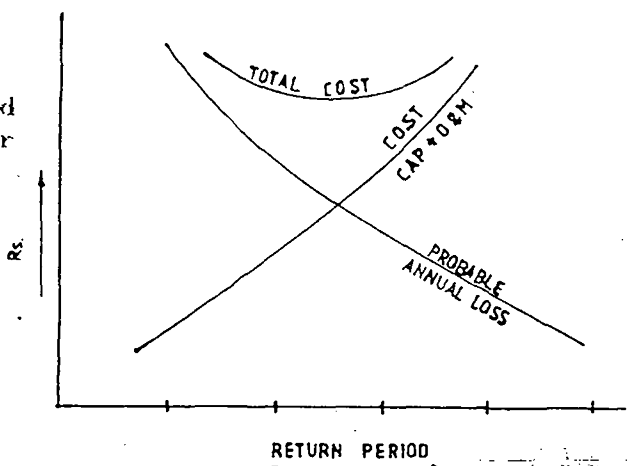
4.8.1. Risk

The question arises as to how to decide which return period or frequency of floods to use in design.

The objective should be to minimise total annual costs including capital and operating costs and the probable losses each year.

Plot (1) Annual capital and operating cost based on amortization over design life, at prevailing rate of interest, and

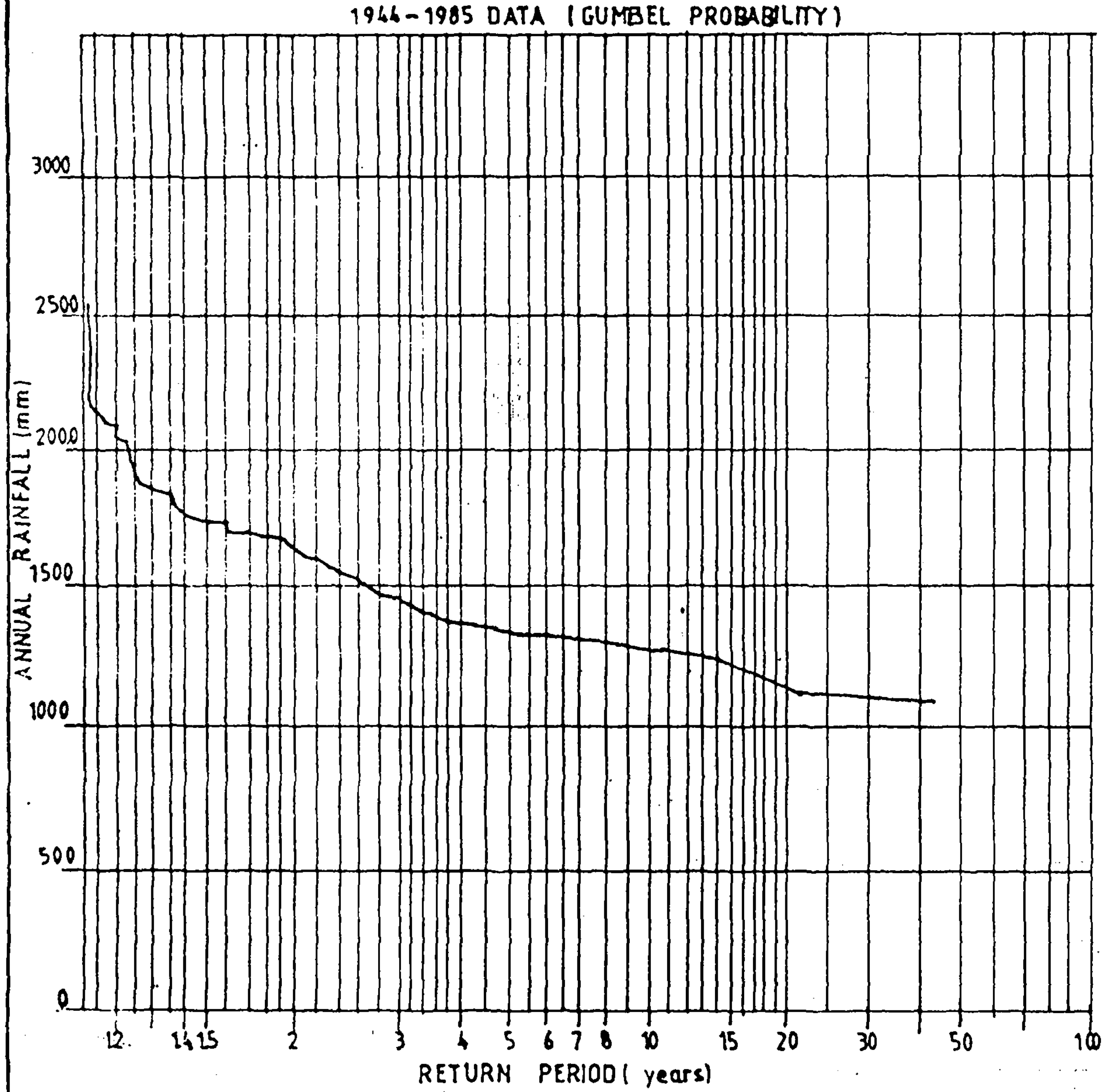
(2) Probable cost of losses each year



= (probability of a loss in 1 year) x (damages likely incurred as a result of failure of structure)

4.8.2 Runoff

A knowledge of flood volumes and frequency is necessary for estimating high water levels, for spillway design, and for storm drainage design.



Rainfall Frequency Analysis
Diyatalawa

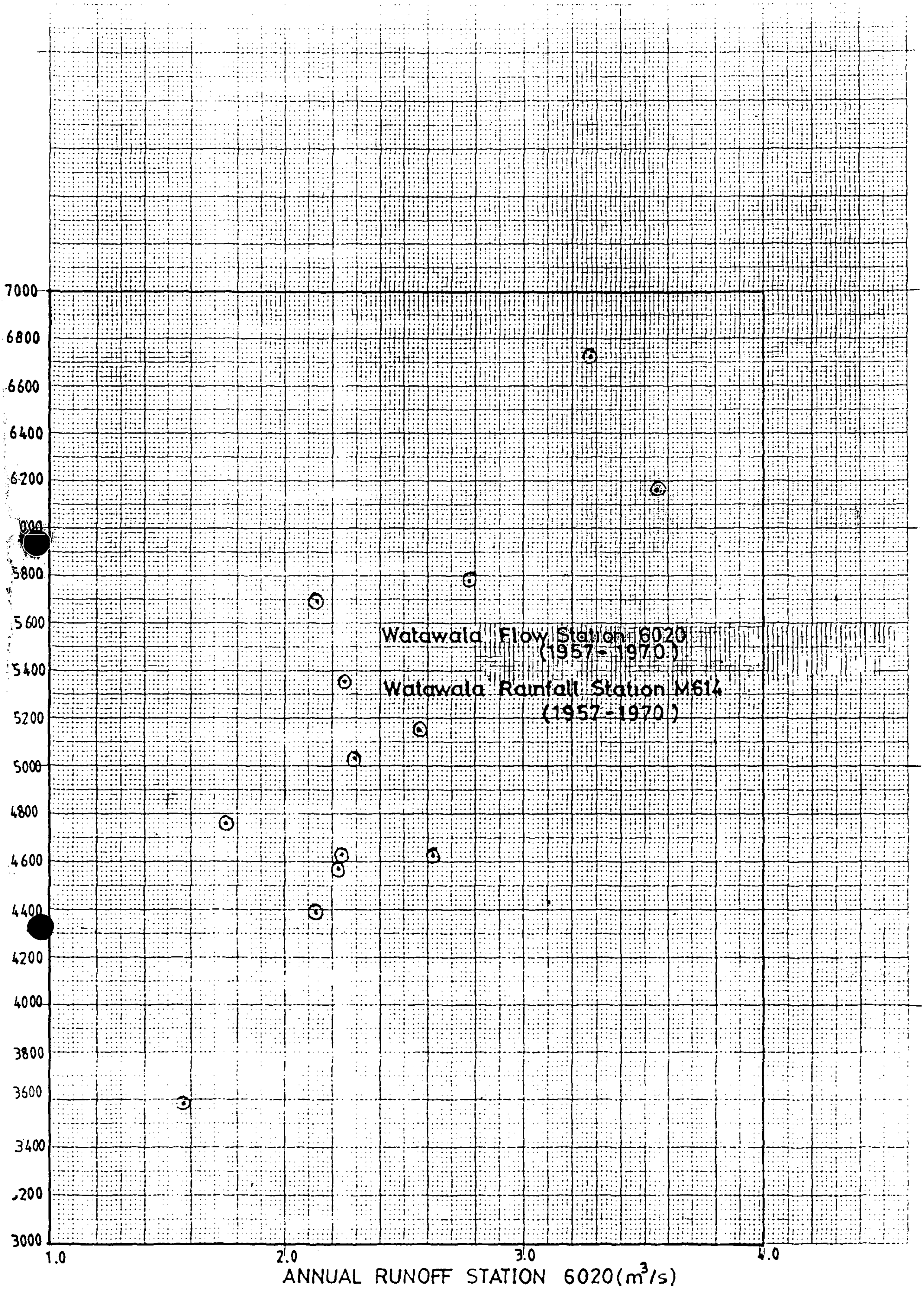
DIYATALAWA - STATION CODE M 107
ELEVATION 1256 m

RAINFALL (mm)

YEAR	YEARLY TOTALS	ORDER (m)	PROBABILITY (%)	RETURN PERIOD (Years)
1944/45	1,875.9	33	79	1.27
1945/46	2,138.9	38	90	1.11
1946/47	2,045.6	34	81	1.24
1947/48	1,167.7	2	5	21
1948/49	1,797.8	31	74	1.35
1949/50	1,295.0	5	12	8.4
1950/51	1,758.3	29	69	1.45
1951/52	2,073.0	37	88	1.14
1952/53	1,650.4	22	52	1.91
1953/54	1,697.7	25	60	1.68
1954/55	2,182.3	39	93	1.08
1955/56	1,381.0	11	26	3.8
1956/57	1,449.1	14	33	3
1957/58	2,501.6	41	98	1.02
1958/59	1,654.3	21	50	2
1959/60	1,819.4	32	76	1.3
1960/61	1,608.7	19	45	2.2
1961/62	1,671.6	23	55	1.83
1962/63	1,538.2	17	40	2.47
1963/64	1,790.5	30	71	1.4
1964/65	1,373.9	10	24	4.2
1965/66	1,673.3	24	57	1.75
1966/67	1,317.4	8	19	5.25
1967/68	1,206.6	3	7	14
1968/69	1,531.1	16	33	2.6
1969/70	2,057.1	35	83	1.2
1970/71	2,069.2	36	86	1.2
1971/72	1,460.1	15	36	2.8
1972/73	1,419.6	12	29	3.5
1973/74	1,715.8	27	64	1.6
1974/75	1,748.0	28	67	1.5
1975/76	1,327.4	9	21	4.7
1976/77	1,552.8	18	43	2.3
1977/78	1,702.3	26	62	1.6
1978/79	1,314.7	7	17	6
1979/80	1,424.0	13	31	3.2
1980/81	1,618.2	20	48	2.1
1981/82	1,259.4	4	9	10.5
1982/83	1,098.1	1	24	42
1983/84	2,311.0	40	95	1.05
1984/85	1,304.4	6	14	7

MEAN	1,653.4
MAX	2,501.6
MIN	1,098.1
STDV	329.7

FIGURE 4.12



Rainfall-Runoff Correlation

FIGURE 4.13

Flood flows may be derived from rainfall analyses and development of unit hydrographs, or from an analysis of actual storm hydrographs. A hydrograph is a plot of discharge against time.

Rainfall analysis involves developing rainfall intensity - duration - frequency curves, and these are available for 6 hydrological zones of Sri Lanka (see Annex F).

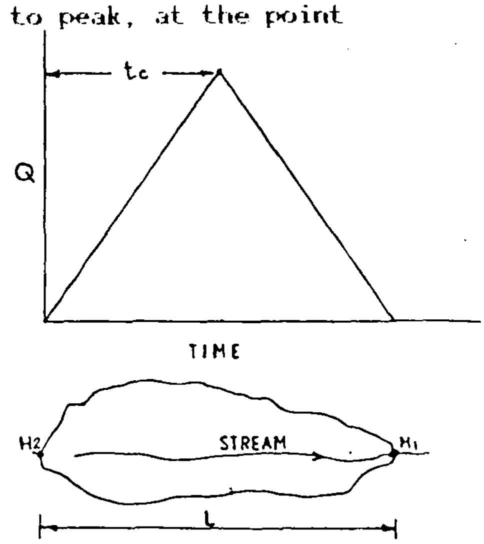
During a storm, the pattern of the resulting stream flow (the hydrograph) depends on numerous factors such as the intensity and duration of rainfall, and catchment characteristics such as area, slope, soil type and vegetation cover. As the rainfall reaches the ground, some infiltrates into the ground to become groundwater, some is intercepted by vegetation, or is retained in surface puddles and the remainder runs off the surface, collecting into streams. At the start of a storm, infiltration, interception and surface retention will be high, reducing as the soil becomes wetter and surface depressions are filled. Hence:

Runoff = Precipitation - (Infiltration + Evaporation + Surface Storage + Losses)

Surface runoff only occurs when the rainfall rate is greater than the infiltration rate.

High rates of infiltration in the catchment will usually result in streams having a substantial sustained flow (base flow) all year from groundwater seepage, with lower flood peaks and slow response, whereas an impervious catchment would result in rapid runoff with fast response and high flood peaks. Due to these catchment characteristics, a stream will usually have a characteristic flood hydrograph, with a similar recession curve for different flood events.

Another useful catchment characteristic is the time of concentration, tc, which is the time in minutes after the start of rainfall for the runoff to peak, at the point under consideration.



tc can be regarded as a function of length and slope of catchment, using the following relationship:

$$t_c = 0.0195\ k^{0.77}\ \text{mins}$$

where $k = \sqrt{\dfrac{L^3}{H}}$

and $H_{(m)}$ $= H_2 - H_1$, the difference in catchment elevation over the distance L (m) to the most distant contributing point.

Another method (Ref.6) uses the average velocity of water across the catchment as follows:

$$t_c = \frac{L}{60\ V + T}$$

where L = length of longest watercourse (m)

V = velocity (m/s)

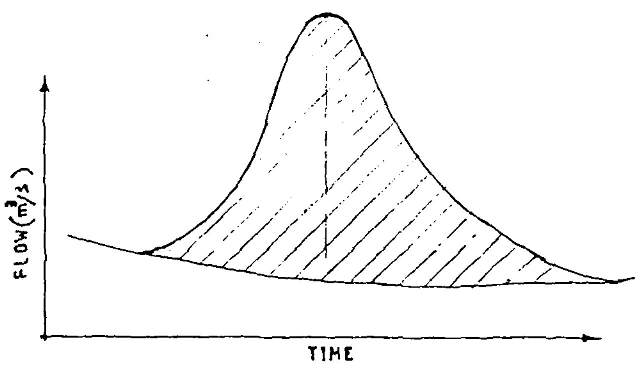
and T is an allowance for inlet time, the time for water to reach the watercourse, usually 2-20 minutes. The estimated velocity depends on slope as follows:

Average Slope of Stream (%)	Average Velocity (m/s)
< 1	0.5
1 - 2	0.6
2 - 4	0.9
4 - 6	1.2
> 6	1.5

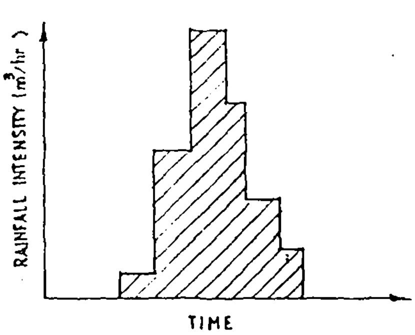
If the storm is of longer duration than the time of concentration the peak, Q will occur at the lapse of t_c and continue to remain at this value until the end of the storm, if uniform intensity rainfall is assumed.

4.8.3 Hydrographs

The runoff volume may be estimated from the area under the hydrograph. If the rainfall pattern was recorded for the storm, the volume of rainfall on the catchment could be compared with the runoff volume.

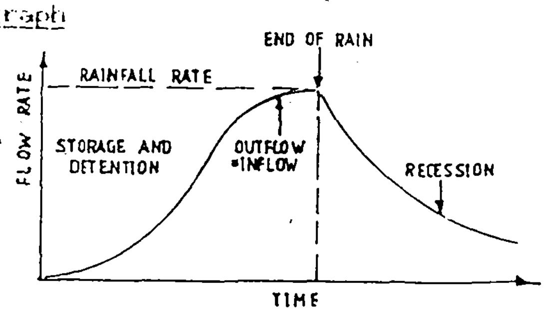


Hydrographs may be estimated from rainfall in the following ways:



a) Theoretical overland flow hydrograph

Assume a constant intensity rainfall over an impervious catchment. Runoff would increase until an equilibrium point has reached, when outflow = inflow.



When the rain stopped, flow would immediately reduce into a recession curve representing the runoff from water temporarily in storage. In this case there are no losses, and rainfall equals runoff.

b) Unit hydrograph

These are unique to a particular catchment and represent a hydrograph of a unit of direct rainfall from a storm of specified intensity. They are derived from analyses of recorded hydrographs and rainfalls in the catchment, and used to predict future runoff.

c) Empirical flood formula

The most common is known as the rational formula, see Section 4.8.4.

d) Computer models

Various software is available for generating hydrographs on the basis of catchment characteristics and rainfall data.

4.8.4 Rational Formula

This is a method, for small catchments up to about 50 km^2, of estimating peak floods. For larger catchments, the rational formula gives too high a value. The formula is:

$$Q\ m^3/s = \frac{CIA}{360}$$

where I = rainfall intensity (mm/hr)

A = area of catchment (ha)

C = constant depending on catchment characteristics

The constant C is a runoff coefficient which depends on the relative imperviousness of the catchment and may vary from as low as 0.01 for flat forest lands to 0.95 for water-tight roof surfaces. The type of catchments encountered in reservoir design are mostly jungle with partial development, and the following C values are appropriate.

Average Catchment Slope %	C
< 2	0.3
2 - 4	0.4
> 4	0.5

Values for other types of surface are given in Table 4.3. Different references list slightly coefficients and judgement must obviously be exercised when selecting runoff coefficients and rates of rainfall. Different values can be assumed, to assess the sensitivity of the result. An example of the use of the Rational Formula is given in Annex G.

4.8.5 Historic Peak Runoffs

An analysis has recently been made of annual maximum discharges (Ref.4), and although the data are not particularly reliable, some interesting results can be seen:

- Three extreme storm events stand out, each occuring over a wide area covering several basins:

  - 15-17 August 1947
  - 24-26 December 1957
  - 24-25 November 1978

- The 1957 flood produced all-time maxima at many locations in the dry/intermediate zone as well as annual maxima in the wet zone.

- Peak flows at lower locations occurred up to 1 day after upper catchment peaks.

- A flood frequency analysis indicates 1 in 10,000 year specific flood discharges of up to 10 $m^3/s/km^2$, reducing to about 4 $m^3/s/km^2$ for catchments of 4000 km^2.

- Specific flood discharge reduces with increasing catchment size.

4.9 Reservoir Sedimentation

Sediment accumulations in reservoirs reduce the space available for storage of water, and thereby reduce the effectiveness of the reservoirs for regulation of flows. When estimating the required volume for a reservoir, it is customary to allow extra volume equal to fifty years' accumulation of sediment.

Few reliable estimates of reservoir sedimentation rates in Sri Lanka have been made, but studies in the upper Mahaweli catchment have indicated an annual sediment yield in the order of 300 m^3/km^2. Sediment rates will differ according to catchment characteristics including geology and landuse.

TABLE 4.3

TYPICAL RUNOFF COEFFICIENTS

TYPE OF AREA	SLOPE		
	Up to 2%	2% to 10%	Over 10%
Roofs and pavements	.90	.90	.90
City business areas	.80	.85	.85
Suburban residential	.45	.50	.55
Hi-density residential	.60	.65	.70
Grassland and meadows	.25	.30	.30
Bare earth	.60	.65	.70
Cultivated land - clay loam	.50	.55	.60
- sandy	.25	.30	.35
Forests and woods	.10	.15	.20
TYPE OF SURFACE			
Watertight roof surfaces	.75 to .95		
Asphalt runway pavements	.80 to .95		
Concrete runway pavements	.70 to .90		
Gravel or macadam pavements	.35 to .70		
Impervious soils (heavy)	.40 to .65		
Impervious soils, with turf	.30 to .55		
Slightly pervious soils	.15 to .40		
Slightly pervious soils, with turf	.10 to .30		
Moderately pervious soils,	.05 to .20		
Moderately pervious soils, with turf	.00 to .10		

Reservoir sounding surveys, if done accurately, can provide valuable information on actual sedimentation rates.

4.10 Evaporation and Seepage Losses

The creation of a new lake behind a dam results in losses due to evaporation and seepage.

Evaporation from reservoirs where the area was heavily vegetated with deep-rooted trees or plants, and from reservoirs in areas with no prolonged dry season, are little if any greater than the evaporation and evapotranspiration from the area prior to construction of the reservoir. Flow losses due to evaporation may be significant only for sparsely-vegetated reservoirs and during the dry season.

Monthly evaporation data is available for about 55 stations throughout Sri Lanka, and values differ widely depending on climate and elevation from 1000 mm to more than 2000 mm annually. Monthly mean values for Colombo and Peradeniya are given in the table below:

	Colombo	Peradeniya
January	165.5	99.0
February	156.6	91.5
March	152.7	107.2
April	178.9	127.7
May	187.7	150.0
June	220.3	159.8
July	176.1	123.0
August	150.2	130.9
September	169.8	107.8
October	168.2	99.0
November	180.8	108.9
December	183.4	104.1
Total	2090.5 mm	1408.9 mm
Elevation	5m	488 m

In general, reservoir seepage losses can be significant depending on dam foundation conditions and construction details. Any sites located on deep pervious sediments could have substantial seepage losses, which should be accounted for during detailed feasibility studies.

4.11 Estimation of Yield

Where a reservoir is used to store water in surplus runoff periods for use during dry periods it is necessary to determine the firm yield of the reservoir which is the supply available during a historic drought or a drought of specified severity, e.g. 1 in 10 year drought, or 1 in 50 year drought.

The starting point is the determination of the area-elevation curve and volume - elevation curves for the proposed reservoir site (see Figure 4.14). Special topographic surveys may be required to define the reservoir area and volume with sufficient accuracy. Normally a scale of 1:5000 with 5 m contours should be adequate. The water demand from the reservoir should be known. An allowance must be made in the bottom of the reservoir for "dead" storage below the lowest water intake level, to provide space for sediment accumulation. Live storage is the usable storage above this level.

Allowances have to be made for overflows (spills), downstream releases and reservoir losses such as evaporation and seepage, thus:

Gross Yield = Demand (Net Yield) + Downstream Releases + Overflows
+ Reservoir Losses

Most projects allow for dry season releases from the dam into the downstream river channel to meet the prior rights of users downstream. (See Section 4.12.2)

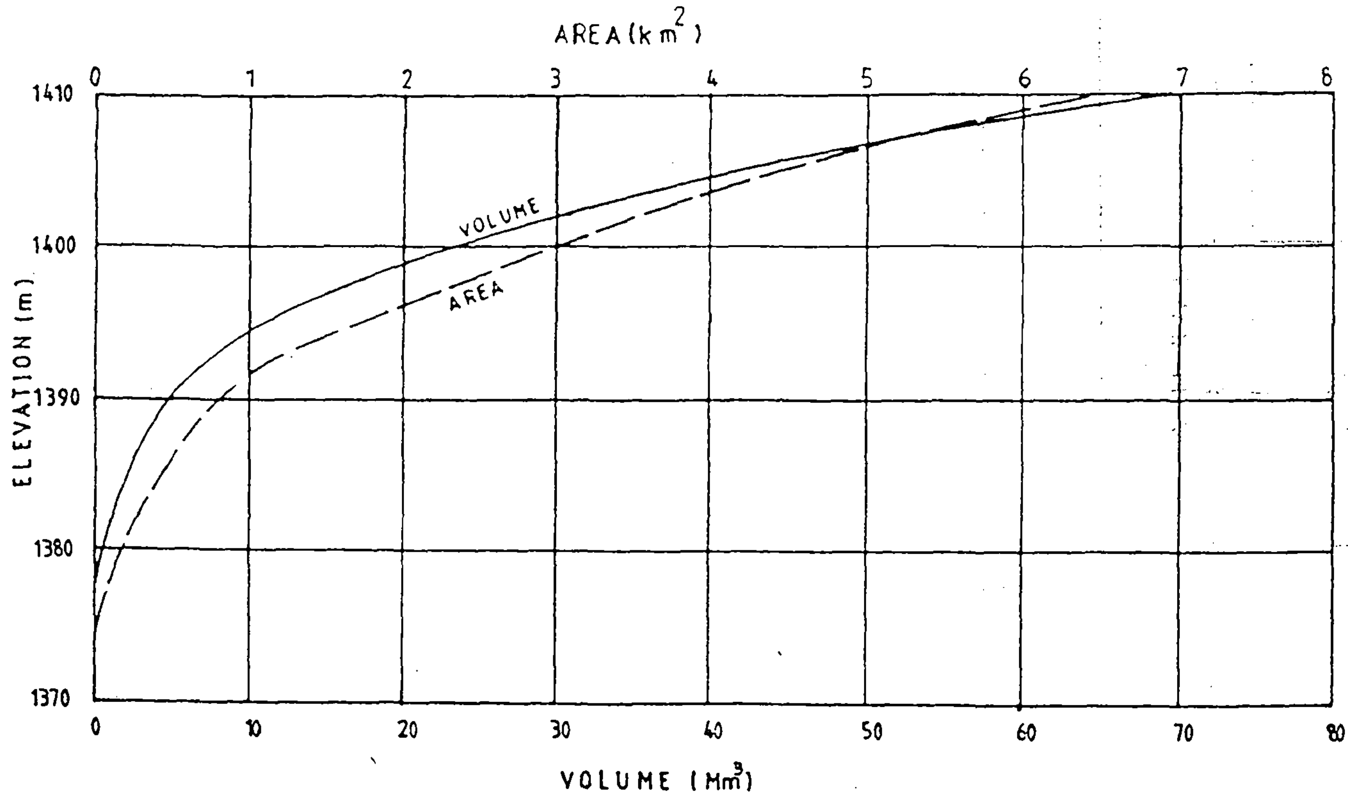
The most important downstream rights are domestic use, irrigation use, and hydroelectric power generation. Domestic use is almost always given priority over other uses, and at least some small releases must be allowed for from each dam for rural domestic use.

Various methods are available for estimation of gross yield as follows:

- Rippl Diagram, a cumulative mass plot of historic data, usually over a significant drought period.
- Stall Analysis, based on a statistical analysis of a long series of historic monthly data, or generated (synthetic) data.
- Operation simulation, again using a long series of either historic or generated data.

The last two methods require a reliable set of flow data. The generation of synthetic data is beyond the scope of this manual. The extension of flow data, and methods of correlation of rainfall and runoff between catchments is not usually justified for small-medium projects, and would require specialist assistance:

An example in the use of the Rippl Diagram is given in Figure 4.15. First obtain the monthly inflows at the site of the proposed dam for a critical drought period. An allowance may be made for evaporation from the reservoir surface, but as noted earlier this may not be any greater than from the area prior to construction of the reservoir.



Typical Reservoir Area/Volume/Elevation Curve

FIGURE 4.14

Plot the cumulative runoff (Figure 4.15). Connect the 2 points A and B: the slope of this line represents the gross yield (firm yield) for the specified drought, in this case 28,500 m^3/d. The required live storage is indicated by the vertical ordinate CD at the point of maximum draw-down of the reservoir. An allowance should be made from the gross yield for reservoir seepage, if necessary, and for downstream releases.

If the water supply demand were, say 15,000 m^3/d, the downstream releases 4000 m^3/d and estimated seepage 1000 m^3/d a gross yield of 20,000 m^3/d would be needed; thus, the required storage would be reduced to 2.8 Mm^3.

For some catchments the critical drought period may last for more than 1 year, and the Rippl diagram should be extended to cover such events if necessary.

4.12 Water Resources Management

This is a topic which has not received much attention in Sri Lanka. There are various aspects to the subject, which are discussed as follows:

4.12.1 Utilization of Resources

In any large catchment or basin, the surface water and groundwater resources need to be shared between a variety of different uses, such as water supply, irrigation, animal watering, power generation, industrial, etc., and as development proceeds it is vital to ensure that the available resources are utilized in an efficient manner. It is unacceptable, for example, for a rubber factory to be discharging noxious effluent into the upper reaches of a stream, rendering it unusable by potential downstream users.

Large users of water, such as for irrigation and hydro-electric power, must make allowances for relatively minor but high priority downstream needs for water supply.

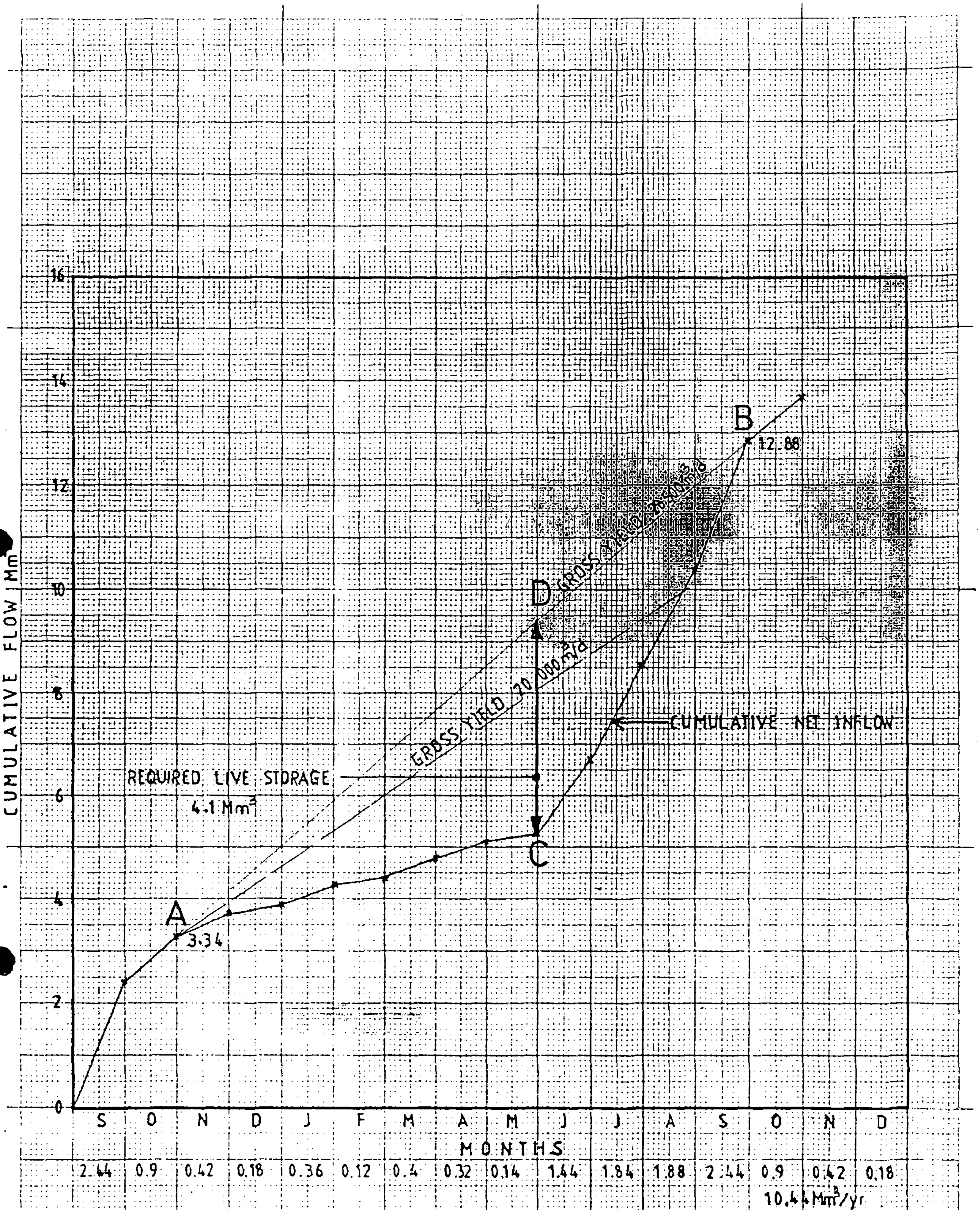
Unfortunately there are at present no effective controls over abstraction of surface or groundwater. Sometimes, more effective utilization of resources could be made by conjunctive use of a surface water source and groundwater source.

4.12.2 Legal Aspects

In many countries, water resources are controlled by state organisations and legal provisions regarding such aspects as:

- Ownership of and rights to water supplies;

- Rules about abstraction and quantity;



Yield Estimation By Rippl Diagram

FIGURE 4.15

- Rules about water purity and pollution;

- Rules about discharges, movement of water and land drainage;

- Rules about water supplies;

For example, it would not be acceptable for an individual to divert water from a stream to irrigate his fields and deprive a downstream community (village) of water which it had been abstracting as a drinking water supply for many years. Neither would it be acceptable for the state to divert the whole flow of a steam for hydroelectric projects and deprive a downstream private individual who has traditionally watered his fields adjacent to the stream. Compensation water - downstream releases past the diversion - would be necessary.

Where the source is limited the question arises as to who is allowed to take how much, for what purpose, to avoid depleting the source and causing conflict between users. Traditionally, such decisions are reached by mutual agreement between the parties, but there are as yet no specific legal requirements for water licences - permits to abstract a certain quantity for a specified use over a specified period.

Rules about pollution would prevent, for example, indiscriminate discharges of noxious effluents to upland streams by rubber factories, or slaughter houses.

Also associated with this topic are the rules concerning protection of rights of private individuals concerning entering or carrying out surveys on private property, acquiring of easements or land by negotiation or compulsory purchase, etc. For these there is usually an established procedure within the existing law which should be followed.

All the above provisions are designed to protect the rights of individuals, communities and the state.

Various steps have been taken in Sri Lanka towards developing a similar approach to control of water resources ~~management~~ including:

- A draft Watershed Management Act (as reported by S. Arumugam (Ref.3)).

- A draft Water Resources Act of 1980, prepared with USAID assistance, which deals with ownership and use of water, and protection of sources.

- The National Environmental Act No.47 of 1980 and establishment of the Central Environmental Authority (CEA). (This Act is currently being amended).

- Various Environmental Standards for discharges and water quality by the CEA and Sri Lanka Standards.

- In March 1987, a move by the Ministry to establish a National Groundwater Authority and enact a National Water Act.

- A Forestry Master Plan for Sri Lanka by the Forest Resources Development Project in 1986.

However, much work remains to be done and in the meantime, before effective legislation and controls are in place, engineers should be aware of the potential problems and plan accordingly. Particularly be aware of any potential conflicts regarding different uses or users of a limited water source.

If any queries or clarifications are required on any of the above topics for specific projects, the NWSDB Legal Officer should be contacted.

4.12.3 Watershed Management

It has recently been reported, in the Forestry Masterplan mentioned above, that the forested area in Sri Lanka has diminished significantly this century:

1900	70%	forested
1956	45%	forested
1981	27%	forested

These are startling statistics concerning land use and water resources, the detrimental effect being erosion, and loss of productive soils, increased sedimentation of reservoirs and reduction of yield, increased flood peaks due to rapid runoff, and reduced infiltration and low flows.

Watershed management is therefore a vital requirement for responsible water resource management, and measures should be taken to reduce and reverse the detrimental effects noted above.

Although erosion and sedimentation are natural phenomena, dependant on the geology, soils and land cover, various types of land use can accelerate or reverse the trend. Those having a negative effect (land misuse) are:

- overgrazing;
- deforestration;
- poor cultivation practices.

Measures to reverse the trend can be:

- grazing controls;
- planting of timber and fruit trees;
- terracing;
- improvement of cultivation practices;

- channel protection;
- gulley plugging;
- construction of stream diversions;
- check and drop structures;
- silt trapping reservoirs;

Some of these measures cost money to implement and it may in some instances be difficult to demonstrate the economic benefits of such expenditure, as is the case with preventive health programmes, for example.

However, Engineers should recognise that watershed management is of considerable importance for the long term future of water resources in Sri Lanka and should strive to promote effective measures in all projects.

5. INTAKES, WEIRS AND DAMS

5.1 Scope

This section deal with surface water intakes for medium or larger urban or semi-urban water schemes. Refer to Rural Water Supply Manual D1 for minor intakes from streams, springs, shallow wells or tanks, including infiltration intakes (wells and galleries). Refer also to Groundwater Manual D4 for tube-well intakes.

5.2 Types of Surface Water Intake

The following types are commonly used:

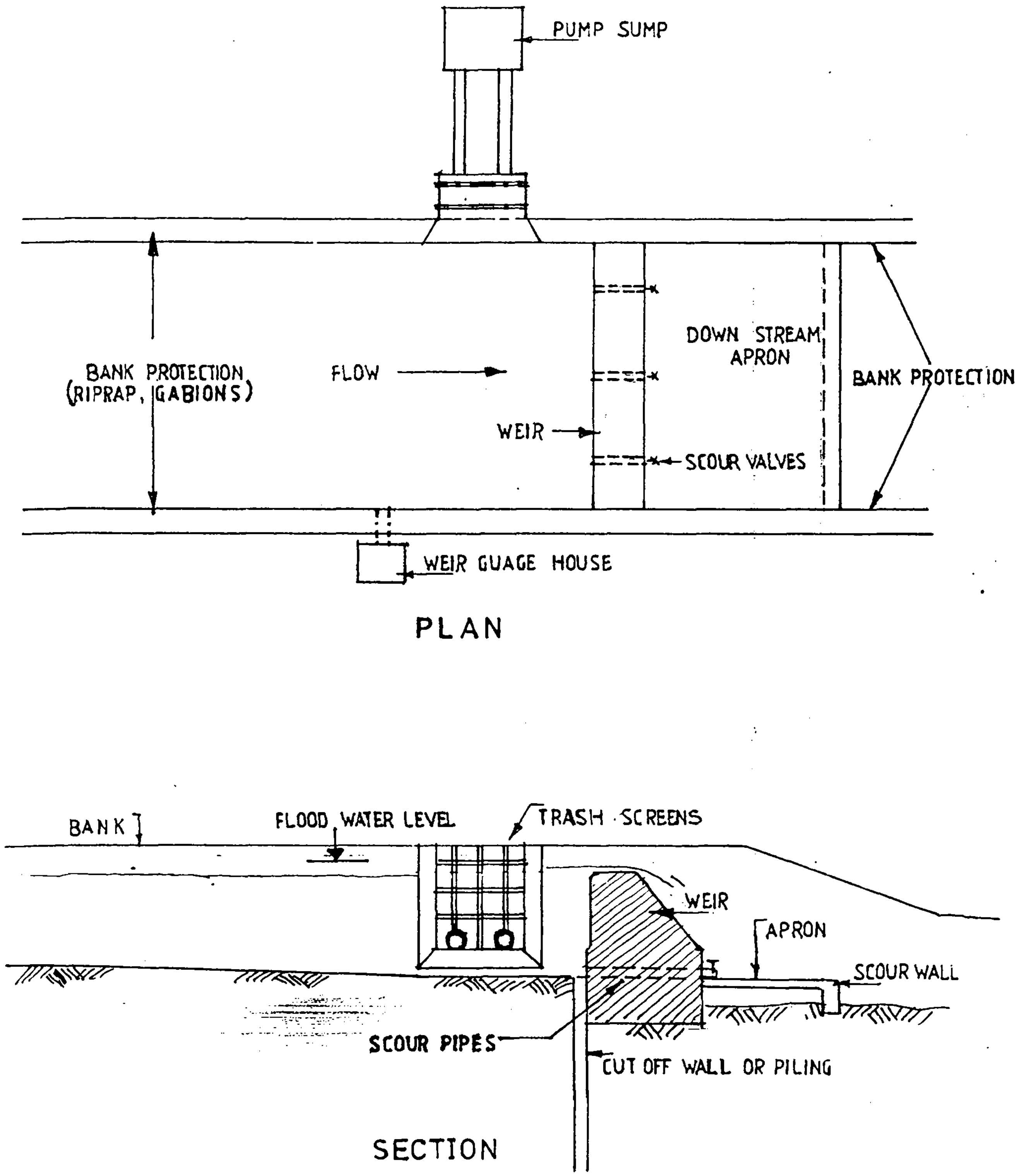
- o ponding by construction of a weir across the channel, with submerged intake upstream, or side intake (see Figs. 5.1, 5.2);
- o where natural depth of water in channel permits, a submerged intake or exposed (direct) intake on the bank; (see Figs. 5.3, 5.4);
- o a floating pontoon with pumps, anchored to the bank; this type provides flexibility when dealing with large water level variations (see Rural Water Manual);
- o a submerged wooden or concrete crib intake anchored to river or bank bed (see Rural Water Manual); pipes should be laid in dredged trench below bed level;
- o an intake tower, dry well type, for reservoir or tank permits abstraction away from bund or bank and at 2 or more water levels.
- o a side weir intake on irrigation canal- not usually reconnended due to dependence on irrigation releases.

5.3 Location and Design Considerations

5.3.1 All Intakes

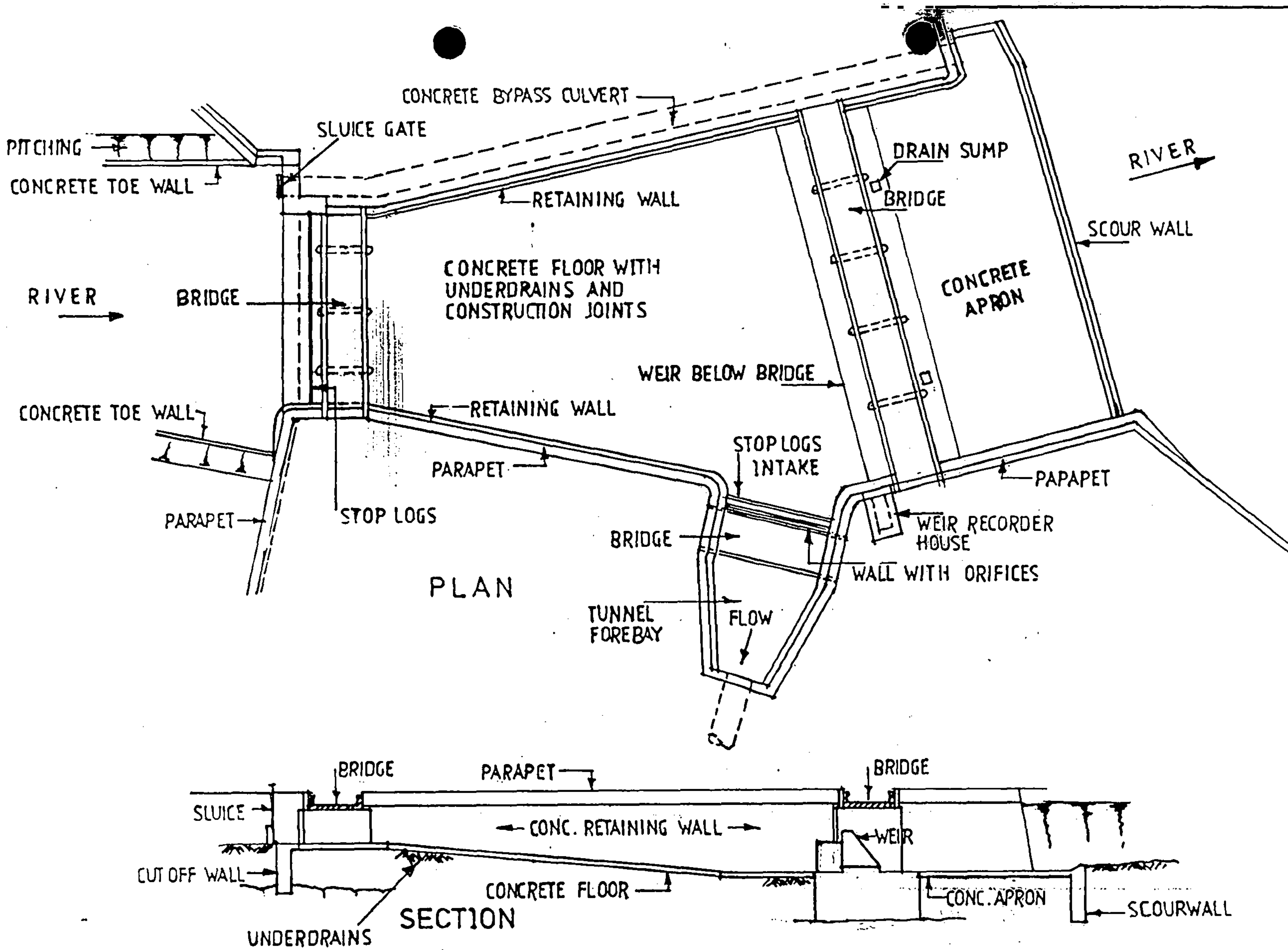
Take into account the following aspects:

- o prior right of other water uses of the same source (e.g. for water supply, irrigation etc.)
- o fluctuations in flow rate and water quality carry out adequate gauging and sampling;
- o locate intake for optimum water quality;
- o is low flow quantity sufficient for demand?
- o low and high water levels - if variation less than 3.5 - 4.0 m, can use suction pump on bank;



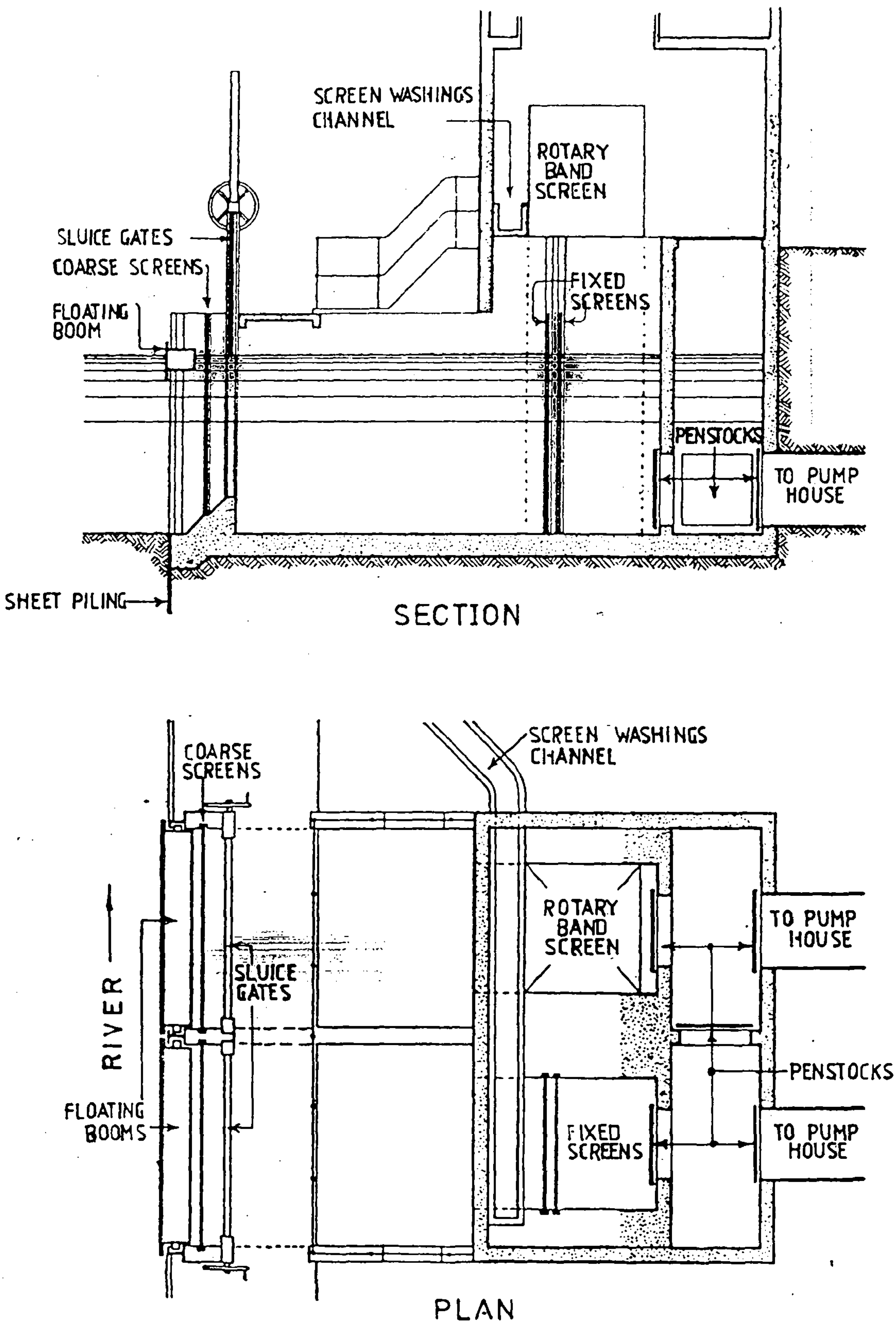
TYPICAL WEIR INTAKE-1

FIGURE 5.1



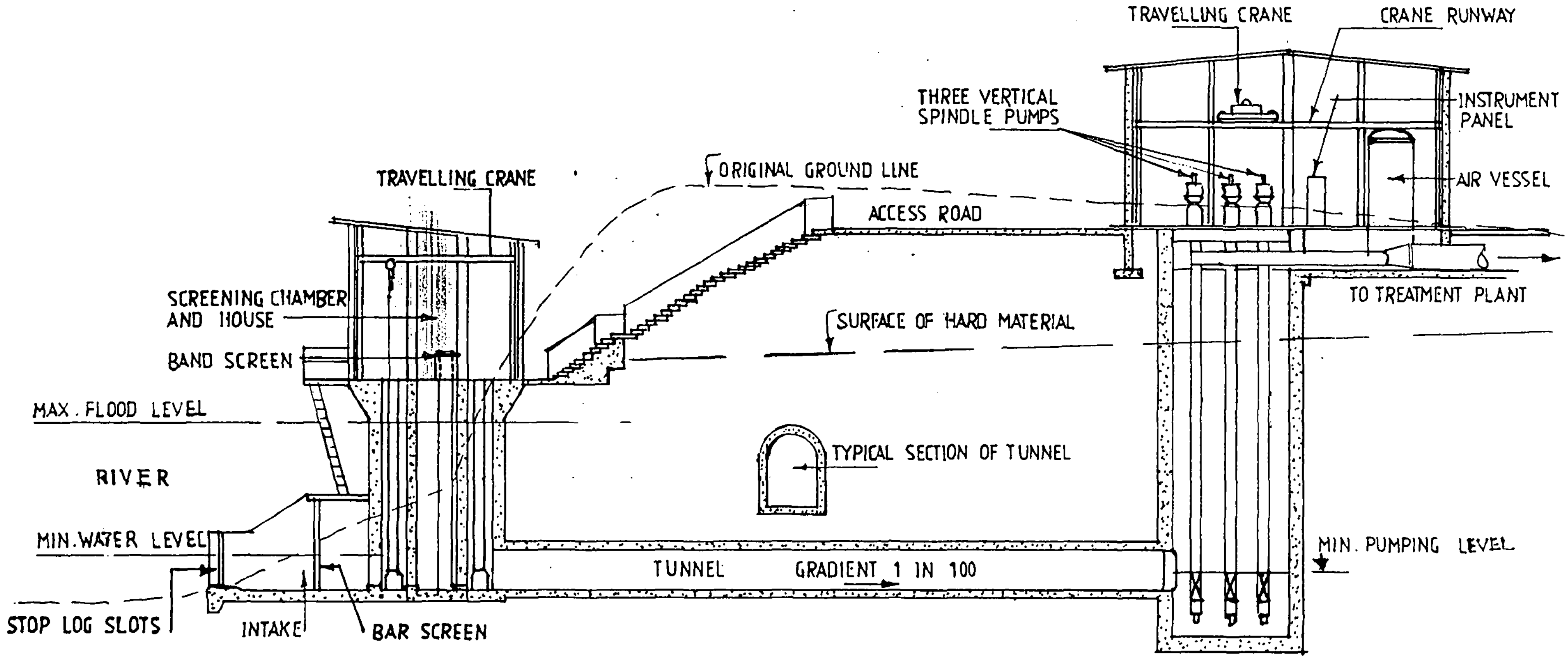
TYPICAL WEIR INTAKE_2

FIGURE 5.2



TYPICAL DIRECT INTAKE - 1

FIGURE 5.3



TYPICAL DIRECT INTAKE - 2

FIGURE : 5.4

- check for historic flood levels and site bank works at suitable level;

- check for upstream pollution sources and evaluate impact on quality;

- accessibility of site and availability of power, if required;

- distance from pump station, treatment plant and demand area;

- assess currents and avoid high velocity channels in river;

- assess stability of structures under flood conditions and impact of floating objects;

- design to minimise scouring of channel or banks, and clogging or siltation of intake;

- intake opening facing downstream will minimise entrance of floating debris during floods;

- in rivers with high sediment content, locate intake near outside bend of river where sediment concentration may be less;

- similarly in rivers with high sediment content, locate intake opening at or near surface as sediment concentration increases towards river bed.

- where there are large fluctuations in river level, consider use of a floating pontoon intake, or multiple level intake.

- care should be taken to minimize air entrainment into pipelines and pumps; the intake pipe or pump suction should be located in a quiescent area. A rule of thumb is to maintain a submergence of at least 1½ to 2 times the diameter of the intake pipe.

- where the intake pipe or conduit discharges into a pump suction well, the horizontal area of the suction well should be 3 to 5 times the vertical x-sectional area of the intake pipe.

- where substantial underground flow is suspected in stream beds which tend to dry up on the surface, consider use of subsurface dam or cut-off across bed to intercept flow;

- assess stability of channel bed and banks - try to avoid formation of shoals and bars;

- submerged intake pipes should end in a screened bellmouth opening at least 1m above bed; entrance velocity should be less than 0.15 m/s to minimise drawing in silt, and pipe flow velocity should be 0.6 - 0.9 m/s;

- exposed intakes should be located on a straight section of channel in area of stable channel and banks;

- assess effect on fish life;

- provide screening as required and means of cleaning or raking;

- provide means of scouring and desilting intake - possibly by backwashing through intake pipe;

- intake pipes should be laid to a constant rising or falling grade to avoid accumulation of air pockets which would reduce capacity;

5.3.2 Weirs

Specifically for weirs, take into account the following:

- downstream scouring action - provide scour-proof foundations and cut-off trench (or sheet piling) as necessary;

- allow for passage of boats past weir if necessary;

- provide protection against erosion of upstream banks due to raised water levels, and against erosion of downstream banks due to turbulence; assess need for rip-rap, gabions or retaining walls, depending on bank material and slopes;

- provide fish pass if necessary;

- all weirs should incorporate a flow measuring device;

- provide for washout or bypass to flush poor quality water;

- consider siting intake upstream of natural drop in river bed (ie. due to fault) which would probably have a good rock foundation;

- provide for any required downstream releases, through sluice gate or side weir;

- design weir to carry flood flows, without damage to structures or banks;

- advantages of weirs are:

    - constant head at intake point,
    - measurement of stream flow possible,
    - can be designed to prevent tidal backflow of sea water when located near coast;

- disadvantages of weirs are:

    - high cost, if river is wide,
    - upstream silting may require provision of scour pipes and regular desilting.

5.3.3 Reservoirs and Tanks

Specifically for reservoir and tank intakes, take into account the following:

- avoidance of areas of poor circulation or stagnation of water, or areas accumulating debris;

- effects of wind and waves on structures; in shallow water, wave action may stir up bottom deposits;

- intake should be at deepest point to take maximum advantage of reservoir capacity.

- assess algae blooms at certain times of year and additional treatment requirements;

- multiple level intake may be preferable to ensure optimum water quality.

5.4 Intake Screens

Coarse screens of 75-100 mm opening prevent the entrance of large objects into the intake and are usually in the form of inclined or vertical bars 25 mm in diameter in the intake channel, or a wooden or concrete crib around the intake pipe entrance. Access should be provided for easy raking and removal of debris.

Removable fine screens, having about 6 mm openings, fabricated in frames, following the coarse screens, are usually necessary to prevent the entry of fish and small objects. Two screens should be provided so that one screen can be cleaned while the other is in place.

Intake screen velocity should be limited to about 1 m/s and the maximum headloss through a clogged screen should be about 150 mm in order to minimize the drop in stream water level where a diversion weir is provided. Headloss through a screen may be estimated by the following formula:

$$h = 0.0729 \, (V^2 - U^2)$$

where h = headloss, (m)

V = Velocity through the screen (m/s)

U = Velocity before the screen (m/s)

Mechanical band or drum screens may be necessary for major intakes.

5.5 Small Dams

5.5.1 Site Investigations

Small dams discussed in this section are considered to be less than 10 m in height. Factors to evaluate in consideration of alternative dam sites are as follows:

- location (distance and elevation) with respect to demand area;
- quantity and quality of water available;
- access to site, roads;
- topography of site - volume of storage and volume of fill for various dam heights (volume - elevation curves). The optimum case will be that giving the greatest storage for the least fill; (i.e. smallest dam size);
- preferred location is a wide flat valley, upstream of a narrow gorge;
- geological conditions, possible seepage under dam, and watertightness of reservoir and necessity for grouting;
- availability of embankment fill material and other construction materials;
- inundation of lands; loss of homes or agricultural or forested land; possibility of biological problems if a shallow reservoir;
- possible erosion and land slide activity in valley due to raised water table; check slope stability;

When a site or sites have been selected for further consideration, it is necessary to conduct detailed soil investigations to assess seepage, foundation conditions and stability.

Foundations must be capable of supporting the weight of the dam, and preventing seepage underneath it. Avoid areas of springs, soaks, or landslips which indicate unstable conditions.

Rock is the best foundation providing it is not too weathered, jointed or faulted, in which case grouting will be necessary. Care must be taken to ensure there is no seepage path between the rock foundation and the earthfill dam.

A clay foundation, if of the same material as the fill, should also be suitable. If soft and saturated, however, additional stabilizing fill will be necessary.

Sands and gravel foundations are not suitable due to high seepage losses.

Sub-surface investigation is usually done with a hand auger boring tool, with test holes sunk to rock or to a depth of three quarters of the proposed dam height. Under the dam, a spacing of 20-40 m should be suitable. A test trench should also be cut to examine the foundation in its natural state.

The same auger boring method is used to locate suitable fill material preferably in the reservoir area. Soil for fill should be sufficiently impervious to restrict seepage and should be stable when compacted in a slope. A suitable soil contains 25% clay with the balance of silt, sand and gravel. With too much clay, the fill will be sensitive to moisture changes, causing it to expand and contract; with too little, it will not be impervious enough. Test holes in borrow pits should be about 3m deep or 0.6 m below the proposed borrow pit depth.

Seepage under the reservoir area can be assessed by a test using 3m deep test holes, 100 mm in diameter, with the following procedure:

- Select 3 or 4 holes;
- presoak each hole to a 2 m depth of water for at least 1 hour;
- commence test and maintain each hole's water level at 2 m depth, recording the amount of water used;
- continue for 1 day and calculate average refilling rate for each hole;

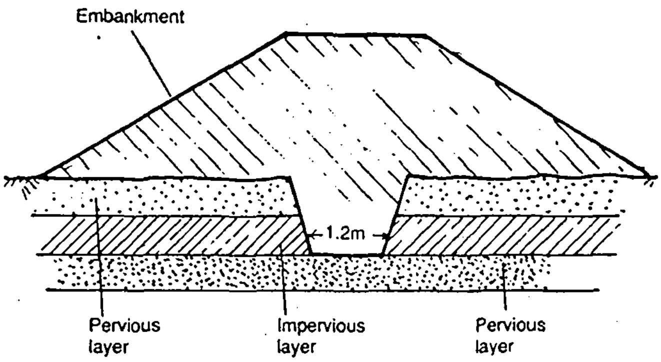
If the rate is less than 3 litres/hr the site should be satisfactory and if more than 30 litres/hr, unsatisfactory. Between these two values, judgement will have to be used to weigh up the potentially high seepage against other factors. If seepage measurements are not available, a conservative assumption would be a monthly seepage loss of 0.5% of the volume of water stored.

For adequately assessing site topgraphy, reservoir storage and dam fill quantities, contour mapping at 1:5000 scale with contour intervals of 5m, for the reservoir area, and 1:2000 scale with contour intervals of 2 m for the dam area, should be satisfactory.

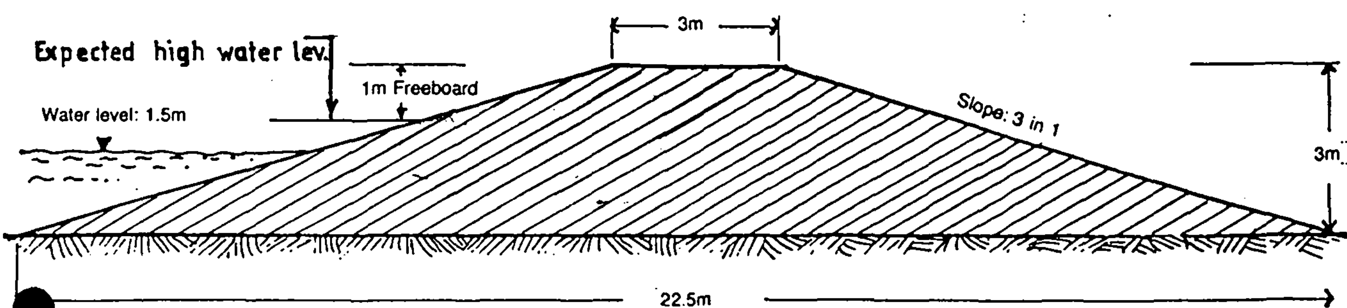
5.5.2 Typical Designs

Figures 5.5, 5.6 give typical embankment sections for small dams. The following design considerations should be taken into account:

- o freeboard - the distance between the top of the dam and the high flood level of the reservoir should be a minimum of 0.9 m;
- o foundation - necessary cut-off trenches to rock or impervious stratum should be allowed for; any soft or unsuitable ground must be excavated down to firm ground or rock;
- o consider maximum flood discharge for spillway design (see Section 5.5.3);
- o assess loss of reservoir capacity over time due to siltation; make allowance for dead storage and calculate minimum operating level;
- o preparation of catchment (watershed management) and stripping vegetation from submerged area;
- o earth dams should be for full development of the source since they are not suitable for raising;
- o do not site new structures (intakes or pump stations) on existing dams or bunds, or allow blasting or piling in their vicinity, which may cause damage or instability; (Ref. 24)
- o make thorough provision for diversion of river flow (including likely flood flows) during construction stage; a diversion pipe can be built in under the dam to act as a future supply pipe or washout; it should have anti-seepage collars to prevent it becoming a seepage path;
- o prevent erosion of the downstream slope by planting creeping grass or other suitable vegetation cover (not trees) and on the upstream side the dam should be paved or lined with stone (rip-rap) to a height of 0.6 m above water level;
- o typical side slopes for earth embankments are as follows:



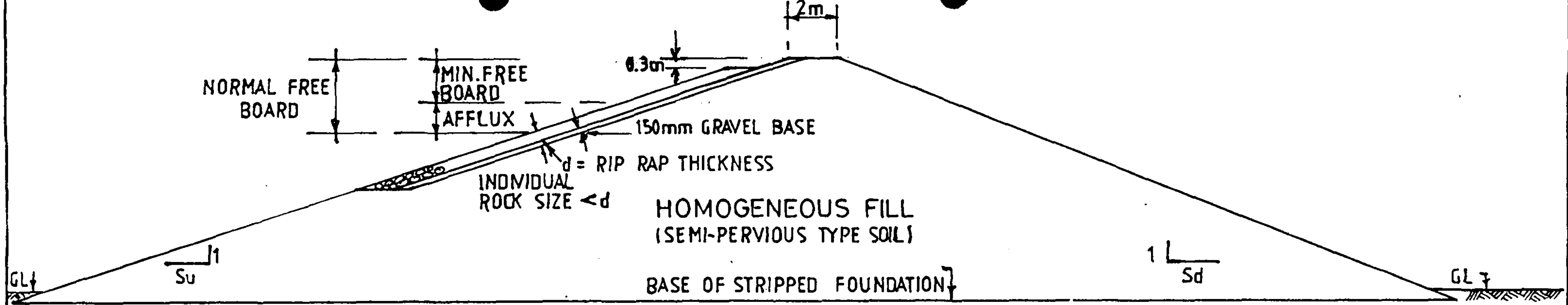
CUT OFF TRENCH



TYPICAL CLAY DAM EMBANKMENT

TYPICAL SMALL DAM SECTIONS

FIGURE 5.5



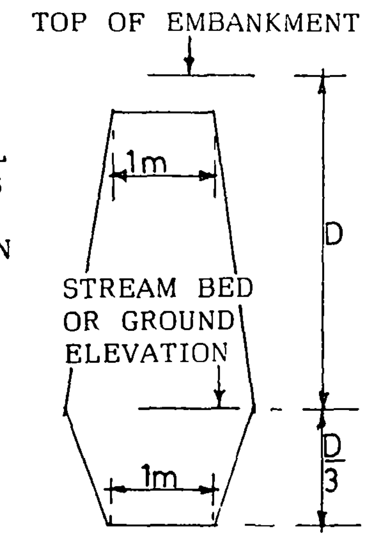
SIDE SLOPES FOR COMPACTION BY MACHINERY & EQUIPMENT	S_u	S_d
HOMOGENEOUS MATERIAL (SEMI-IMPERVIOUS TYPE SOIL) FOR EMBANKMENT HEIGHT UP TO 6 m	2	2
HOMOGENEOUS MATERIAL (SEMI-IMPERVIOUS TYPE SOIL) FOR EMBANKMENT HEIGHT OVER 6 m up to 9 m	2½	2

RIP-RAP THICKNESS "d" IN mm		
FETCH IN km	MIN. AVERAGE ROCK SIZE IN mm	RIP-RAP LAYER THICKNESS IN mm
UP TO 0.6	250	300
OVER 0.6	300	450

FOR HOMOGENEOUS FILL MATERIAL OF PERVIOUS SOIL, USE IMPERVIOUS CENTER CORE AS SHOWN

$S_u = 2$

$S_d = 2$



NOTE

AT STREAM/BREACH, EXCAVATE FOUNDATION TO MIN. 1/3 HEIGHT OF EMBANKMENT

TYPICAL SMALL DAM SECTIONS - 2

NOTE: HEIGHT NOT EXCEEDING 9m

Source: Adapted from Ref. 6

FIGURE 5.6

Fill Material	Side Slopes	
	Upstream	Downstream
Clay, clayey sand, sandy clay, silty sand	3:1-3.5:1	2.5:1-3:1
silty clay, clayey gravel, silty gravel	3:1-3.5:1	2.5:1-3:1
silt or clayey silt	3.5:1-4:1	3:1-3.5:1

o typical crest widths for earth embankments are as follows:

Height of Dam	Top Width
Under 3 m	2.5 m
3 - 4.5 m	3.0
4.5 - 6 m	3.5 m
6 - 7.5 m	4.0 m

5.5.3 Spillways

The provision of properly engineered spillways, to pass floods larger than the dam can regulate, is essential for protection of the investment, security of water supply and protection against catastrophic dowstream inundation. Flood waters must never be permitted to spill over the top of the earth dam, which would cause dam failure. Spillways for dams of the size and importance of those required by NWSDB must be designed to safely pass the flood referred to by hydrologists as the "probable maximum flood" (PMF). The probable maximum flood is the largest flood that could occur at the site under the current climatic regime.

In final design, PMF volumes should be considered, as well as peak flows, and the PMF hydrographs should be routed through the proposed reservoirs. For the sizes of reservoirs considered, attenuation of flood peaks due to routing would likely be small.

The selection of PMF, and routing the flood through the reservoir are beyond the scope of this manual, and reference should be made to the Hydrology Division, Department of Irrigation, and References 6 and 7 .

There are various types of spillways:

- o natural spillway (ungated);
- o gated spillway (not recommended);
- o clear overfall spillway;
- o morning glory spillway.

Natural spillways are excavated channels or natural spill sites, located with an invert level equal to the reservoir top water level. Commencing with a broad crested weir, or control section the channel (of suitable section) is routed around the side of the dam in original ground, with a gradient designed to limit the flow velocity to the permissible scour velocity. A terminal stilling basin may be necessary for downstream scour protection. Channels may be lined with masonry or concrete, or unlined. The spillway must always be built well clear of the dam, and the channel continued downstream, away from the downstream edge of the dam base, in order to prevent erosion of the dam by flood water. (see Fig. 5.7)

Details of other types of spillways are available in References (6), (7), (8).

5.5.4 Maintenance and Inspection

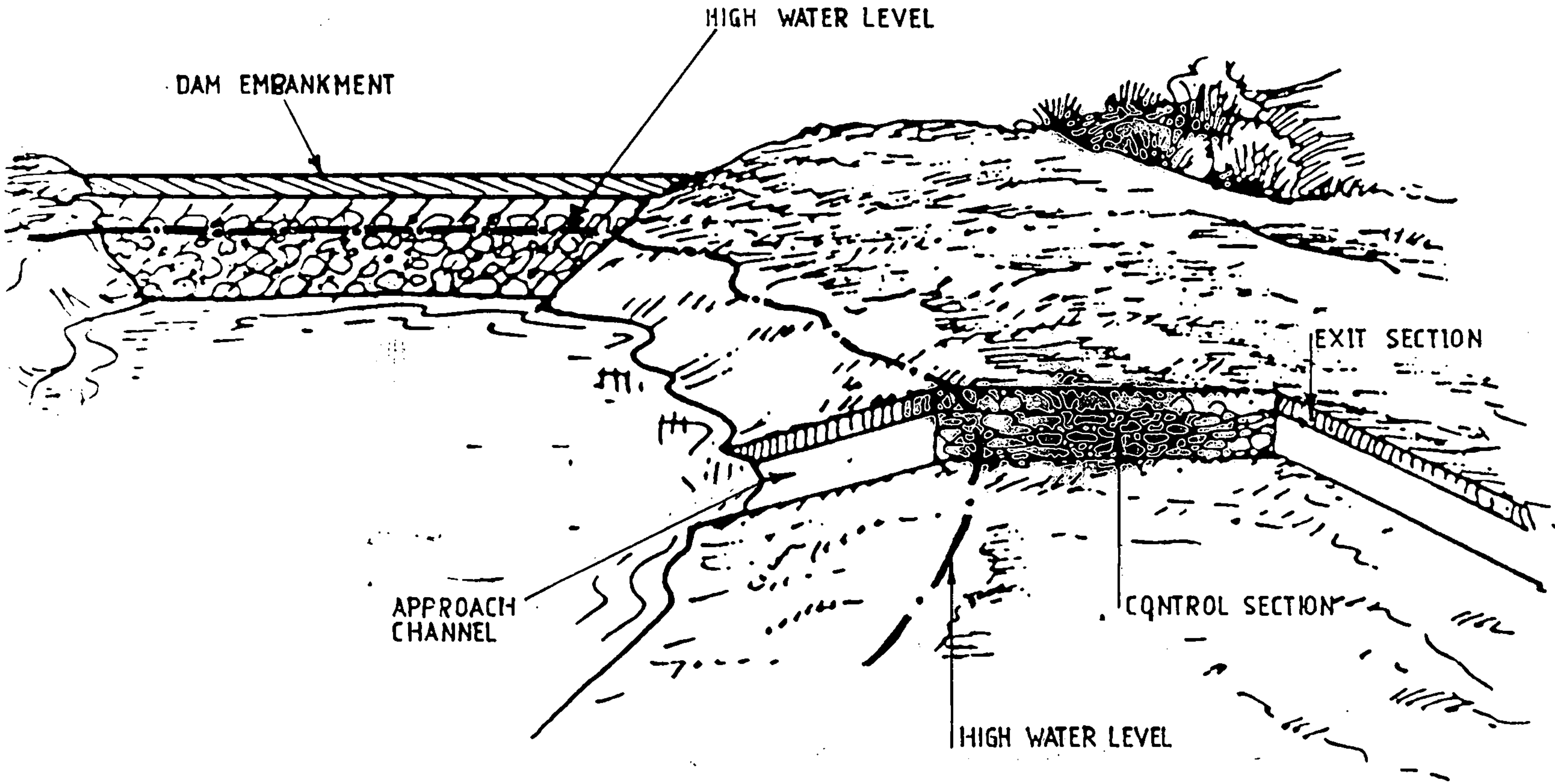
Regular preventive maintenance and periodic inspection of dams by a qualified engineer is necessary to ensure their safety.

Common problems, which may lead to failure if left unattended are:

- o leakage through the embankment;
- o undermining, by flow underneath the structure;
- o erosion of the embankment; and
- o slope failure, or slips

Check for the following:

- o seepage through the embankment or at downstream toe;
- o erosion or irregularity of the slopes;
- o movement of the embankment;
- o burrowing by animals;
- o maintenance and cutting of grass covering of downstream slope;



LOCATION OF SPILLWAY

FIGURE :5.7

- ensure trees or bushes do not develop on embankment - the roots of which may form drainage channels;

- wave damage to upstream rip-rap;

- settlement or irregularity of crest;

- erosion or debris in spillway channel and stilling basin;

- proper functioning of draw-off works, gates, valves pipes and culverts - gates and valves should be exercised at regular intervals.

- leakage through or around pipes or structures which form part of the dam;

Any seepage, erosion or other damage should be treated seriously and corrective measures taken immediately.

6. PUMPING STATIONS

6.1 Scope

This section covers basic considerations for hydraulic, structural and equipment design of pumping stations. For more detailed information on selection of pumps, motors and instrumentation, refer to Manual D5 Mechanical Electrical and Instrumentation Design.

6.2 Hydraulic Design

6.2.1 General

Centrifugal pumps are most commonly used for pumping water. The deep-well turbine pump is a combination of several stages of centrifugal impellers connected in series to a common shaft.

There are three different types of centrifugal pumps depending on the direction of movement of water from impeller. These are 'radial flow', 'axial flow' and a combination of the two, called 'mixed flow'.

The ordinary centrifugal pump (radial flow) is preferred for a wide range of capacities with low-head applications and the turbine type is for high heads (with two or more impellers in series). For large flows under small heads, axial flow pumps are best suited.

The operating variables of a pump are the discharge, the head, the speed of rotation, the shaft power and the suction head. Factors to be considered in selection of pumps are:

- o capacity;
- o depth of well and pumping level;
- o inside diameter of well;
- o verticality of well;
- o abrasive properties;
- o total head;
- o type of power available;
- o cost.

In selecting a pump, the operating service characteristics should be compared to the manufacturer's performance characteristics of the selected pump for best efficiency and satisfactory operation.

6.2.2 Suction Conditions

More pumping installations fail because of poor suction conditions than from any other single cause. A centrifugal pump has no capability to "suck" from a lower level, such as a well, unless it is initially primed and all the air is removed. In constrast, a reciprocating pump is capable of self priming, providing the plunger and valves are tight. A centrifugal pump is a kinetic energy machine designed to accelerate a volume of water from a low to a high velocity, and to convert this velocity into developed head at the pump discharge flange.

Net positive suction head (NPSH)

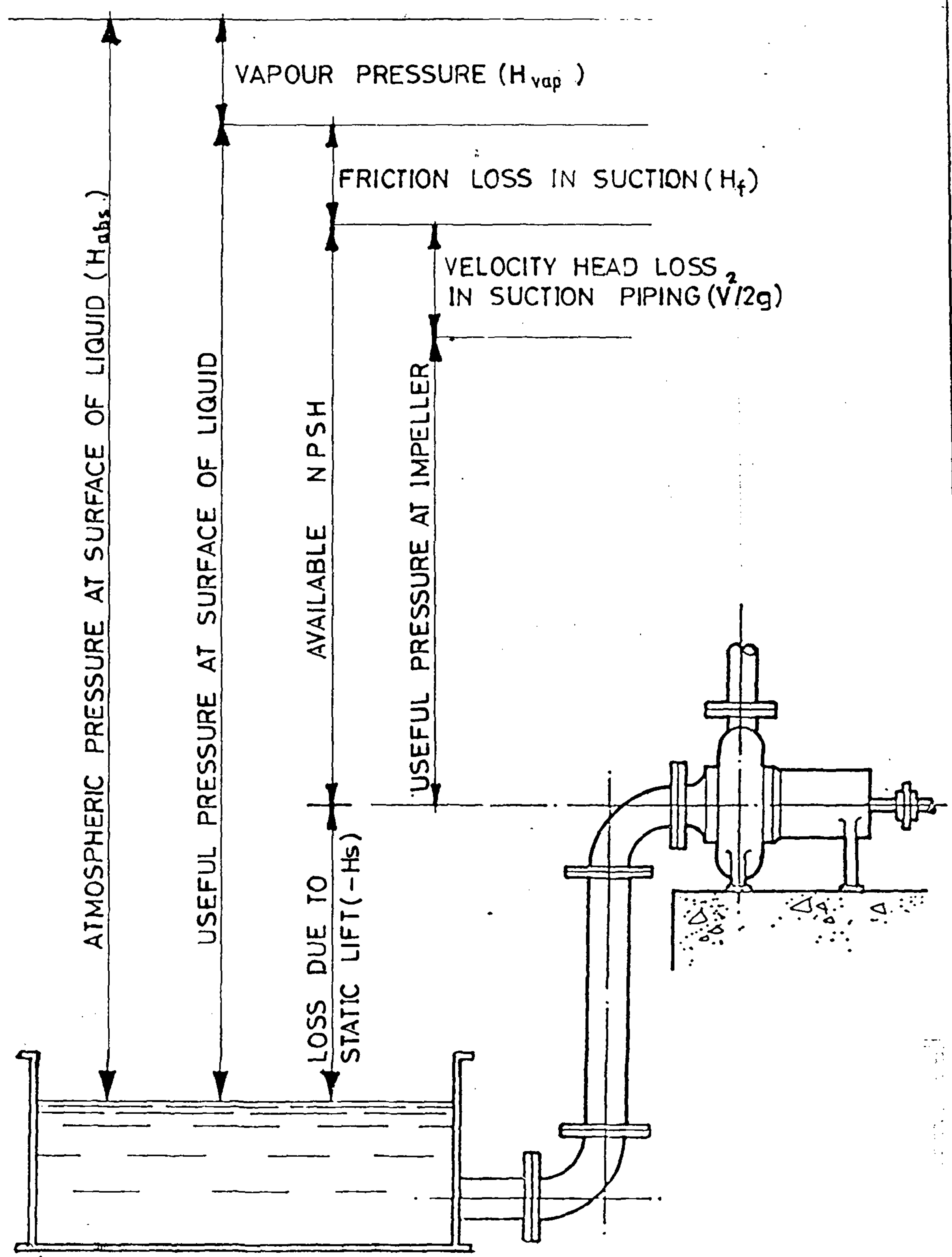
For a centrifugal pump to operate, the water must enter the eye of the impeller under pressure, usually atmospheric pressure, referred to as Net Positive Suction Head (NPSH). It is important to realize that there are two values of NPSH, the available NPSH, which depends on the location and design of the intake system, and can be calculated by the Engineer; there is also the required NPSH, determined by manufacturer's bench scale tests. The required NPSH is the suction head required at the inlet of the impeller to ensure that the water will not boil under the reduced pressure conditions and the impeller will operate smoothly without cavitation. It is essential that the available NPSH exceeds the required NPSH with a reasonable margin of safety of at least 0.7 to 1.0 m or more if possible. See Figures 6.1 and 6.2.

$$NPSH_{(available)} = H_{abs} + H_s - H_f - H_{vap}$$

where $NPSH_{(available)}$ is Net Positive Suction Head (m)

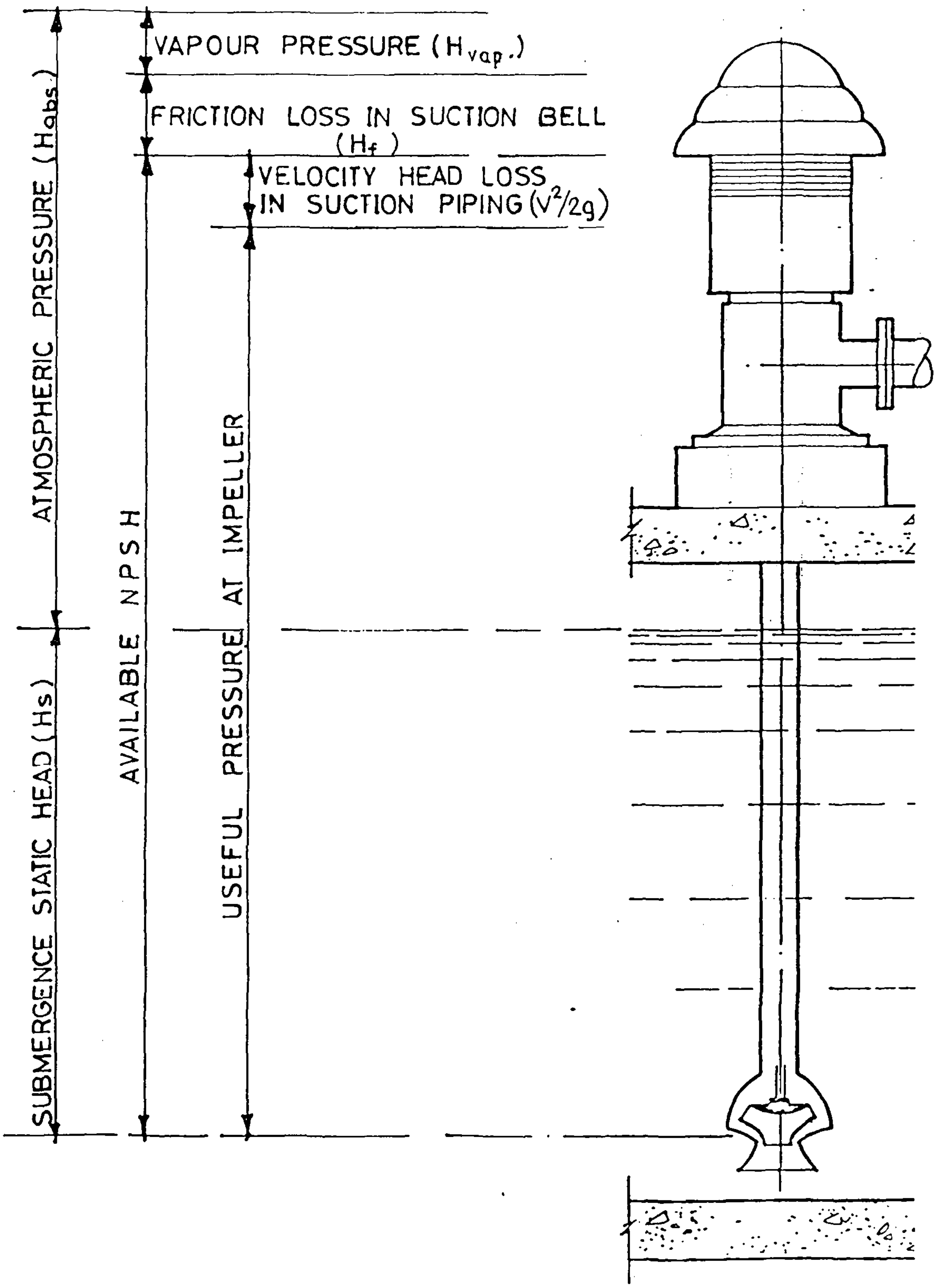
H_{abs} is the absolute pressure on the surface of the water in the suction well (m head of water) (see Fig.6.3).

H_s is the static elevation (m) of the water above the centreline of the pump (on vertical turbine pumps above the entrance eye of the first stage impeller). If the water level is below the pump centreline, H_s is negative.



NPSH FOR
CENTRIFUGAL PUMP
WITH LIFT

FIGURE 6.1



NPSH FOR
VERTICAL TURBINE PUMP

FIGURE 6.2

H_f	is the friction head and entrance losses in the suction piping (m).
H_{vap}	is the absolute vapour pressure of water at the pumping temperature (m head of water)

A "Standard Atmosphere" at sea level is equivalent to:-

1 atmosphere = 14.7 lbf/in²
= 760 mm of mercury = 1014 m bar
= 29.92 inches of mercury
= 33.93 feet of water

With changes of altitude, the "Standard Atmosphere" is modified (see Fig.6.3). Storms will also cause the atmospheric pressure to drop by up to 15%.

The vapour pressure of water increases with temperature which also reduces the available pressure at the pump suction (see Table 6.1).

<u>Example</u>:

A 200 l/s vertical turbine pump is located at 1500 m above sea level and is pumping water at a maximum temperature of 38° C. The suction bell is 600 mm in diameter reducing to 300 mm diameter at the first bowl assembly. The water level is never less than 2.3 m above the first stage impeller. What is the available NPSH under the worst conditions?

NPSH available = Habs + Hs − Hf − Hvap (m H_2O)

Habs = 630 mm Hg (see Fig 6.1)

$= \frac{630 \times 1.33322}{98.0665}$ m H_2O (see Table 6.2 Notes)

= 8.56 m × 0.85 (for storm conditions)

= 7.28 m

H_s = 2.3 m

$H_f = 0.1 \frac{V^2}{2g}$ for a suction bell

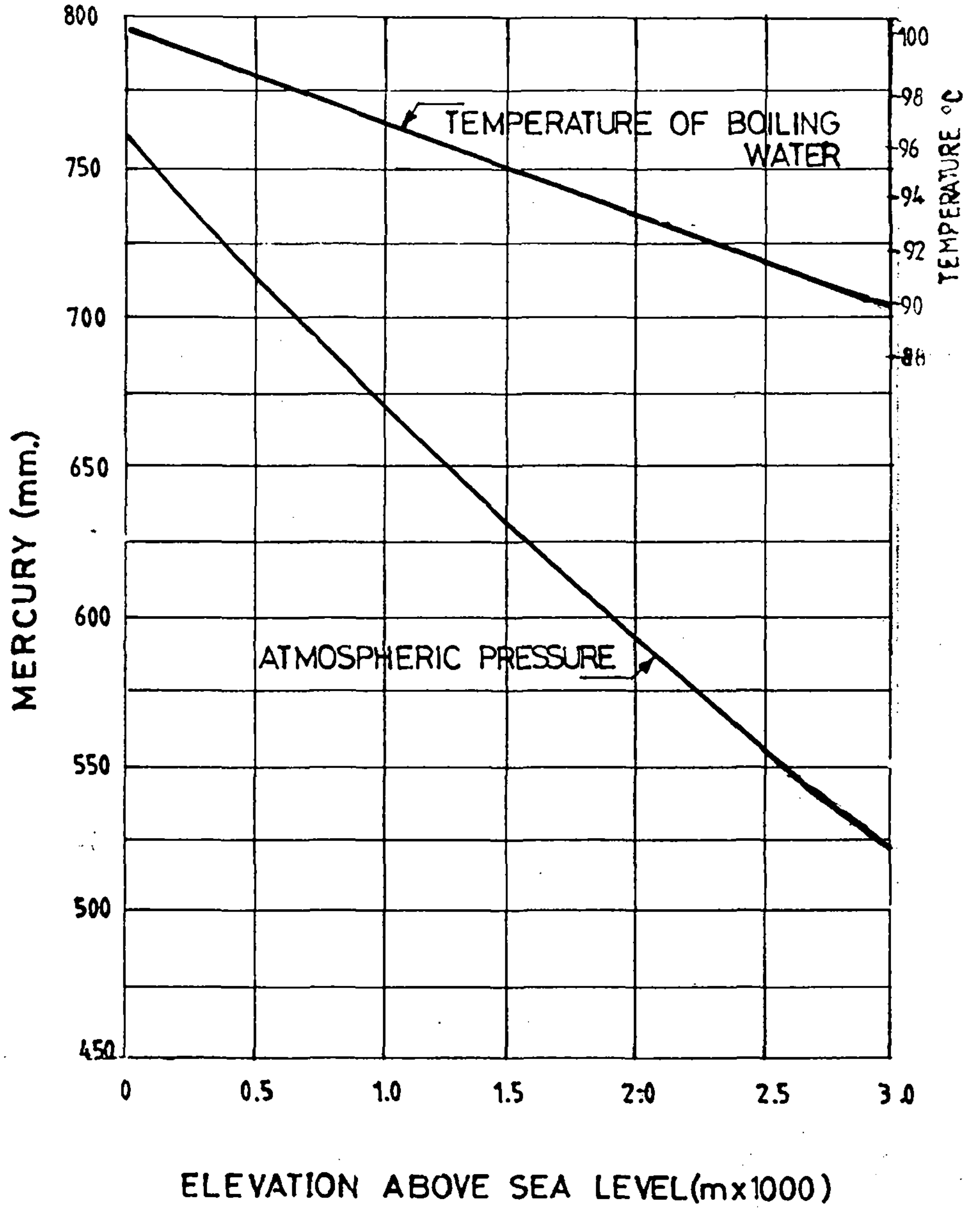
Where $V = \frac{Q}{A} = \frac{0.200}{\pi 0.3^2/4} = 2.83$ m/s

$H_f = 0.1 \cdot \frac{2.83^2}{2 \times 9.78} = 0.04$ m

H_{vap} = 0.67 m (Table 6.2)

Therefore $NPSH_{avail}$ = 7.28 + 2.3 − 0.04 − 0.67

= 8.87 m



ADAPTED FROM PUMP HANDBOOK, KARASSIK.

ATMOSPHERIC PRESSURES

FIGURE 6.3

Table 6.1
Properties of Water

Water Temperature		Absolute Viscosity (Centipoises)	Specific Gravity		Specific Weight		Absolute Vapour Pressure	
° F	°C		16°C Reference	21°C Reference	lb/ft	kg/l	mbar	mH_2O
40	4.4	1.54	1.001	1.002	62.42	1.000	8.39	0.09
50	10.0	1.31	1.001	1.002	62.38	0.999	11.85	0.12
60	15.6	1.12	1.000	1.001	62.34	0.998	17.67	0.18
70	21.1	0.98	0.999	1.000	62.27	0.997	25.03	0.26
80	26.7	0.86	0.998	0.999	62.19	0.996	34.95	0.36
90	32.2	0.81	0.996	0.997	62.11	0.995	48.14	0.49
100	37.8	0.68	0.994	0.995	62.00	0.993	65.45	0.67
120	48.9	0.56	0.990	0.991	61.73	0.989	116.66	1.19
140	60.0	0.47	0.985	0.986	61.39	0.983	199.19	2.03
160	71.1	0.40	0.979	0.979	61.01	0.977	326.88	3.33
180	82.1	0.35	0.972	0.973	60.57	0.970	517.80	5.28
200	93.3	0.31	0.964	0.966	60.13	0.963	794.69	8.10
212	100.0	0.29	0.959	0.960	59.81	0.958	1013.25	10.33

Conversion Notes:

1 mbar	= 100 N/m^2
1 mm Hg	= 1.33322 mbar
1m H_2O	= 98.0665 mbar
1 lb/ft^3	= 16.0185 kg $/m^3$
1 m^3	= 1000 l
1 lbf/in^2	= 6894.76 N/m^2

Table 6.2

Pumping Efficiencies and Costs

Pumps	Efficiencies
Horizontal centrifugal	Medium size 80 to 82% perhaps 85% large size. Even higher with special construction but at higher price.
Vertical spindle shaft driven	Tending towards about 3% less than the horizontal centrifugal
Submersible	75 to 81% and can be lower to about 70% for small sizes. Generally about 3% less again than the vertical spindle pump, the reason being that the pump is restricted in diameter
Electric motors	
For horizontal pumps	93 to 95%. Fixed speed a.c. induction
For vertical pumps	90 to 94%. Fixed speed a.c. induction
For submersibles	85 to 89%. Less than the above because of the restrictions imposed on the design
Variable speed	About 3 to 5% less than with a squirrel cage a.c. motor

Overall Fuel Consumption and Cost

Electrical driven pumps	About 1.0 kW for every 0.75 kW of water power output; this implies an overall efficiency of about 75% which would be usual. Up to 1.3 kW per 0.75 kW water power output or higher for small pumps or variable speed pumps. (0.75 kW is approximately equivalent to 1 bhp in in British units since 746 watts = 1 bhp)
Diesel engines	0.21 kg of diesel fuel oil consumed per kWh of engine power exerted would be considered good (0.35 lb per bhp hour). 0.28 kg per kWH not unusually high. For lubricating oil add 5% to fuel oil cost.
Capital costs for complete electric driven pumping sets. (These figures include switchgear and station pipework)	Refer to Costing Section, P&D Department

Source: (Adapted from Twort, Water Supply, 1985

For this installation the required NPSH of the selected pump should not exceed, say, 8.0 m (allowing 0.87 m as a safety margin. It should be noted that the required NPSH increases as the capacity of the pump increases beyond the normal operating range (Fig 6.4)

Cavitation

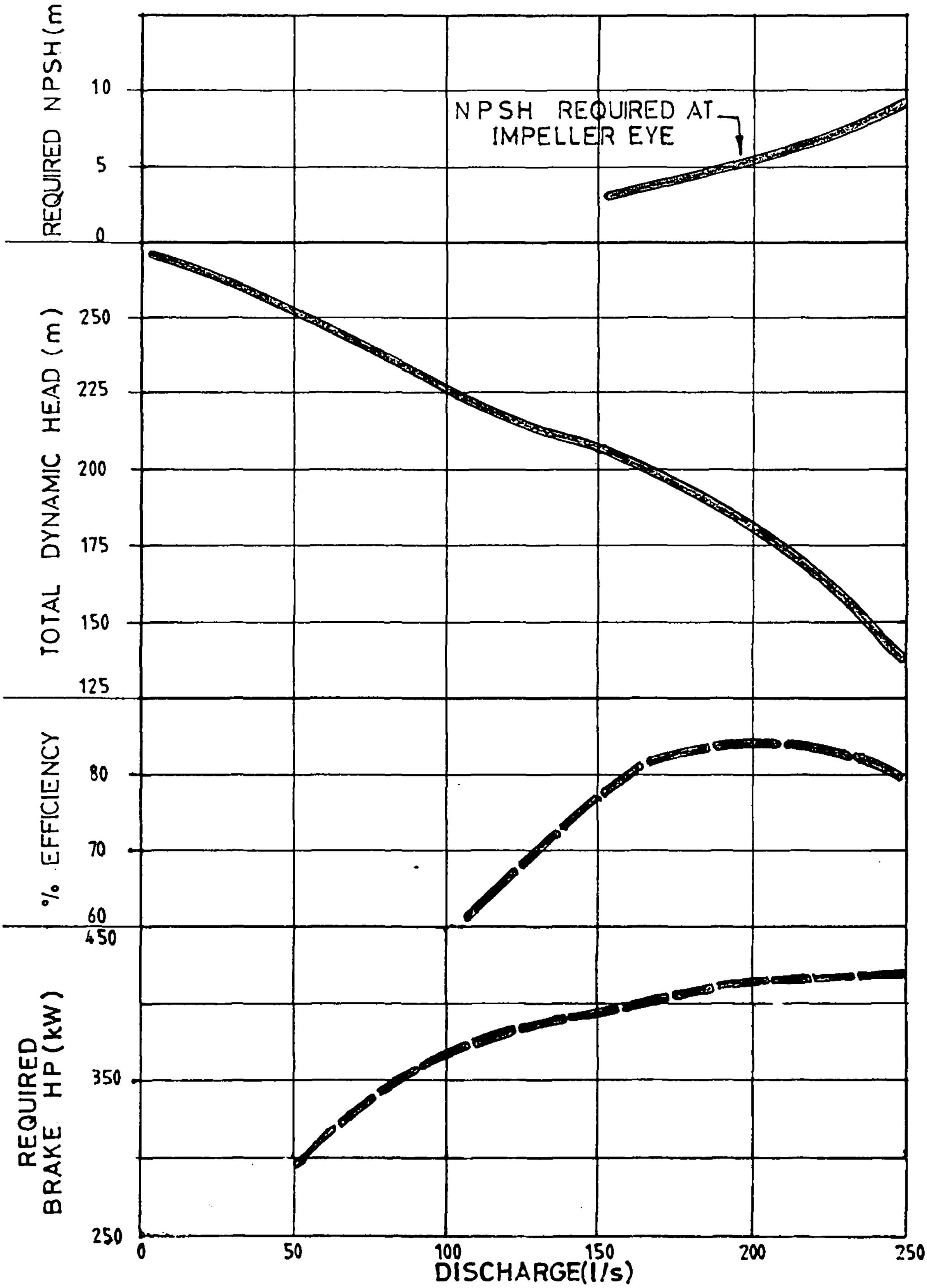
Cavitation is defined as the formation of cavities beneath the back surface of an impeller vane and the liquid normally in contact with it.

It can be caused in a centrifugal pump by

- the impeller vane travelling faster, at higher rpm than the liquid can keep up with it.
- by a restricted suction. Hence, never throttle the suction of a centrifugal pump.
- when the <u>required NPSH</u> is equal to or greater than the <u>available NPSH</u>.
- when the specific speed is too high for optimum design parameters.
- when the temperature of the liquid is too high for the suction conditions.

The cavity consists of a partial vacuum, gradually being filled with vapour as the liquid at the interface boils at the reduced pressure in the cavity. As the cavity moves along the underside of the vane towards the outer circumference of the impeller, the pressure in the surrounding liquid increases and the cavity collapses against the tip of the impeller vane with considerable force. A pump which is cavitating can usually be detected by the noise inside the casing, but this is not always the case. When a pump impeller is examined, the evidence that cavitation has occurred will be deep pitting and general erosion on the under-side of the vanes near the outer periphery. If a pump is cavitating due to a temporary upset in the system, it can sometimes be reduced by allowing a small amount of air to enter the pump suction or by throttling the pump discharge valve. These are temporary expedients, used only until the problem can be finally eliminated.

In order to prevent cavitation in centrifugal pumps, suction head should be carefully evaluated. Suction piping should be as short as possible and the velocity in the pipe should not exceed 1.5 m/s. Where strainers are used, the waterway area should not be less than 3 times the area of the suction pipe. Total pump suction which includes static lift as well as friction losses in the pipe and fittings should be considered carefully with due attention to pump speed, water temperature and site elevation. Refer to Annex I for more details of selection of the proper suction condition.



TYPICAL VERTICAL TURBINE CHARACTERISTIC CURVES

FIGURE 6.4

Intake sump

Intake sump designs have changed considerably during recent years. Complicated baffle wall designs are not favoured since they tend to cause vortexing; however, it is important to ensure that one side of the suction bell is close to one wall of the pump chamber, and that the bottom opening is reasonably close to the floor. Additional side clearance is necessary for vertical turbine pumps, particularly if they have a deep setting and small diameter columns, since the lower extremities of the pump column will gyrate. If this movement is restricted by rubbing against the wall, bearing problems may develop. See Figure 6.5.

Protective screens

Protective screens should always be provided whenever there is possibility of suspended or floating debris entering the pump suction. A ball of discharged electrician's insulation tape or a roll of plastic foil is particularly damaging to the bowl assembly of a multi-stage vertical turbine. However, wire screens, bolted or welded directly on to the suction bowl as protection devices are not recommended. They can cause serious suction problems if they become plugged and there is always the possibility that they may corrode, fail, and be drawn into the pump suction causing the damage they were designed to prevent (see Section 5.4)

Low level cut-out

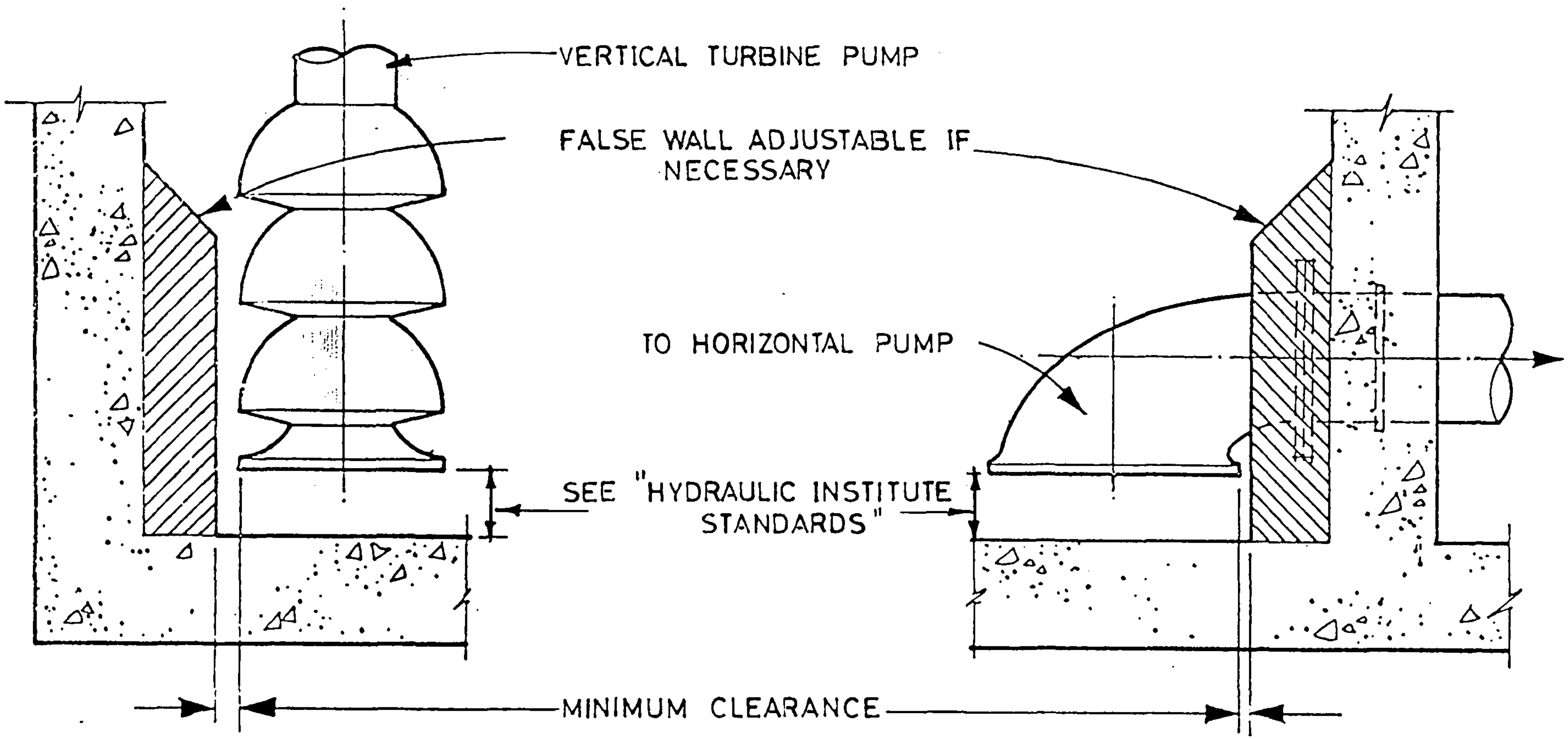
A low level cut-out switch should be installed on the pump side of the screen to stop the pumps when there is insufficient water in the pump wet well to provide adequate available NPSH.

6.2.3 Discharge conditions

The total discharge head of a pumping installation consists of:

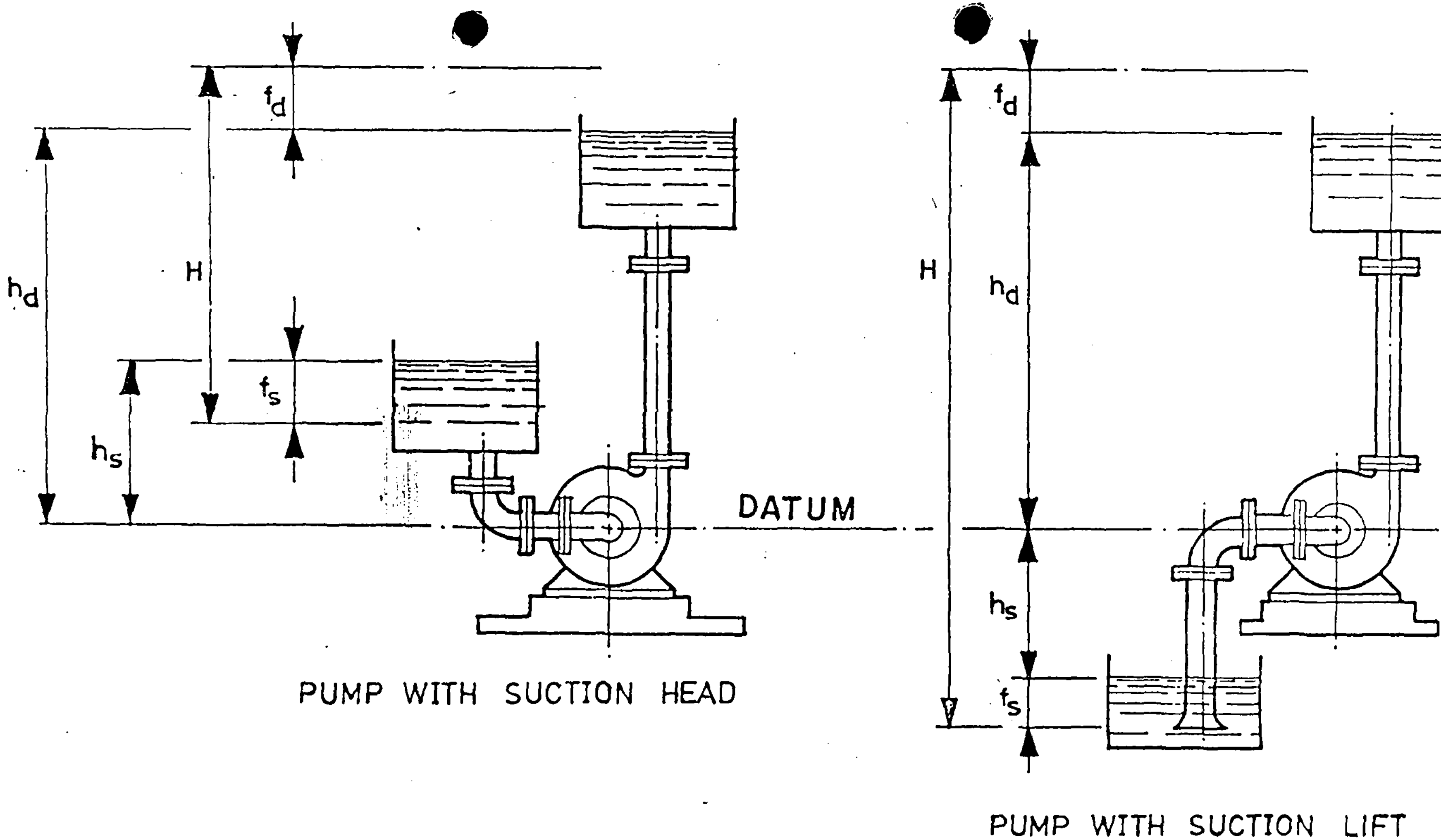
- o Static head or static lift.
- o Friction head or dynamic head

Static head: **This is measured from the surface of the liquid in the suction well to the surface of the liquid at the discharge reservoir.** See Figure 6.6. **Variations in terminal levels both at the suction well and at the discharge reservoir must be considered when calculating the upper and lower limits of the static head.**



PUMP INLET DESIGN

FIGURE 6.5



PUMP HEADS

FIGURE 6.6

Friction head: This is the head lost in overcoming pipe friction and depends upon the size of pipe, smoothness of the inside surface, the number and type of fittings, orifice plates and control valves, velocity of flow, viscosity, and density of the liquid.

Total head: The total head developed by the pump can be expressed by the following equation (see Figure 6.6)

Pump with suction lift -

$$H = h_d + h_s + f_d + f_s + \frac{V^2}{2g}$$

Pump with suction head -

$$H = h_d - h_s + f_d + f_s + \frac{V^2}{2g}$$

Where:-

H = Total head in metres of liquid pumped when operating at the desired capacity.

h_d = Static discharge head in metres, equal to the vertical distance between the pump datum and the surface of liquid in the discharge reservoir. The datum is taken from the shaft centre-line of horizontal centrifugal pumps or the entrance eye of the first stage impeller of vertical turbine pumps.

h_s = Static suction head or lift in metres equal to the vertical distance from the water surface to the pump datum. Notice that this value is positive when operating with a suction lift and negative when operating with a suction head.

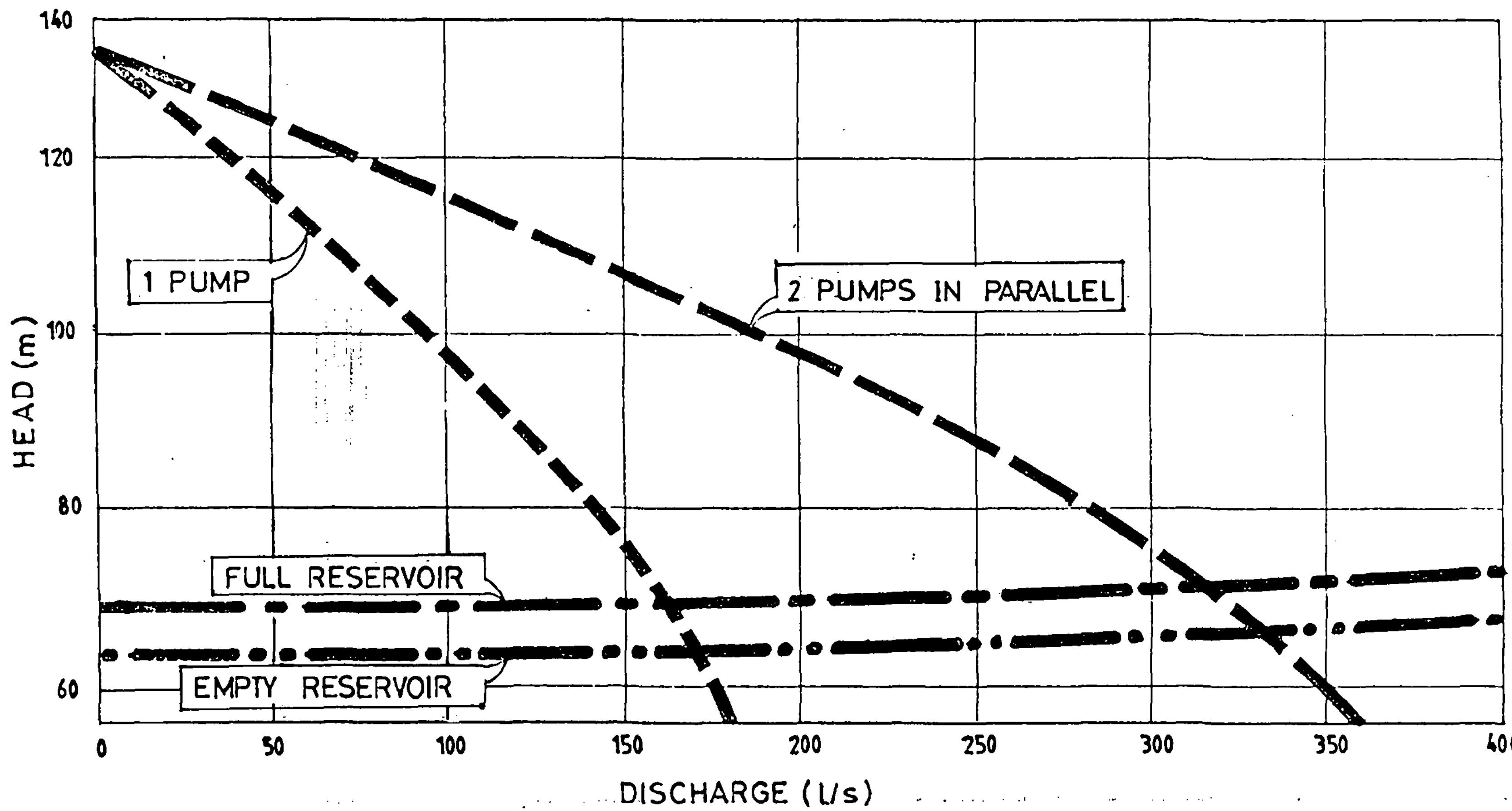
f_d = Friction head loss in the discharge piping measured in metres.

f_s = Friction head loss in the suction piping measured in metres.

$\frac{V^2}{2g}$ = Velocity head in metres as measured at the discharge pipe. For most pumping systems it is a small percentage of the total head and is usually erroneously neglected.

System head curve

The total discharge head is found from a system head curve, plotted for various conditions of flow. A typical system head curve with two pumps operating in parallel is shown in Figure 6.7. In this instance, the system head curve is very flat, since only a very small portion of the total head is due to friction, and the two pumps in parallel will deliver almost double the flow of a single pump.



SYSTEM HEAD CURVE
(WITH LOW FRICTION LOSS)

FIGURE 6.7

If, however, the system head curve is steep due to high friction losses (see Figure 6.8), the second pump operating in parallel with the first pump will deliver considerably less than double the original flow. The friction head loss in a pipe system is approximately proportional to the velocity squared, i.e. if the velocity is doubled, the head loss will be approximately four times the original value. This is illustrated in Figure 6.8 where one pump will deliver 220 l/s against a system head of 52 m. If, however, two pumps are in parallel, they will deliver 325 l/s (48% increase) against a system head of 90 m and will absorb approximately 360 kW compared to 140 kW for one pump only. This represents a 48% increase in flow for over 2.5 times the horsepower.

Pump discharge head

The pump discharge head can be specified in different ways depending on the particular design code.

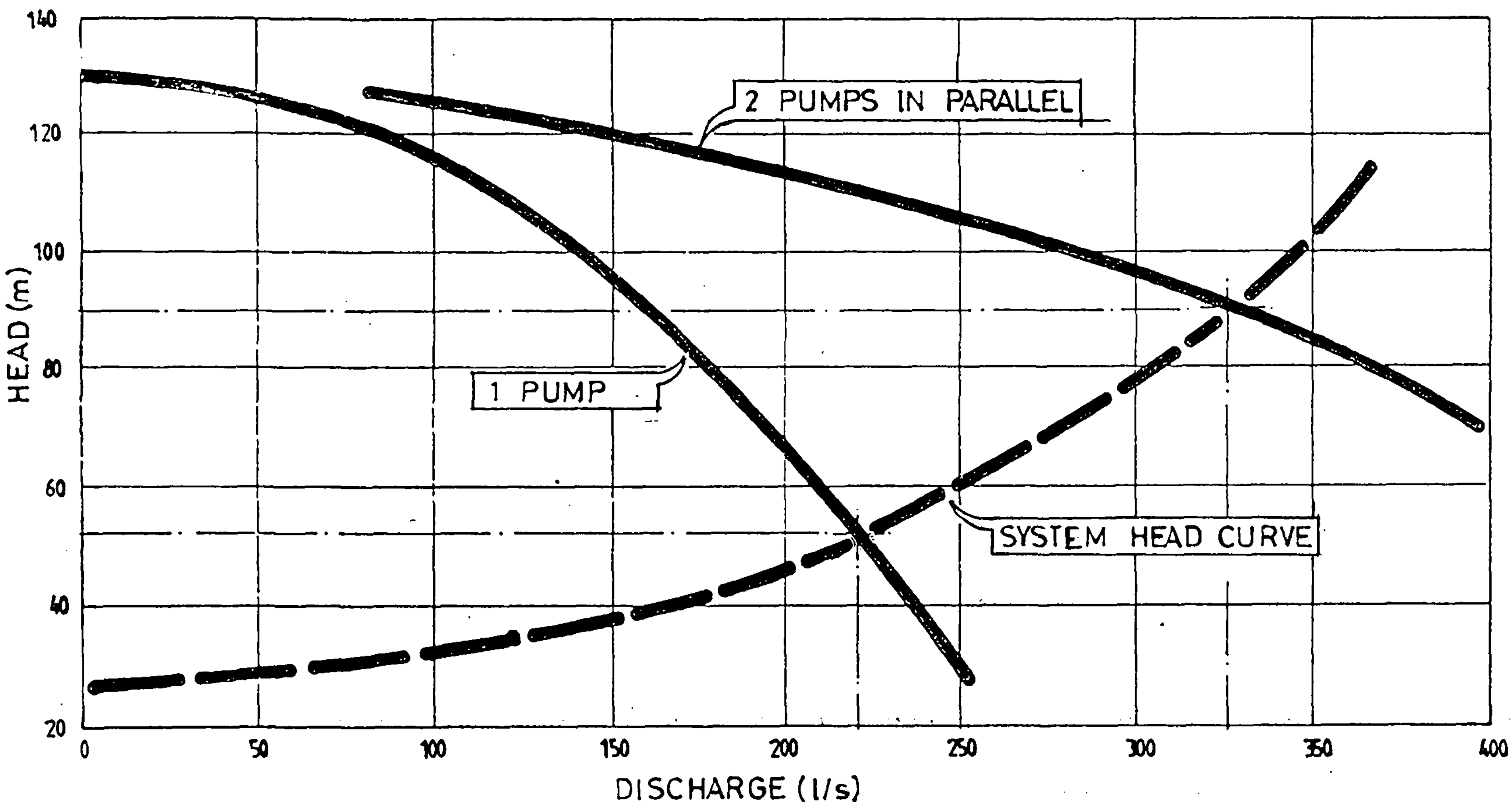
With horizontal centrifugal pumps, it is usual to define the Total Dynamic Head (TDH), as the difference between the elevation corresponding to the pressure at the discharge flange of the pump and the elevation corresponding to the vacuum or pressure at the suction flange of the pump, corrected to the same datum plane, plus the velocity head at the discharge flange of the pump, minus the velocity head at the suction flange of the pump.

Pumps in parallel and series

When pumping requirements vary, it is common to install one or more pumps in parallel rather than use a single large one. As the demand drops, one or more pumps may be shut down, allowing the others to operate at maximum efficiency. Moreover, this arrangement allows easy maintenance of each pump in turn. Pumps may be used in series when water has to be delivered at high heads.

Provided the pumps have the same operating characteristics, the total discharge of several pumps in parallel is the sum of the individual pump discharges: their shut-off heads should be about the same or some pumps will be ineffective. Careful analyses of the characteristics of pumps are necessary for efficient operation of pumps in parallel.

In the case of pumps operated in series, the total head for any flow is equal to the sum of the individual heads.



STEEP SYSTEM HEAD CURVE
(WITH HIGH FRICTION LOSS)

FIGURE 6.8

6.2.4 Shaft Speed

The developed head of a centrifugal pump is proportional to the peripheral speed of the impeller. A specific developed head can be obtained either by installing a larger diameter impeller and operating it at a lower speed, or by installing a smaller diameter impeller and operating it at higher speeds, so that the circumferential velocity in both cases is the same.

The smaller impeller is housed in a smaller casing or bowl assembly, and can therefore be built at a lower initial capital cost than the larger, slower speed unit. The faster the shaft revolves, the greater is the wear in the bearings, wear rings, neck bushings, sleeves, and stuffing boxes. It is reported that the rate of wear, and therefore the maintenance costs are proportional to the shaft speed squared, so that doubling the speed may result in four times the wear. Vibration becomes more pronounced in amplitude as the wear increases the running clearances, and since vibrations occur at greater frequency with higher shaft speeds, maintenance costs are higher than those of lower speed machines. If pumps are to be purchased on the basis of the lowest bidder, higher speed pumps will invariably be the result since they are less expensive to build (but more expensive to maintain). Shaft speeds are normally 1450 or 2900 rpm.

The most troublesome maintenance item of a centrifugal pump is the gland; whether a soft packed stuffing box or a mechanical seal, it will give far less trouble if it operates at a lower shaft speed.

There is obviously an optimum balance between lower capital and higher maintenance costs as opposed to higher capital and lower maintenance costs. With increasing inflation, maintenance costs are rising sharply, and therefore must be considered when selecting high speed pumping equipment.

Variable Speeds

Variable speed drives are becoming increasingly popular. The following pump characteristics are influenced by shaft speed:-

a) Flow (l/m) varies directly with (rpm)

b) Head (m) varies as the $(rpm)^2$

c) Horsepower (kW) varies as the $(rpm)^3$

To be able to reduce the rpm is analogous to having an impeller of variable diameter.

Initially, the only variable speed drives available were engines, steam and water turbines, or wound rotor motors. Later, induction motors with electro-magnetic drives became available and, in the last few years, solid state variable speed controls for standard squirrel cage induction motors have been developed. The later devices are limited in their range and reduce the full load speed by approximately 20%. However, in most cases, 20% is all that is required to reduce the speed of a pump motor since the developed head is proportional to speed squared. A reduction in rpm of 20% would result in a 36% reduction in head, which is usually all that is required to reduce the developed head to below the system head curve, with the result that the pump capacity, in relation to the system head curve, is reduced to zero.

The wound rotor motor, however, still has a place, particularly where engine driven generators are installed to provide power to operate the pumps during periods of failure of the normal power supply. The starting current of a standard induction motor can be up to six times the normal full load current. Even with low inrush current motors and reduced voltage starting, the starting current may be as much as three times the normal full load current. However, a wound rotor motor can be started at reduced speeds below full load current; consequently, the capacity of the engine-driven genrator can be reduced to a nominal full load capacity machine, eliminating the necessity to purchase additional horsepower for the sole purpose of providing starting current.

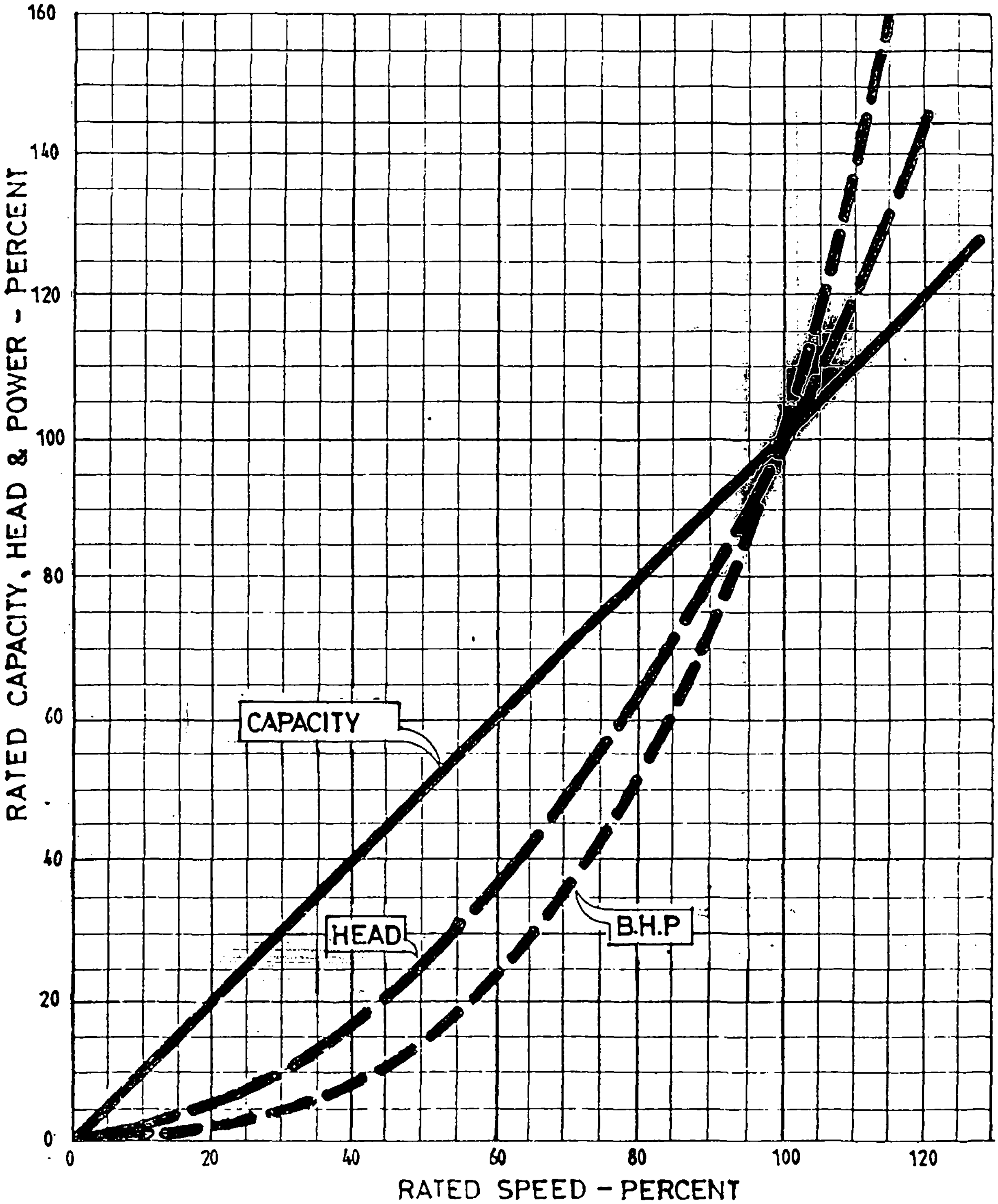
Affinity Laws

The relationships between flow (l/m), head (m), horsepower (kW), and shaft speed (rpm), defined above, are referred to as the "affinity laws," and are shown graphically on Figure 6.9.

6.2.5 Performance Curves

When selecting the most suitable pump from a number of supplier quotations, it is advisable to plot all the curves on the same scale. Figure 6.10 shows three typical H-Q curves from three different manufacturers. Curve A has preferred characteristics to B or C, since it has a lower shut-off head and generally better performance characteristics to the right of the rated point. This pump was selected and, when it was factory tested, the curve shown as AA on Figure 6.10 was the actual certified performance curve.

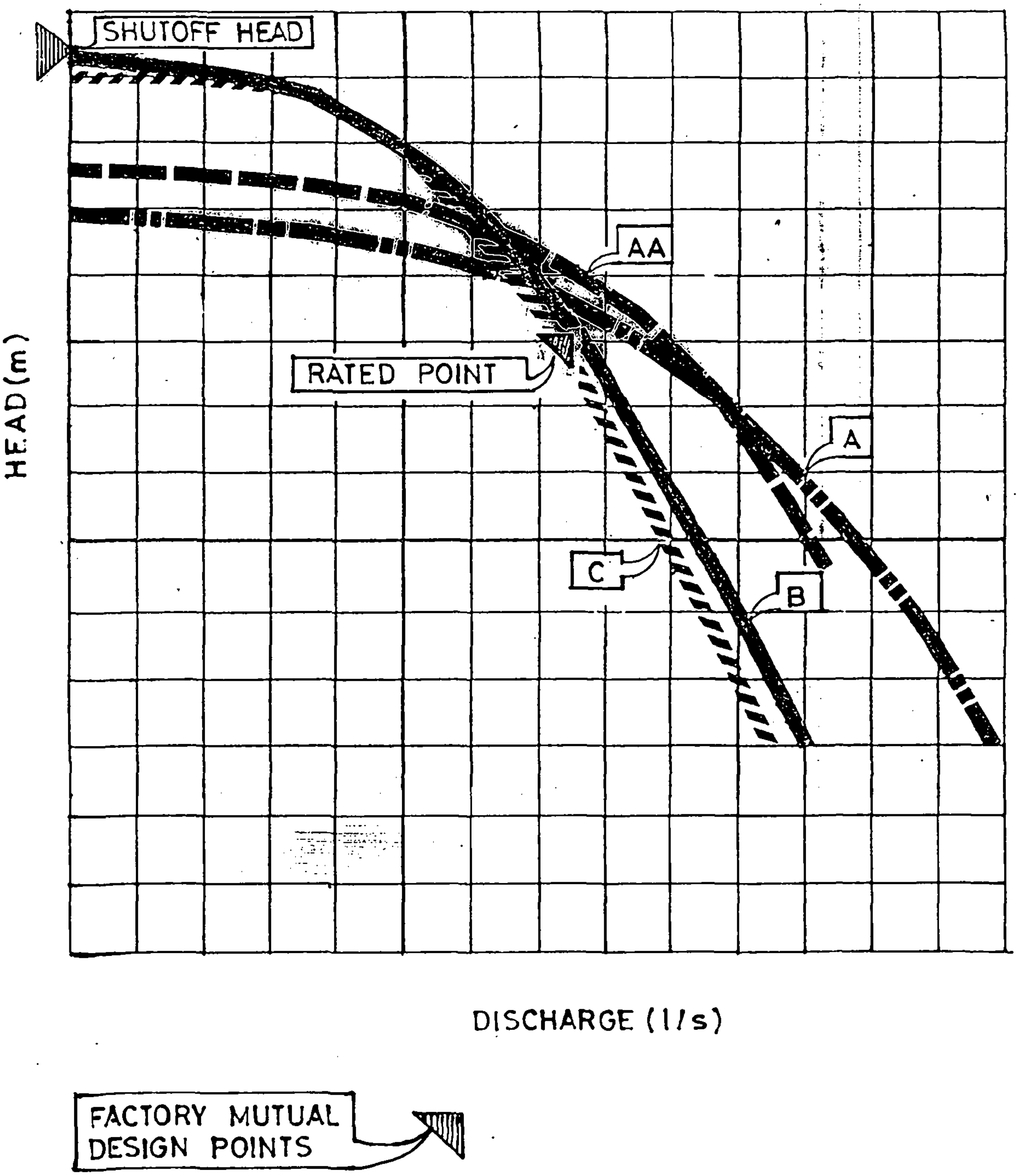
It must be emphasized that published characteristics performance curves are at best only good approximations, and a manufacturer will guarantee only one or two points on the H-Q curve generally in the range of maximum efficiency.



AFFINITY LAWS

CHART SHOWING EFFECT OF SPEED
CHANGE ON CENTRIFUGAL PUMP PERFORMANCE

FIGURE 6.9



THREE TYPICAL PUMP CURVES

FIGURE 6.10

It is difficult to test a pump on site in complete accordance with the requirements of the standard factory test. Therefore, the only way in which NWSDB can be certain that it is getting the performance specified is to witness the factory test and make certain that the pumps are tested in accordance with standard requirements and that they are capable of producing the performance characteristics quoted in the specifications. If the witnessed factory tests do not come up to the standards anticipated, it is usually possible to polish the impellers or the diffuser vanes in the casing, until the required results are obtained. If the published characteristics cannot be obtained by modifications to the impeller or casings, the manufacturer should reimburse NWSDB accordingly. If it is inconvenient for an NWSDB representative to witness the test, NWSDB should insist on a signed certified performance curve giving the necessary data obtained under test bed conditions.

Efficiency

The efficiency of a large installation can be a critical factor when choosing the optimum pumping units from a number of competitive quotations. This can be illustrated by the following example.

A municipal pumping station is to house three vertical turbine pumps, each capable of pumping 190 l/s against a 60 m head. Three pumps will operate together in parallel for 50% of the year, and two pumps in parallel for the remaining time. The average power cost for the whole station can be taken at approximately Rs.1.45 * per kWh. Calculate the monetary value of one percentage point of efficiency.

From a brief look at a number of catalogues, it would appear that most manufacturers have pumps capable of this performance at efficiencies ranging from 83 to 87%. Using 85% as an efficiency datum:

$$\text{Horsepower kW} = \frac{9.797}{1000} \cdot \frac{190.60}{0.85}$$

$$= 131.4 \text{ kW}$$

Efficiency of a 150 kW induction motor will be approximately 92% plus a further 2% for transformer and switchgear losses, so that the overall electrical efficiency will be approximately 90% (Note that power factor is not allowed for in this example).

$$\text{Kilowatts per pumping unit} = \frac{131.4}{0.9} = 146.0 \text{ kW}$$

---

* Excludes fuel adjustment charge, see CEB tariff Annex K.

Total hours per year (365 days) = 8760 hours

For 3 pumps in operation for 50% of the time:

$$3 \times 8760 \times 0.5 \times 146 = 1.92 \times 10^6 \text{ kWh}$$

For 2 pumps in operation for 50% of the time:

$$2 \times 8760 \times \frac{50}{100} \times 148 = 1.28 \times 10^6 \text{ kWh}$$

Total power consumption = $(1.92 + 1.28) \times 10^6)$

$$= 3.20 \times 10^6 \text{ kWh per year}$$

Annual power costs at Rs.1.45 per kWh = Rs.4.64 million per year.

From a similar calculation, but using a pump efficiency of 84% instead of 85%, the annual power cost would amount to Rs.4.68 million, an increase of Rs.40,000 per year, which is approximately equivalent to a capital expenditure now of Rs.400,000 based on an interest rate of 8% for 20 years (present value). This sum divided by 3 is equivalent to Rs.133,000 per pump per 10% of efficiency.

See table below for comparision of the other quotations in this example against the "datum" of 85% efficiency:

Quotation Number	Basic Unit Price Rs.million	Pump Efficiency (percent)	Bonus or Penalty Rs.million	Comparative Price Rs.million
1	2.105	85		2.105
2	1.957	84	+ 0.133	2.090
3	2.162	86	− 0.133	2.029
4	1.878	83½	+ 0.200	2.078
5	2.276	87	− 0.266	2.010

On the basis of comparative price, Quotation 5 is the "best buy." Provided the pumps are to be tested in accordance with recognised standards* and competently witnessed, there should be no difficulty in ensuring that the pumps are capable of their specified test bed performance. However, the question as to wheher or not they will continue to operate at the specified efficiency with the minimum of maintenance depends upon shaft speed, pump design, workmanship, materials of construction, column diameter, lubrication, bearings, and other features. Therefore, it is essential to ensure that, if a premium price has been paid for a high efficiency pump, the efficiency can be maintained at its peak performance throughout its economic life. Otherwise it would be better to recommend Quotation 3, or even Quotation 1. It is assumed that all other annual operating costs are the same for the five quotations.

Typical pump and motor efficiencies and costs are given in Table 6.2 and useful formulae in Table 6.3.

6.2.6 Operating Conditions

Pumps used in water and sewage pumping facilities are designed for continuous operation and do not need to rest, although their operators may do so. Identical pumps should be alternated, if not on each pumping cycle, at least weekly. This is not for purposes of rest but rather to equalise the wear and to be sure each pump is kept in running order. Where differenct sizes of pumps are involved, the proper size should be used to fit the pumping needs, regardless of whether it has just been running. Wet well size and pumping cycles should be selected to avoid any problems due to too frequent starts and stops.

The operating temperature of motors is normally about 10°C above ambient, depending on the effectiveness of ventilation, and therefore will normally in Sri Lanka run at about 40 – 45°C, which will feel hot to the touch.

* Such as Hydraulic Institute Standards of the US Hydraulic Institute.

Table 6.3

Pump & Motor Efficiency Formulae

Overall efficiency of pump/motor combination	=	$\frac{\text{Water horsepower}}{\text{Wire horsepower}}$
Water horsepower (whp)	=	$\frac{Q.H}{3300}$ (hp) where Q in gal/min, H in feet
	=	.1134 QH (W) Where Q in m^3/d, H in m
	=	9.797 QH (W) where Q in L/s, H in m
Wire horsepower	=	$V\ I\ \sqrt{3}\ \cos\phi$ (W)
Where Cos ϕ	=	Power factor (0.8 - 0.9)
V	=	volts
I	=	amps
Required BHP of pump (W)	=	whp ÷ pump efficiency
	=	$\frac{.1134.\ Q.H.}{(P.EFF)}$ where Q in m^3/d, H in m
Required motor hp (W)	=	$\frac{.1134.\ Q.H.}{(P.EFF).\ (M.EFF)}$
Power used (W)	=	$\frac{.1134.\ Q.H.}{(P.EFF).\ (M.EFF).\ (P.F)}$

Note

1) Power used in raising 1000 m^3/d by 8.81 m = 1 kW at 100% efficiency.

2) 1 hp = .7457 kW

3) P.EFF = Pump efficiency

M.EFF = Motor efficiency

The proper way to evaluate motor temperature is with a thermometer. If the ambient temperature is normal, the ventilation adequate, and the amperage within the allowable range, the motor should not overheat. One thing that can cause a motor to get hot is rapid cycling - being turned on and off too frequently. The excessive current required for starting the motor accumulates heat faster than it can be dissipated by the normal cooling system of the motor. For this reason, large motors are usually limited to about six cycles per hour or even less.

Pumping plant, therefore, should be designed on the basis of 24-hour pumping at the design (or stage) year, not 20 hour pumping with a rest period. This may be reduced, however, on the basis of operator shift periods where the pumping hours are limited due to cost or source constraints. Pump capacity should not exceed the yield of a groundwater source.

6.3 Structural Design

Take into account the following design considerations:

- purpose of building, functional requirements;
- external appearance and architectural design;
- area requirements for building, access and possible extension;
- site fencing and security; operator facilities;
- subsoil conditions, groundwater and existing services;
- vibration, noise, insulation;
- windows, lighting and ventilation, equipment access and removal;
- type of construction and materials - reinforced concrete, prestressed, insitu, precast, steel frame, brick or timber;
- type of foundations - piled, raft, strip footings, allow for sump and ducting;
- stability - settlement and overturning;
- structural materials specifications and quality - concrete grade, reinforcement, brick and mortar quality, grade of structural steel or timber;
- movement joints;
- finishes - cladding, floor, walls, roof, ceiling;

- o requirements for internal crane or equipment handling apparatus.

- o structural analysis for service and erection conditions;

- o design of individual elements - characteristic stresses will be as recommended by the applicable codes (see Annex B) and in general will be:

Concrete f_{cu}	=	20 MPa for normal structural concrete cube strength at 28 days.
		25 MPa for water retaining structural concrete
Reinforcing Steel f_y	=	250 MPa for mild steel.
		410 MPa for high tensile steel.
Structural Steel f_y	=	250 MPa.

Allowable soil bearing, pile, shear and friction values will be as determined by the results of soils investigations. The bearing on concrete piles will be determined by pile size and subsoil conditions. Loadings considered in the design of the structure should include the following:

- o dead weight of the structure;

- o dead weight of the pipes including weight of the contained water;

- o reactions due to hydraulic thrust and internal water pressure, including water hammer surge pressure;

- o hoisting and lifting loads;

- o mechanical and operating forces;

- o external earth pressure against structures;

- o foundation bearing pressure, differential settlement and/or hydrostatic loads;

- o traffic live load, where applicable;

- o seismic loads.

Uniform live loads should be established by the applicable codes. Concentrated loads, including concentrated erection loads, should govern the design where the resultant effect is greater than that of the uniform live loads. Concrete members subject to water pressure should be designed using service loads and permissible service load stresses for all conditions of loading.

Tanks subject to floatation when empty should be resisted from uplifting due to high groundwater level or recurring floods by dead weights of structures using a safety factor of 1.5 or protected with interior relief valves or under-drain systems to neutralise the effect of uplift.

A reasonable resistance to seismic loading without any significant extra cost may be by the use of reinforced concrete framework for super-structures. Connections for structural steel members should be bolted whenever necessary.

Every effort should be made to minimise the pumping head by careful location of structures.

Other criteria regarding building use and safety are as follows:

- stair criteria: maximum rise 200 mm; minimum rise 125 mm minimum run 230 mm; minimum tread width 250 mm
  run times rise, > 45,000 < 48,000;
- headroom for stairs, from outer edge of nosing, 2.05 m minimum, preferred 2.35 m based on 760 mm arm swing;
- vertical height between landings < 3.6 m;
- at least three risers for interior stairs in any flight;
- height of handrails for stairs between 800 and 900 mm; use 840 mm; maintain 40 mm clearance between handrail and wall;
- height of guard-rails for open-sided floor, working platforms, walkways - 1070 mm, with intermediate rail and toe-board or curb 100 mm high;
- use flights of stairs with landings for conventional pump stations, and vertical ladders for prefabricated stations.
- do not use fixed ladders in pitch range of 60° to 75° from horizontal;
- do not use cicular steel stairs with centre post;
- provide safety cages with vertical ladders;
- provide minimum of 0.6 m clearance on all sides of equipment where access or maintenance is needed;
- provide minimum clearance of 0.6 m behind and in front of floor mounted control panels;
- install floor drains or sumps with sump pumps adjacent to sources of spills in buildings, galleries or dry walls;
- for diesel or gasoline operated pumps locate motors and accessories away from sump to avoid contamination of supply;

- o slope building, gallery or dry well floors to floor drains;

- o do not drain seal water across floors; pick up at discharge point;

- o provide supports, concrete pads, under valves: remember that a 300 mm gate valve weighs 300 kg;

- o to determine size of access openings, make layout of unit in question; remember that bonnet of gate valve is larger than flange dimensions;

- o provide access openings in floors to take largest piece of equipment without disassembly;

- o provide space to move equipment horizontally to access opening, for removal;

- o make sure that you have means of removing any piece of equipment, i.e., dry type transformer hung from roof and inaccessible;

- o reinforce access opening gratings or cover in floors where trucks can operate, or use raised curb around opening;

- o seal all pipes and conduits going through common wall, wet well to dry well;

- o provide complete separation bwetween dry and wet wells;

- o design wet well wall for the maximum water level in the wet well obtainable, such as under power failure condition;

- o provide openings for pipes passing through walls, to be grouted in later. Do not count on pipe with its thrust plates being available at time of pour;

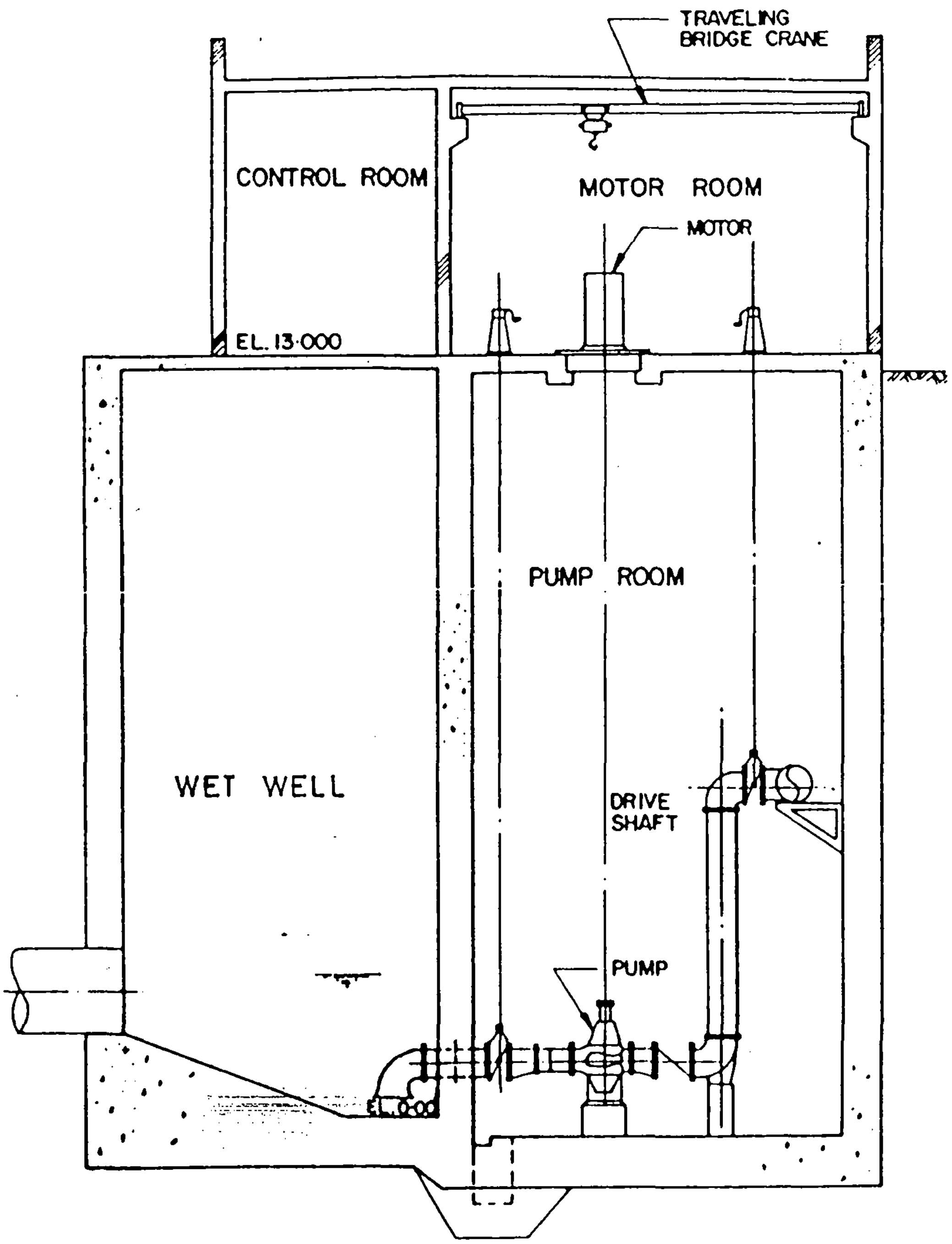
- o provide sleeves on conduit in walls for flexible type pipes;

- o do not use aluminium windows in chlorine rooms, the aluminium will be attacked by fumes unless painted;

- o use portable ladders in hazardous areas instead of man-hole steps, to discourage easy access.

A typical intake pump station is shown in Figure 6.11.

General references for structural design and detailing are Refs. (10) to (16).



TYPICAL INTAKE PUMP STATION

FIGURE 6.11

6.4 Equipment Considerations

6.4.1 Stand-by Pumps

All essential pumping systems should include stand-by pumps and motors to provide a reserve capacity of not less 30% as follows:

Duty pumps	1	2	3	4	5
Stand-by pumps	1	1	1	2	2

Greater reserve capacity, either connected or on hand, may be warranted for special types of pumps, in applications where failure is expected to be frequent, or where continuity of supply is especially important.

All essential pumping systems using electric motors should be provided with an alternative source of power, usually an engine driven generator. If engine driven pumps are used instead of generators as back-up for electric pumps, the system must be capable of operating without electric power supply.

All pumping systems should be investigated for potential problems of surge and be given adequate protection (see Section 6.5).

6.4.2 Design Details

Remember the following design details:

- o locate valves on suction and discharge sides of pumps for ease of working on, or removal;
- o arrange mechanical equipment (pumps, motors, blowers, etc.) where individual components weigh in excess of 45 kg to allow for means of mechanically lifting equipment for purposes of removal, installation, and/ or maintenance;
- o use flexible couplings for inlet and outlet pipes (if required) for ease of assembly, and removal. Consider flanged elbows for discharge pipes;
- o on dry sides of water retaining structures, provide solidly connected valves on pipe;
- o install flanged adapters on downstream side of watermain valves; do not put on upstream side;
- o provide built-in catch basins or channel grooves to contain any leaking oil from oil-filled stationary equipment (hydraulic controls, pumps, blower gear drives). Provide drain valves with safety plugs at units; pipe to drain pans.

- provide mechanical operators on valves mounted higher than 1.8 m above floor.

- use solid piping or equivalent on pump discharges;

- use joint harness across flexible couplings to take thrusts;

- do not expect flexible couplings to transmit vertical loads; they are not designed for this. Use supports on each side of coupling.

- remember that forged steel welding fittings, 350 mm and above, are O.D pipe size. When connecting to pump flanges, you will have 20 mm protrusion into flow. Consider fabricated steel fittings, with I.D. pipe size.

- do not use raised face steel flanges mating to cast iron flanges. Use flat face flanges, carbon steel bolts and full face non-metallic gaskets.

- consider mechanical method of lifting debris from bar screen raking when elevation is more than 900 mm below ground level. Always use debris dewatering platform.

- provide proper ventilation: for ventilation of pump stations, always force air into the wet well instead of exhausting it;

- beware of cross-connections; do not provide physical connection between potable water supply and sewage pump seals. Consider positive break with air gap and seal water pump;

- do not locate valves in wet well.

- provide pipe sleeves in floors, projecting 100 mm above floor, where pipes are not grouted in;

- provide curbs on which to mount equipment in areas where wash down operations will be carried out;

- protect electrical controls from splash during wash down operations;

- consider 25.4 mm diameter hosebib for washdown, with no more than 15 m of hose for washdown;

- do not use PVC pipe for above-ground air header.

- avoid use of light-weight sluice gates; they are inadequate for most intended services;

- provide guards on or around all mechanical equipment where operator may come in contact with belt drives, gears, chain drives, rotating shafting and the like;

- give elevation of cutoffs, start and alarms for level probe in wet well;

- coordinate between civil electrical and mechanical work;

- make sure that PRV or altitude valve is suitable for purpose.

A centrifugal pump should be started with a closed or restricted discharge valve. At shut-off head and zero flow, there is no danger of cavitation, and the pump is operating under stable conditions with minimum horspower and presumably adequate net positive suction head. A vertical turbine pump at shut-off head has maximum downthrust. However, once stable conditions have been established, the pump must not be permitted to continue to operate under shut-off head conditions for more than a few seconds, since heat will be generated in the bowl assembly or casings, equivalent to the horsepower input at shut-off head. There would be some cooling from the water in the wet well surrounding the bowl assembly; however, it would not be long before the water in the bowl would be boiling, the water in the column would be displaced by steam, and the shaft bearings and stuffing box or mechanical seal would run dry, resulting in considerable damage.

It is sometimes expedient to operate a pump for short periods at shut-off head, but there must be sufficient circulation of fluid through the pump impellers to remove the heat generated by the horsepower absorbed.

6.5 Surge Suppression

6.5.1 Causes of Surge

Any system containing a fluid which is set in motion through pipes or tunnels can experience pressure surges when the flow is varied. This can cause disturbances (ie. waves of pressure, velocity, stress and strain) to be propagated through the system from the point where the change is initiated.

These disturbances are known by various names, including pressure surges, pressure transients, fluid or hydraulic transients and water hammer.

In many types of installations, the pressure variations associated with the surges are not likely to be dangerous, but extreme caution in design and subsequent operation may be needed in other cases to ensure that the pipes, pumps, valves and other components are adquately protected against severe damage or complete failure.

Common causes of severe pressure surges include:

- rapid changes in valve settings;
- starting and stopping of pumps and turbines;
- governor hunting;
- effects of reciprocating pumps;
- filling of empty pipelines and hoses;
- rapid chemical reactions and thermal changes (eg. vapour generation) within a system;
- mechanical vibration of system components (eg. seals and guide vanes).

6.5.2 Methods of Surge Control

Undesirably large pressure variations arising from changed flow conditions in a pipe system normally occur because the change of flow is too rapid. Therefore, surge control devices should effectively reduce the rate of change of flow. Various methods of surge control are described briefly below:

Slow valve closure: This leads to lower pressures being generated, especially if a two-part closure is used, a slower rate being employed for the final 10-20%.

Increased pump inertia: To avoid the sudden change resulting from stopping a pump, the stopping time can be increased by fitting a flywheel to increase the inertia.

Surge shafts: If a sudden change in pressure is likely, a surge shaft may be provided into which the water flows, allowing it to come to rest more gradually.

Air vessels and accumulators: If the system is under considerable pressure, the height of the surge shaft required might be excessive. If, however, the liquid is allowed to enter a closed vessel partly filled with gas, the incoming liquid compresses the gas which gradually brings the liquid to rest. The term 'accumulator' is often applied to such devices where the gas is isolated from the liquid by a flexible membrane. Figure 6.12 (a) shows that an cessation of pumping, water initially flows out of the air vessel and surge shaft. On subseqent reversal of flow, water flows into the air vessel and surge shaft.

Air admission valves: If, after flow changes, the pressure in a pipeline is liable to drop to the vapour pressure of the liquid, undesirable vapourous cavities may form. To combat this, it is possible to fit valves which open and admit air to the system when the pressure drops below atmospheric, thus preventing it from dropping much further.

Relief valves: These can be set to open at a given pressure and allow liquid to escape from the system.

By-passes: By-passes can be fitted around a pump to admit water from the sump when the pump discharge pressure drops below sump pressure, or to allow water to by-pass a booster pump in a long pipeline when the booster pump fails.

Feed tanks: These may be used to reduce negative pressures when air admission is not acceptable.

Check valves: These valves are used on pump discharges to avoid reverse flow through, and rotation of pumps. They are also useful in some locations to reduce risks of large pressure rises.

6.5.3 Influence of Valves on Surges

The rate of closure of a valve is very important in determining surge pressure amplitudes. For example, if the closure time is shorter than the pipeline period the full surge of pressure may be developed. In keeping the pressure changes within acceptable limits, it is not enough to ensure that the closure time is slightly longer than the pipeline period, as a considerable proportion of the pressure change may still be developed. Charts are included in Ref.19 report to aid the choice of suitable closure times. These charts enable an estimated pressure rise to be obtained, though a computer solution is generally recommended.

If the closure time chosen in relation to surge suppression is unacceptably long (eg. leading to too much liquid being drained from the system), it may be possible to reduce the time by using two closure rates. Many types of valve do not have a great influence in changing the flow-rate until they are operated through a considerable proportion of their total travel.

With a computer, it is comparatively simple to optimise the mode of valve closure. It is also possible to design valve operating systems using cams, or pressure sensing devices, to minimise pressure surge. This is known as 'valve stroking'.

Valve closure times are frequently influenced by conflicting requirements. For example, with controlled closure pump discharge valves, too fast a closure leads to column separation, and too slow a valve closure, in the absence of a check valve, leads to high reverse flows and reverse pump rotation. To calculate pressure changes for too slow valve closures, several approximate formulae have been proposed, but it is well known that they frequently lead to considerable error. To overcome this difficulty to some extent, a selection of charts for various types of valve are reproduced in Ref. 19.

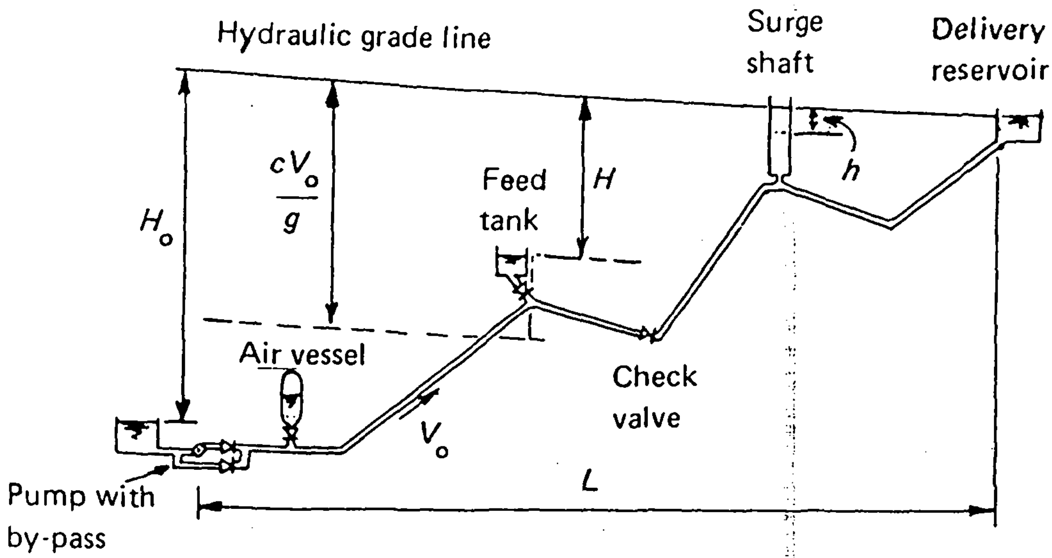
If the pressure in a pipeline under surge conditions is falling to vapour pressure causing separation of the liquid and later recombining with a subsequent high pressure rise, it may be acceptable to place air valves at appropriate points which open when the line pressure starts to fall below atmospheric. These admit air which cushions the blow as the separated liquid columns recombine. The valves must be carefully designed to allow sufficient air into the line and yet to ensure that it is not expelled too rapidly.

In potable water supply schemes, air admission is generally undesirable because, if the pressure is allowed to drop to atmospheric or below, contamination may enter the line either through joints in the pipe or the air valves.

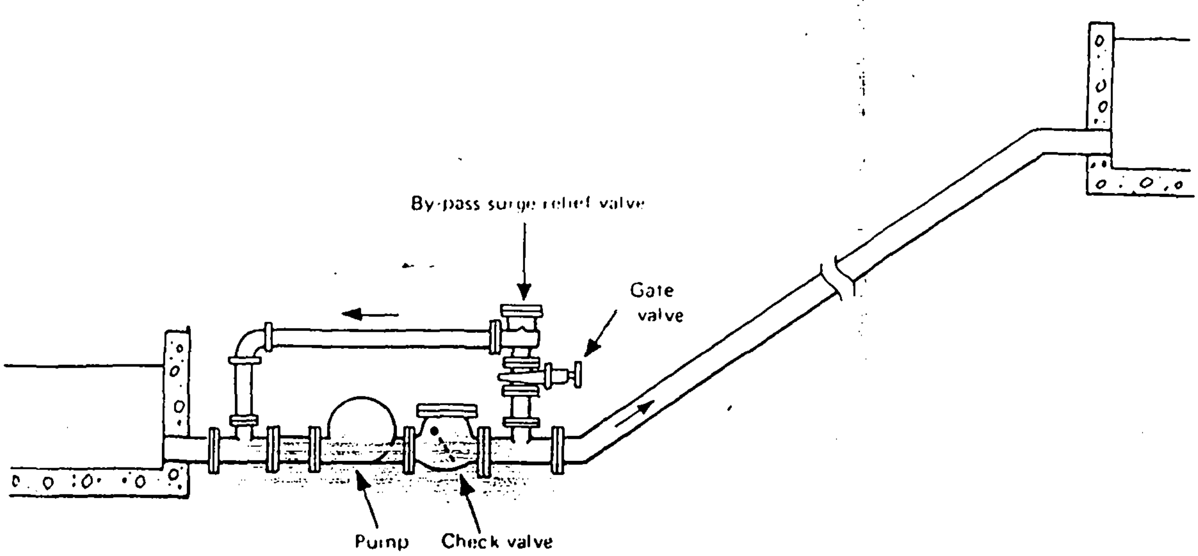
In some circumstances, a suitable alternative to surge shafts and air vessels for limiting pressure rises are relief valves (see Figure 6.12 (b), 6.13.). They can be of the conventional spring- loaded type or controlled by a hydraulically operated pilot valve. The essential features are a rapid opening (ie. much less than a pipeline period), followed usually by a slow closure. They should also be of an adequate size and number to pass the full flow allowing for valve isolation for maintenance purposes. It is often convenient to use two or more valves, set to open at slightly different pressures.

An important attribute of this type of valve is that it opens only as far as necessary to control the pressure, thereby keeping the system pressure relatively stable. A variation on the conventional relief valve is to incorporate sensing devices to cause the valves to open in anticipation of a pressure surge.

Discharge pipelines on deep-well pump installations are protected against high start-up pressures by the use of air release valves and pressure regulating valves (PRV). As well as providing surge protection, the PRV allows the discharge to waste of discoloured water from the pump riser.



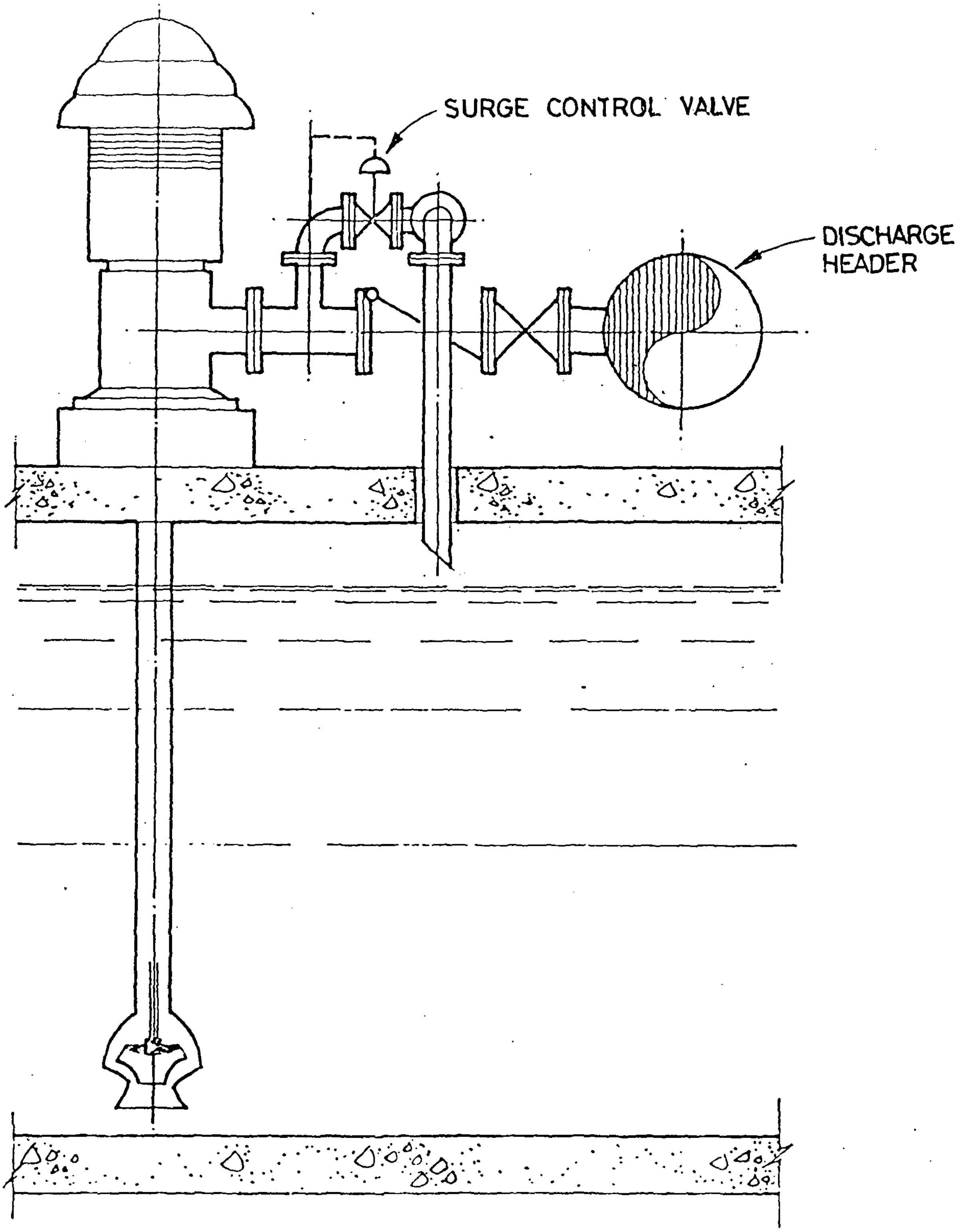
PIPELINE PROFILE SHOWING SUITABLE LOCATIONS FOR SURGE PROTECTION DEVICES



TYPICAL PUMPING STATION WITH BYPASS SURGE RELIEF VALVE

SURGE PROTECTION DEVICES

FIGURE 6.12



SURGE CONTROL VALVE

FIGURE 6.13

To sum up, it is best, whenever possible, to avoid rapid changes of flows in pipeline systems by using suitable valve closure times or increasing the inertia of pumps by fitting flywhweels. If rapid changes cannot be avoided, flow can be diverted into or drawn from open shafts or closed vessels. If none of these possibilities is feasible, it may be possible to limit pressures in certain circumstances by the use of air inlet valves, relief valves or by-passes.

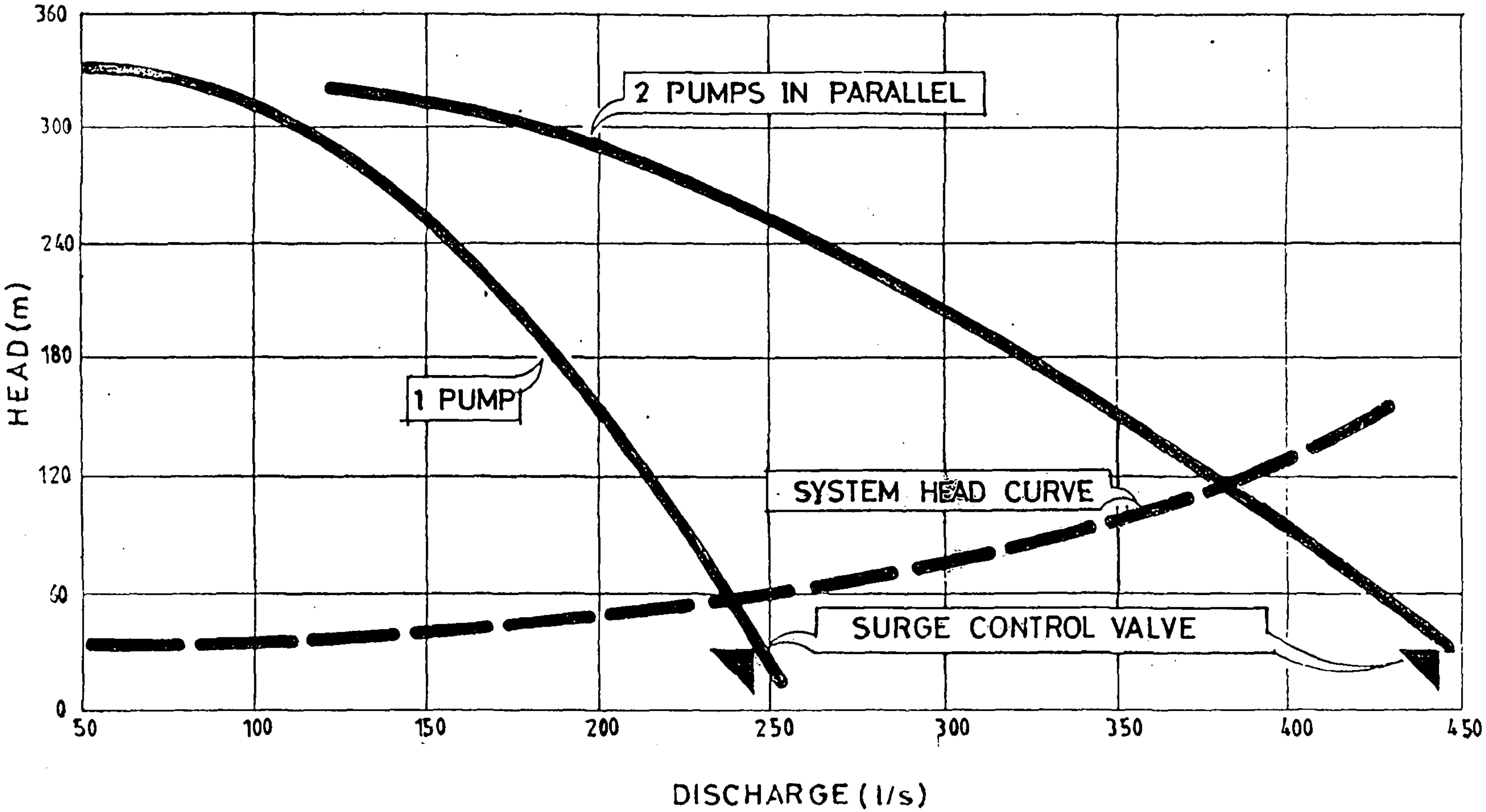
Surge shafts are most commonly used for hydro-electric installations and tunnel mains, air vessels or pump flywhweels for large-scale pumping schemes and accumulators in fluid power applications. However, any method of surge alleviation can be used if it is practical, economic and given adequate protection.

Site investigations of surge conditions are highly desirable as part of the normal commissioning process. They serve two purposes: provision of a comparison with the theoretical predictions made previously, which can be revised if necessary, and provision of more reliable data for future surge studies if, for example, the capacity of the system is likely to be increased at a later stage.

The most convenient way of obtaining data on time-varying conditions is to record experimental results directly as pressure-time histories at one or more units in the system. A basic measuring and recording system comprises three parts: a transducer for sensing pressure (and changes thereof) and converting it into, or varying an electric signal; signal-conditioning equipment for processing the transducer output into a suitable form, usually a voltage signal from the conditioning equipment against time.

6.5.4 Surge Control Valves

The surge control valve is often suitable for relatively small pumping applications. The valve is fully open when the pump is started, and passes sufficient flow in the fully open position to prevent the developed head from reaching the system head curve requirements. An example of valve setting may be seen from Figure 6.14. In this case, two pumps are designed to operate in parallel against a rising system head curve. The surge control valve should be designed to pass 250 l/s at 30 m head. The pump will be operating in the stable operating range, although at the opposite end from the shut-off head position. Providing the suction conditions are correctly designed and the pump is operating within the limits of the required NPSH, there should be no danger of cavitation. As soon as the pump is running at full speed, the surge control valve slowly closes and the developed head increases until it reaches the point of intersection with the system head curve. In this way the pump goes on line with the minimum of surge and water hammer.



SURGE CONTROL VALVE SETTING

FIGURE 6.14

Likewise, the surge control valve must be slowly opened prior to stopping the pump. The time to close the surge control valve when the pump is starting, and the time to open the valve when the pump is stopping must exceed the time required for the reflection of the shock wave.

equal to $\frac{2L}{C}$ seconds

where L = length of pipe (m); and

C = celerity (velocity) of the shock wave, usually 1200–1300 m/s.

When a vertical turbine pump is stopped, the water in the column will tend to drop to the level in the wet well. This will occur if air can enter the pump through the surge control valve, or if the water level is below 9 m. If the water level in the well is deeper than approximately 9 m below the ground surface, the water level in the column will fall until it can be supported by the prevailing atmospheric pressure. If it is a deep well where the level of the aquifer is 30 m or more below the pump discharge head, there will be a partial vacuum in the top 20 m of column, assuming that the stuffing box or mechanical seal is bottle tight and the check valve does not leak. When the pump starts, it momentarily discharges against zero head until it has pumped sufficient water to fill the column. Under these conditions, the pump will be operating off the head-capacity curve at the extreme right-hand end, with the result that the motor may stop on overload before sufficient head has been developed to reach stable pumping conditions. This is frequently a fault with the installation of deep well submersible pumps.

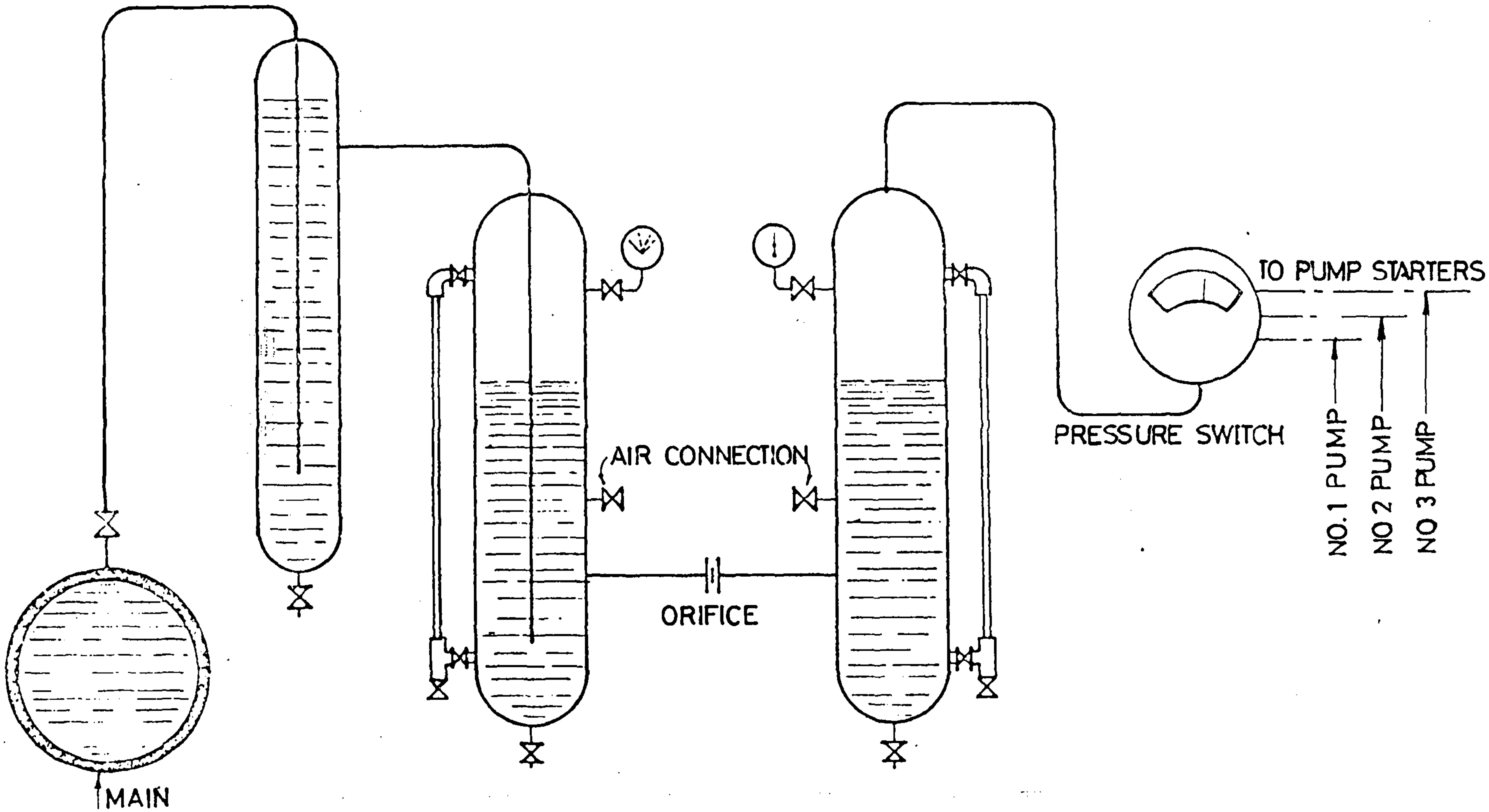
If a partial vacuum exists in the top section of a pump column, a considerable pressure surge can develop when the pump starts and the water in the column hits the underside of the pump discharge head. The pressures resulting from these shock waves exist for only fractions of a second, but they have been known to crack the pump discharge head castings. Allowing air to enter the column through a surge control valve does tend to cushion the shock wave and assists in building up the developed head, thus reducing the problems associated with starting pumps against zero head conditions. The air admitted to the pump column is discharged to atmosphere through the surge control valve, but it is important to ensure that the surge control valve is open long enough for all the air to escape before it closes and the pump discharges to the system. The venting of air can be noisy, particularly with large high head pumps.

6.5.5 Hydraulic Surge Suppressors

Pumps are frequently operated to maintain a municipal water supply system within prescribed limits of pressure. For example, a pump station consisting of two or more pumps may be required to maintain a certain level of water in an elevated reservoir several km away from the pump station. The distribution system will probably be connected to the main between the pump station and the elevated storage. The pumps are usually operated to maintain a prescribed water level in the elevated storage tank by sensing the pressure in the pump discharge header at the pump station. The pressures in the pump header are influenced by demand changes in the distribution system, and the pumps are frequently started and stopped in response to surge pressures rather than level changes in the elevated tank. To reduce these fluctuations at the pressure switch in the pump station, a surge suppressor can be located between the main and the pressure switch. (See Figure 6.15) This will usually reduce the frequency and amplitude of the surge sufficiently to sense the level changes in the elevated tank.

Tables for water hammer evaluation are given in Annex J, together with further references. The material in this section was partly extracted from an article by Gray in World Water, November 1979.

Further references are (19), (20), (21).



HYDRAULIC SURGE SUPPRESSOR

FIGURE 6.15

7. PIPELINES & VALVES

7.1 Pipe Materials & Joints

Pipe materials available for waterworks use are as follows:

Ductile iron
Steel
Asbestos Cement
Concrete
Plastic

An ideal pipe material should possess the following attributes:

- o durability;
- o resistance to internal and external corrosion;
- o a smooth internal surface which will not deteriorate, or contaminate the water carried;
- o capability to withstand internal pressures and external loadings;
- o ease of handling, cutting and laying;
- o ease and reliability of jointing;
- o for distribution mains, ease of tapping;
- o economical in purchase and maintenance

Unfortunately no single type of pipe material is able to meet these requirements, and a choice must be made depending on the application, working pressures, laying environment and construction experience. Some factors are discussed below:

Corrosion: some waters react with the pipe material due to an imbalance in the chemical make-up of the water or to minerals in it. This can result in corrosion of metal pipes, leaching of cement in concrete pipes, or deposits of minerals which reduce the water flow in all types of pipe. Chemical analysis of the water to be carried should point to any likely problems.

Flow: pipe materials vary in smoothness which affects their resistance to the flow of water. The rougher the surface, the more energy is required to move water from one point to another, thus increasing operating costs. These losses are compounded as the rate of flow increases.

Water pressure: pipe materials vary greatly in their recommended working pressure ratings. It is therefore important that the pressure at all points in the system be known, under static and surge conditions.

Soil characteristics: soils can react with pipe materials under some conditions and soil chemical testing or resistivity surveys should be carried out if problems are anticipated. Other problems include rocks or boulders which might damage the pipe, swamps or bogs which do not provide adequate support, and sand which can shift and expose the pipe. Important physical properties of the pipe material include resistance to crushing, degree of stiffness, and reaction to temperature changes, exposure to ultraviolet rays and chemicals.

Physical characteristics of pipe: resistance to impact, such as due to a rock falling on the pipe or the pipe being dropped, is important. The stiffness or flexibility of the material indicates how it will react to impacts. Pipe materials which are inflexible include concrete, asbestos cement and cast iron. Care must be taken to prevent unintentional bridging of inflexible materials due to uneven bedding. Steel and ductile iron pipe is moderately flexible, particularly in smaller diameters. Plastic pipes are usually quite flexible.

Heat and sunlight: usually affect only plastic pipe. Plastic has a relatively high expansion/contraction factor when exposed to variations in temperature. For this reason, polyethylene pipe should be "snaked" in the trench. Ultraviolet rays in sunlight can cause deterioration in plastic pipe so it should not be exposed for long periods of time. The weight of the pipe is an important consideration for transport, handling and laying. Toxicity is a potential problem if recycled plastic products are used in pipe manufacture, if toxic materials such as lead are used, or if the pipe is contaminated by prior use or improper storage.

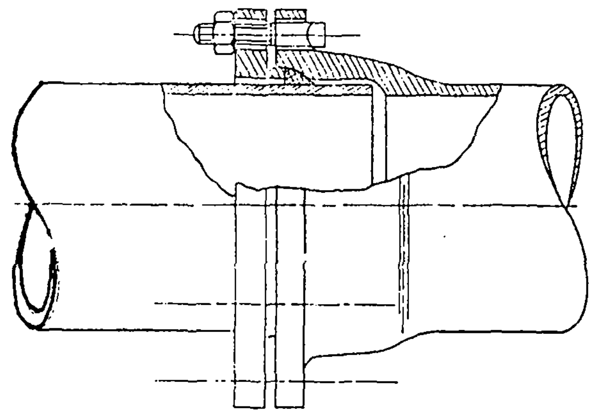
7.1.1 Ductile Iron

This has replaced cast or grey iron pipe, being stronger, more flexible and less liable to fracturing. Although fairly resistant to corrosion, internal and external protective coatings with bitumen or coal tar are standard. For aggressive waters or soils, further protection such as cement mortar or epoxy resin internal lining and external bituminous sheathing, protective tape or polythene sleeving may be necessary, and there are various standard specifications for these.

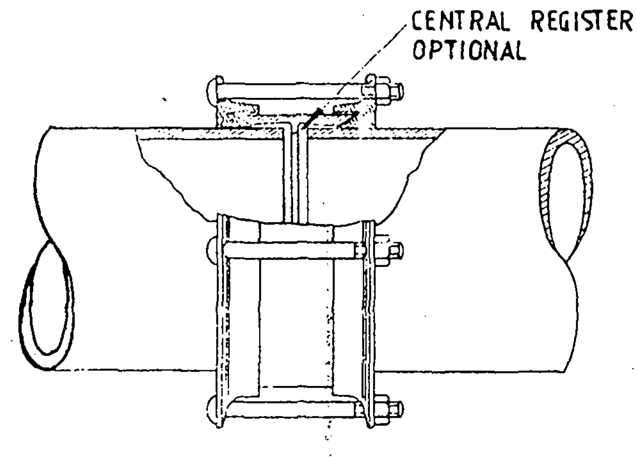
The pipe is available in sizes from 80 mm up to 2600 mm in standard lengths of 5.5 m in UK, and up to 8 m elsewhere. Pipes usually have spigot and socket joints, but flanged or plain-ended pipes are available. Types of joints are shown in Fig. 7.1.

Classes of pipe and pressure ratings are given in Table 7.1. Ductile pipe is generally suitable for all pipeline applications, particularly in poor laying conditions or when extra impact-resistance is required. It is relatively expensive.

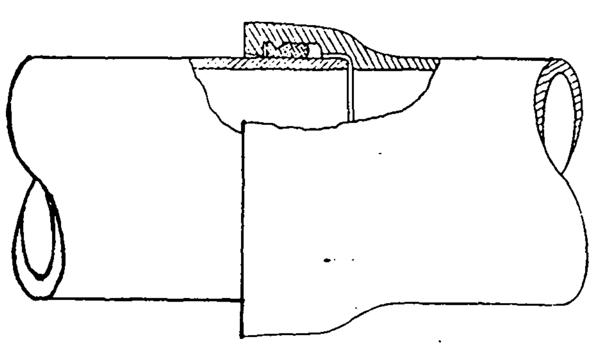
A full range of ductile iron fittings are available, and service pipe connections may readily be tapped into the main. (Ref: BS 8010, CP 2010 Part 3, BS 4772, ISO 2531, AWWA C110 etc.)



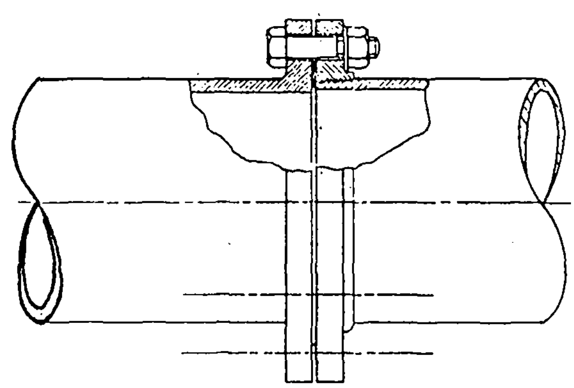
BOLTED GLAND JOINT



SLIP ON COUPLING JOINT
(eg:VIKING JOHNSON OR DRESSER COUPLING)



PUSH IN JOINT
(eg: TYTON JOINT.)



FLANGED JOINT

JOINTS FOR DUCTILE PIPES

FIGURE 7.1

Table 7.1

Pressure Ratings of Ductile Iron Pipe

Diameter (mm)	Works Hydraulic Test Pressure		'Pressure Rating' i.e. maximum sustained operating pressure on pipeline CP 2010 (bars)	Maximum Field Hydrostatic Pressure on Pipeline CP 2010 (bars)
	For pipes: ISO 2531, BS 4772 (bars)	For fittings BS 4772 only (bars)		
80– 300	50	25	40	45
350– 600	40	16	25	30
700–1000	32	10	16	21
1100–1200	25	10	16	21
1300–2000	25	–	–	–

Notes:
(1) Figures for works test pressures of fittings are lower than for pipes because of the need to avoid distortion during test.

(2) As Table 2 of CP 2010 Part 3. Also maximum sustained operating pressure plus surge must not exceed 1.1 times pressure rating.

(3) As Table 6 of CP 2010 Part 3. Pressure is measured at the lowest point of the pipeline. The actual test pressure applied can be 5 bar in excess of the actual sustained operating pressure, if this is less than the figures shown.

It will be noted that BS 4772 covers pipe only up to 1200 mm size: ISO 2531 extends the range up to 2000 mm, but does not specify pressure rating and field test pressure.
1 bar = 10.17 m head of water.

(Source: Ref. 25

7.1.2 Steel

Steel pipe is strong and lightweight and available in any thickness, size or length. The wall thickness is designed for a particular application depending on internal pressures and external loading and bedding conditions. There are no standard classes, though CP 2010 Part 2 defines the minimum thickness, t, (mm) as:

$$t = \frac{pD}{2afe} \quad (mm)$$

where p = internal pressure (N/mm²)

D = outside diameter (mm)

a = design safety factor (usually 0.5)

f = specified minimum yield stress of steel (N/mm^2)

e = joint factor (normally 1.0 (See CP 2010))

Internal and external corrosion protective linings (of types as for ductile pipe) are very important, should be applied to clean pipe with great care, and made continuous at the joints after laying, both inside and outside the pipe. Usually, the finished lining should be tested with a holiday detector for discontinuities prior to back filling.

Pipes are usually welded together at sleeve joints, after covering in the trench. Flanged joints are also used (see Fig. 7.2). Normally, steel pipes are used for major high pressure transmission pipelines in larger diameters, which allow internal access for joint welding and completion of protective lining. They are not suitable for distribution system use.

(Ref: CP 2010 Part 2, BS 534, AWWA C 206 etc.)

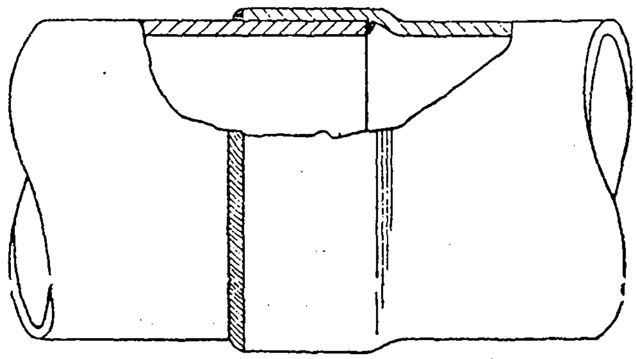
7.1.3 Asbestos Cement

Asbestos cement pipe is made of asbestos fibres mixed with cement and silica. The pipe is available in 3 and 4m lengths and in diameters of 50–900 mm. Working pressure ratings are 75, 100 & 125 m depending on wall thickness. Asbestos cement is commonly available and moderately easy to install, tap and repair in diameters of 150 mm or less. The pipe walls are smooth and resistant to corrosion, except in environments aggressive to cement (sulphated soils).

The principal disadvantage of asbestos cement is its stiffness. It must be handled with care or it will break. It also must be installed with care, often with a select backfill and bedding material. Bridging must not be allowed to occur. Although asbestos cement is resistant to corrosion, highly aggressive water can leach out the cement and expose and release asbestos fibres which may be ingested by drinking the water.

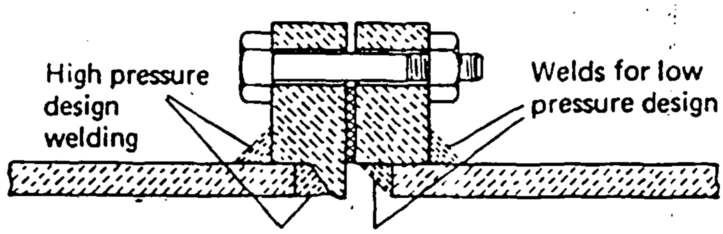
This aspect has recently (1986) led to prohibition of the use of AC pipe in the USA, although they continue to be used widely elsewhere, and there seems to be no justifiable reason for prohibiting their use except in areas where water supplies are naturally aggressive. Pipes are usually protected with a thin bitumen coating.

Pipes are plain ended and jointed with an AC sleeve sealed by rubber rings. Fittings are usually of grey or ductile iron. AC pipe is suitable for distribution pipelines and for gravity and pumping mains where normal operating pressures and surge pressures are not excessive, and in controlled conditions of transport handling and laying.



NOTE. For small diameter pipes the internal weld may be omitted.

SLEEVE WELDED JOINT



FLANGED JOINT

NOTE: THE SLIP-ON COUPLING TYPE JOINT MAY ALSO BE USED

JOINTS FOR STEEL PIPES

FIGURE 7.2

They may be cheaper than ductile iron pipe, and are more suitable for laying in aggressive ground conditions. Saddles are required when making service pipe connections.

(Ref CP 2010 Part 4 and BS 486)).

7.1.4 Concrete

These are either prestressed or reinforced concrete cylinder pipes, in which the wire or reinforcement is wound around a thin steel cylinder and encased in concrete. Concrete pipes are generally only economic for large diameter, low pressure (16 bar maximum) transmission purposes. They are heavy, and somewhat inflexible in use, as joints permit limited deflection, and the pipes cannot be cut. Like AC pipes, they are vulnerable in environments aggressive to cement, though can be made in sulphate resisting cement. Connections are difficult once the line has been laid. The pipes have a reasonably good resistance to rough handling or poor backfilling, although care is needed to prevent damage to the joint sockets and spigot ends. Joints are socket and spigot push-in type with a rubber O-ring.

(Ref: BS 4625, AWWA C 301 etc.)

7.1.5 Plastics

Unplasticised polyvinylchloride (UPVC) pipe is widely used in water distribution systems. Available in sizes up to 600 mm in 6 or 9 m lengths, they are suitable only for relatively low pressure duties (up to 150 m working pressure). Major advantages are light weight, flexibility, ease of handling and laying, resistance to corrosion and excellent flow characteristics. It has, however, a limited useful life as pipes tend to become brittle with age. UPVC pipes are also degraded by ultraviolet light and should not be exposed to sunlight in hot climates.

UPVC pipe should not be used for hot water, or at temperatures above 60°C. CP 312 Part 2 recommends a reduction of 2% allowable working pressure per 1°C temperature rise above 20°C. Maximum pressure surges due to water hammer must be included within the allowable working pressure due to fatigue problems with UPVC pipe.

Joints are spigot and socket type for use with solvent cement or a rubber ring. Some failures have occurred with solvent joints, possibly due to use of inferior solvent. UPVC pipe is usually cheaper than ductile iron and AC pipe.

(Ref: SLS 147: 1972, SLS 659 Part 1: 1984, BS 3505, CP 312 Part 2, AWWA C 900 etc.).

Polyetheylene (PE) pipe is more flexible than UPVC. In small sizes it is supplied in coils, reducing the need for joints. It is available as low density PE (LDPE) used for small diameter distribution and service piping, and high density PE in larger sizes. Pressure ratings are up to 12 bars at 20°C, reducing to 5.4 bars at 50°C. Joints for LDPE pipe are made using metal or plastic compression couplings.

(Ref. BS 1972, BS 3284, AWWA C 900 etc).

7.2 Pipelaying

7.2.1 Design Aspects of Pipeline Route

During the design stage, give consideration to the following aspects of the pipeline:

- o pipes should where possible be laid to an even grade of not less than 1:500.
- o existing underground and overhead services, roads, railways, watercourses and drains which cross or are adjacent to the pipeline route;
- o future areas for development or mining - pipeline route should avoid these; consult local planning authority;
- o avoid routing pipeline through highly productive agricultural land, forests or other areas where environmental damage might result;
- o for pipelines crossing private land, ensure correct procedures for rights of access, easements or acquisition are followed; access will also be required for surveys and subsequent maintenance;
- o obtain necessary local authority planning permissions, and approvals from any other concerned authority, ie. road, railway, irrigation, etc.

7.2.2 Pipelaying

Before pipelaying commences:

- o plan for storage of pipe, access for machinery, temporary offices, camps, and sanitary facilities;
- o check necessary approvals and easements have been obtained;
- o arrange for delivery of pipe, careful offloading, inspection and stacking according to manufacturers's recommendations - arrange for necessary cranes and slings. Avoid damage to pipes especially spigot and socket ends;

- reject damaged pipes;

- clear and grade the pipeline route over the planned working width (a minimum of 12 m where mechanical equipment is used), and peg out the route;

- provide temporary fencing where required;

- string pipes along the route;

- note location of land drains and ensure reinstatement after backfilling.

Trenching and laying:

- minimum trench width should be d + 300 mm (see Figure 7.3);

- prepare and level pipe bed carefully: pipe barrel must rest evenly on bed for whole length; excavate joint holes for sockets; remove large stones (see Fig.7.4);

- if trench is rock or hard material, bed pipes on compacted sand or concrete bedding (Fig.7.3) and provide for this in Bill of Quantities;

- do not lay water and sewer or drainage pipes in the same trench;

- lay water pipes above sewer or drainage pipes when in close proximity;

- when crossing other underground service provide minimum of 150 mm clearance and necessary protection and support; where frequent crossings are expected (e.g. in urban distribution pipelines) allow extra 11¼ ° bends for avoiding obstacles.

- under ditches and culverts, protect pipe with a 150 mm thickness of concrete on top and sides of pipe over width of trench (see Fig. 7.5).

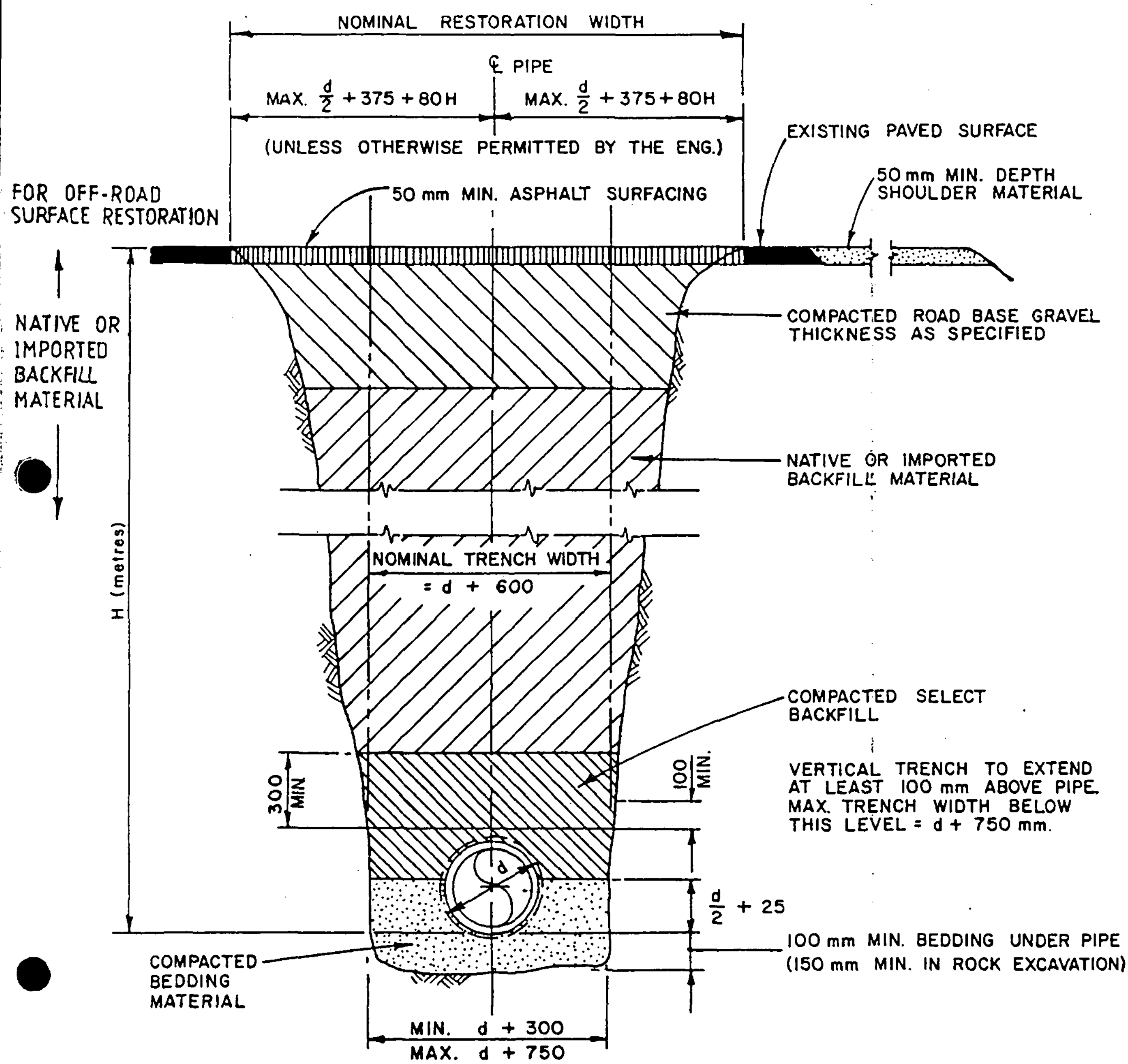
- clean insides of pipes before laying to remove all mud, dirt and grease;

- joint pipes carefully according to manufacturers instructions making sure rubber ring is not displaced;

- inspect pipe protection, and repair any faults that are found;

- plug open ends of pipe during breaks in pipelaying to keep it clean and prevent entry of animals;

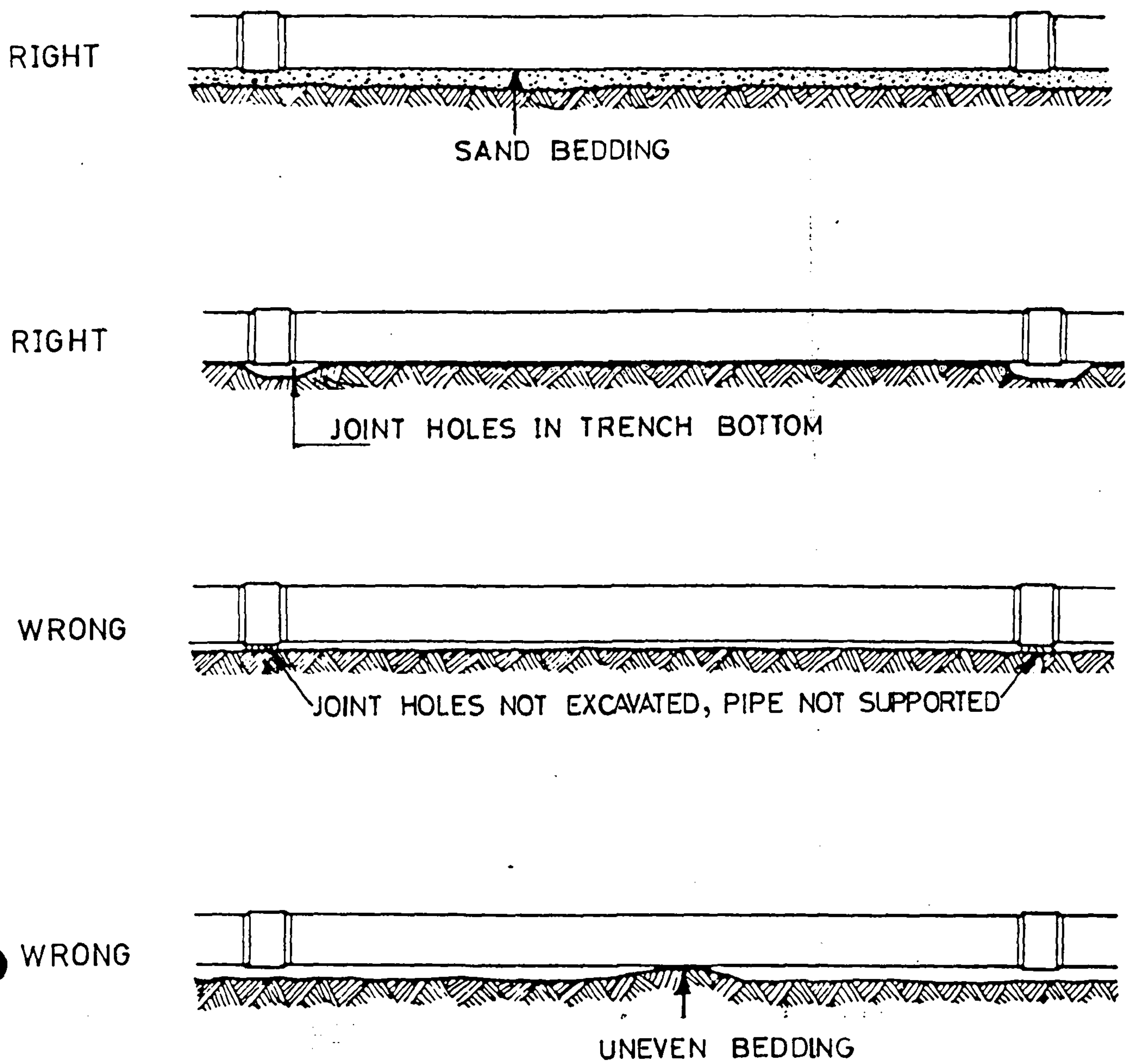
- restrain or block all tees, bends, caps, plugs, hydrants and other fittings to prevent movement under surge conditions which might cause leakage or disjointing; (See Section 7.4.8)



NOTE:
1. ALL DIMENSIONS ARE GIVEN IN mm.
2. d = OUTSIDE DIAMETER OF THE PIPE AT ITS LARGEST SECTION

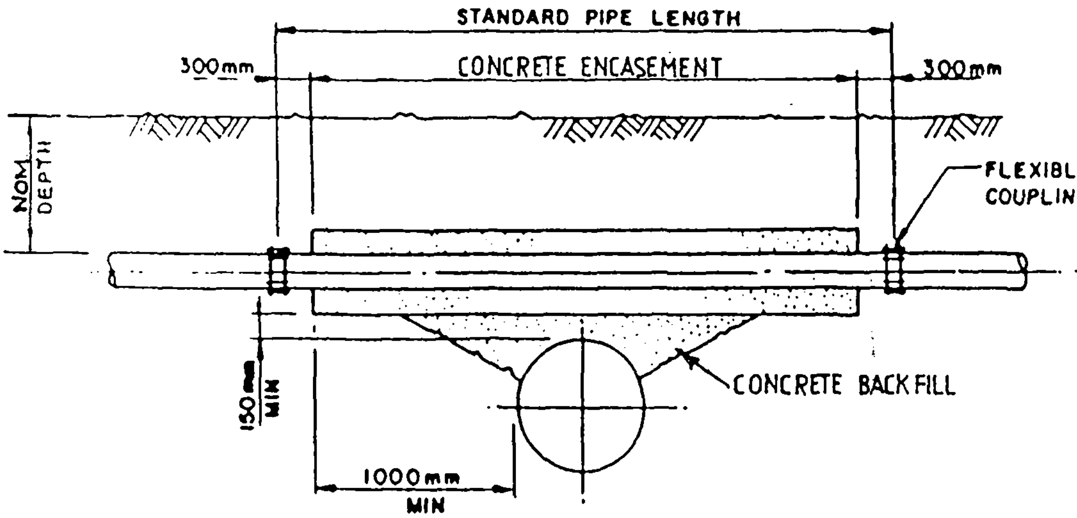
TRENCH DETAILS

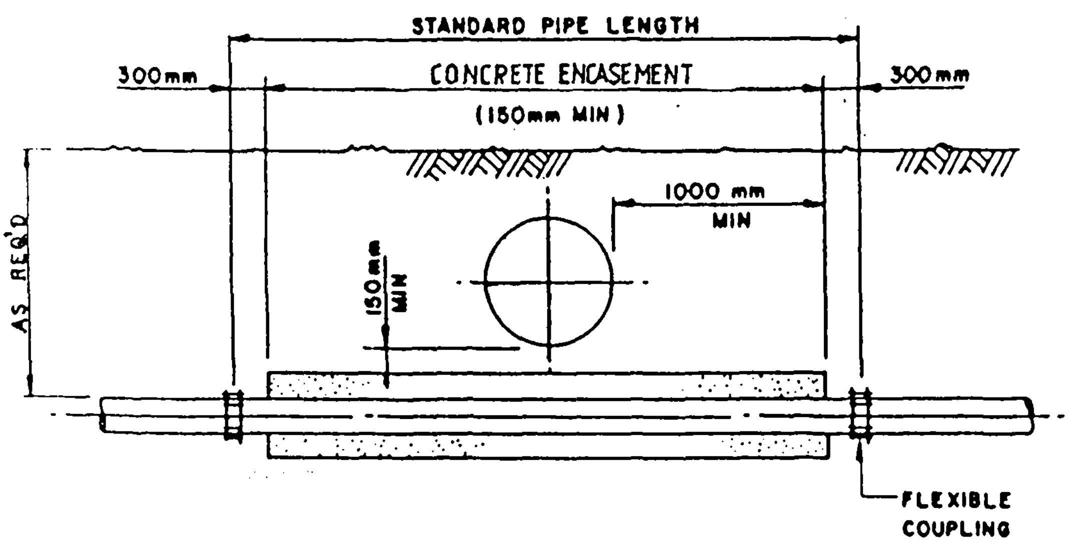
FIGURE 7.3



BEDDING OF PIPE-RIGHT AND WRONG

FIGURE 7.4





CULVERT CROSSINGS

FIGURE 7.5

- o backfill carefully in even layers up to 300 mm above soffit with selected material free from sharp stones which could damage the pipe or its protection.

7.2.3 Testing and Disinfection

After the trench has been partially backfilled, preferably with exposed joints, the pipes should be tested for leakage. The usual procedure for leakage testing is to fill the pipe - either the complete pipe or sections of it - slowly with water, allowing the air to escape through air valves or by some other means. When the pipe is full of water is should be allowed to remain for up to 24 hr. after which the pressure in the line should be increased to 1.5 times normal operating pressure for 1-2 hr. Observations can then be made for any leakage, by sight or by the use of a meter to detect any flow into the pipe. Some leakage is to be expected. However, joints that leak excessively should be relaid. Formulae are available to calculate the amount of allowable leakage.

Before the new main can be put into service the line must be flushed and disinfected. Flushing is necessary to remove any dirt or foreign material from the pipeline and should be done with sufficient velocity to cleanse the pipe's interior thoroughly.

After flushing, the line should be disinfected with bleaching powder solution at a concentration of not less than 20 mg/l for at least 24 hours.

(Ref: CP 2010 Part 1)

7.3 Pipeline Design

7.3.1 Hazen-Williams Formula

This formula is widely used in calculation of pipeline flows due to the fact that it is easy to apply and reasonably accurate. When more accuracy is required, however, the Colebrook-White equation is recognised as being the best available means of estimating friction loss and charts are available for its use (Ref. 20,21)

The Hazen-Williams formula can be expressed in various forms depending on the units, as follows:

$$Q = \frac{3.59}{10^6} CD^{2.63} (H/L)^{0.54}$$

where Q in l/s
D in mm
H & L in m

$$\text{or } Q = 0.279\, C\, D^{2.63}\, (H/L)^{0.54}$$

$$V = 0.355\, C\, D^{0.63}\, (H/L)^{0.54}$$

$$H = \frac{6.78\, L}{D^{1.165}} \left(\frac{V}{C} \right)^{1.85}$$

where Q in m^3/s
D in mm
H & L in m

Typical values of roughness coefficient C are given in Figure 7.6 and Figure 7.7 gives a graphical solution to the formula.

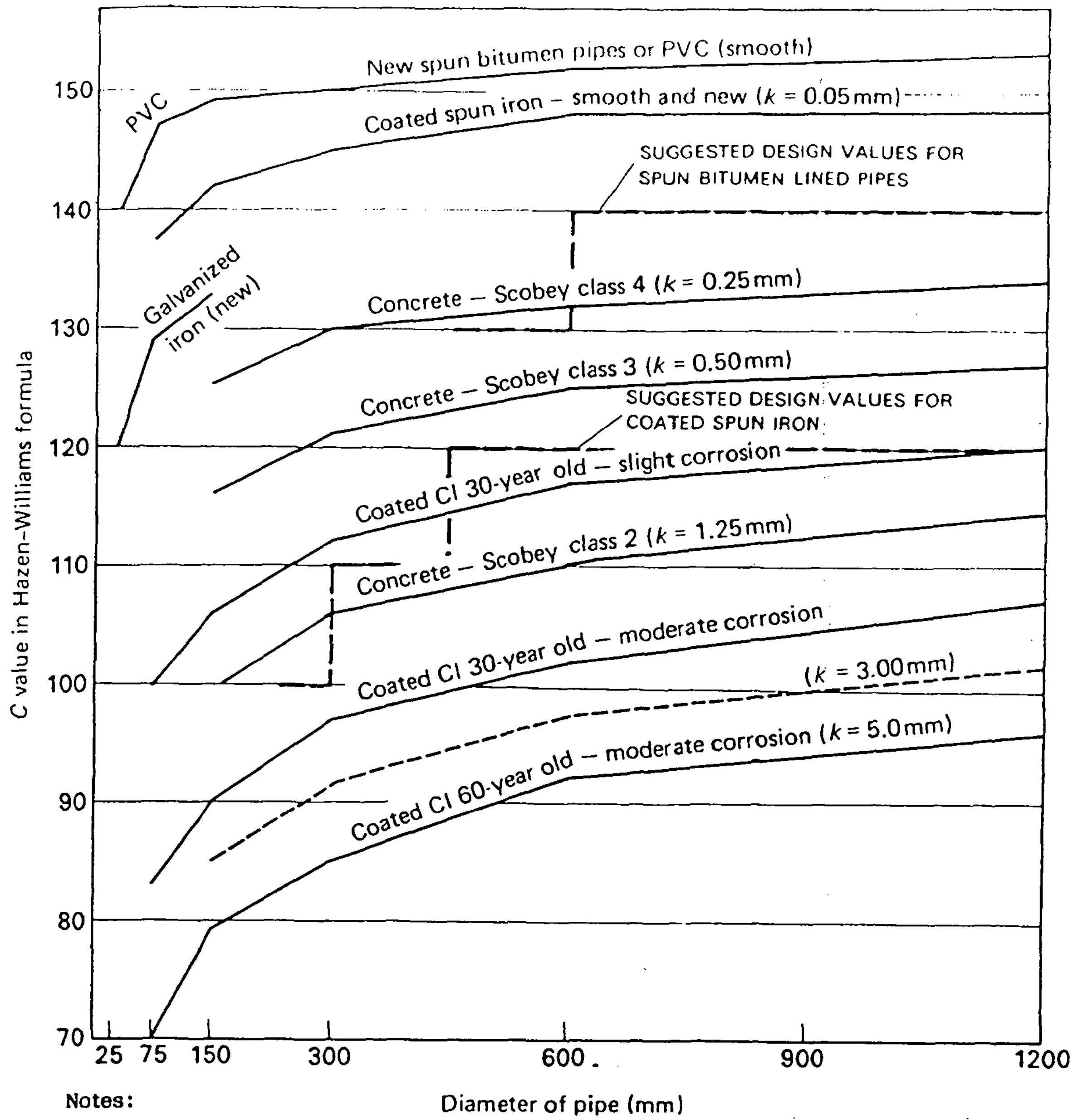
7.3.2 General Design Considerations

Working pressures of pipes should not be exceeded. Table 7.2 gives the recommended pressures for various pipe materials and classes. The pipe materials must be selected to withstand the highest pressure that can occur in the pipeline. The maximum pressure in gravity mains frequently is the static pressure under no flow conditions. In order to limit the maximum pressure in a pipeline and, thus, the cost of the pipes, it can be divided into sections or pressure zones separated by a break-pressure tank. The function of this is to limit the static pressure by providing an open water surface at certain places along the pipeline. The flow from the upstream section can be throtted when necessary. For the protection of ball valves in break pressure tanks, the flow capacity of the outlet pipe is usually made larger than that of the inlet pipe. Where overflow is acceptable, ball valves should be eliminated in break pressure tanks (see Figure 7.8).

Critical pressures may also develop as a result of pressure surge or water hammer in the pipeline. These are caused by the instant or too rapid closure of valves, or by sudden pump starts or stops. The resulting pressure surges create over and under pressure that may damage the pipeline.

The minimum velocity in a pipeline should not be less than 0.6 m/s to prevent deposition of silt. The maximum velocity should be limited to 1.8 – 2.5 m/s to protect the pipe from excessive water hammer. Surge pressures should be evaluated (see Section 6.5 and Annex J) in pumping mains with high pumping heads relative to the class of pipe and material, and pressure relief deviced provided as found necessary. Design flow for transmission mains should be based on maximum day demand based on constant flow over 24 hours.

All hourly variations in the water demand during a day of maximum consumption are then assumed to be levelled out by the service reservoir.



Notes:

(1) k values refer to the Colebrook-White formula.

(2) Suggested design values are for a non-aggressive and non-sliming water.

(3) The curves apply to a 1m/s flow rate. For 2m/s reduce C values by 5% below 100, 3% below 130, and 1% below 140. For 0.5m/s increase C values by the same amounts.

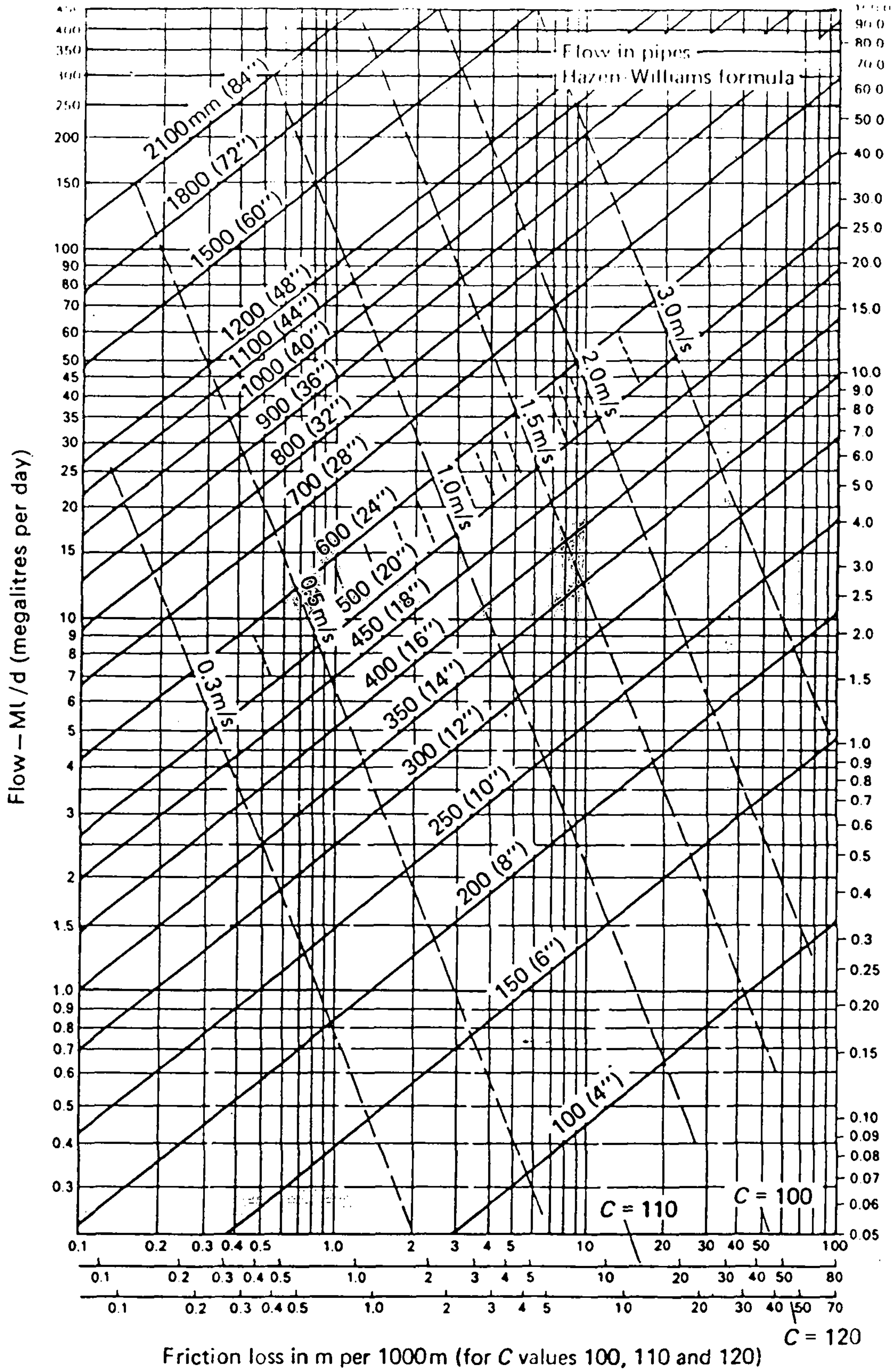
(4) Scobey classes are: class 4 - first class interior finish with all joint irregularities removed; class 3 - good interior finish with joints filled and concrete made on steel forms; class 2 - imperfect interior finish, and as tunnel linings; class 1 - old concrete pipes with mortar not wiped from joints (k= 5.00 mm).

(5) For asbestos cement pipes use values for spun bitumen lined pipes.

TYPICAL VALUES OF ROUGHNESS COEFFICIENT.

Source : Ref. 25

FIGURE 7.6



SOLUTION OF HAZEN WILLIAMS FORMULA

Source: Ref.25

FIGURE 7.7

Table 7.2

PIPE DESIGN CHARACTERISTICS

Material	Standard	Class	Nominal Dia (mm)	Standard Length (m)	Test Pressure (bar)	Working Pressure Rating (bar)	Working Pressure Rating (m_2H 0)
uPVC	SLS 147	Type 250 400 600 1000 Class A B	75- 500 50- 500 32- 500 20- 500 40- 300 10- 300	4,6		2.5 4.0 6.0 10.0	25 (1) 41 61 102
uPVC	BS 3505	Class C D E	50- 600 32- 450 10- 400	6,9		9 12 15	92 (2) 122 153
Ductile Iron (spigot & socket & flanged)	BS 4772, CP 3010 Part 3, BS 8010	K9, K12	80- 300 350- 600 700-1000 1100-1200	5.5	50 40 32 25	40 25 16 16 See also BS 8010	408 (3) 255 163 163
Cast (Grey) Iron (spigot & socket)	BS 78 1211,2035 BS 4622	Class B C D 1 2 3	50- 900 " " 80- 700		12 (4) 18 24 16 20 25	8 12 16 (See Standard) " "	82 122 163
Asbestos Cement	BS 486	Class 15 20 25	150- 900 200- 900 50- 900	3,4,5	15 20 25	7.5 10.0 12.5	76 102 127
Prestressed Concrete Cylinder	BS 4625		40-1800	4,4.5	1.5 times working pressure	As designed	
Steel (welded or seamless)	BS 534, CP 2010 Part 2		60-1800		See Standard	See CP	
Polyetheylene Pipe	BS 1972	Type 425 Normal gauge Heavy gauge	15 20 25 32 40-50 6 10 15 20 25 32 40				91 (2) 81 64 52 49 201 160 146 107 78 75 72

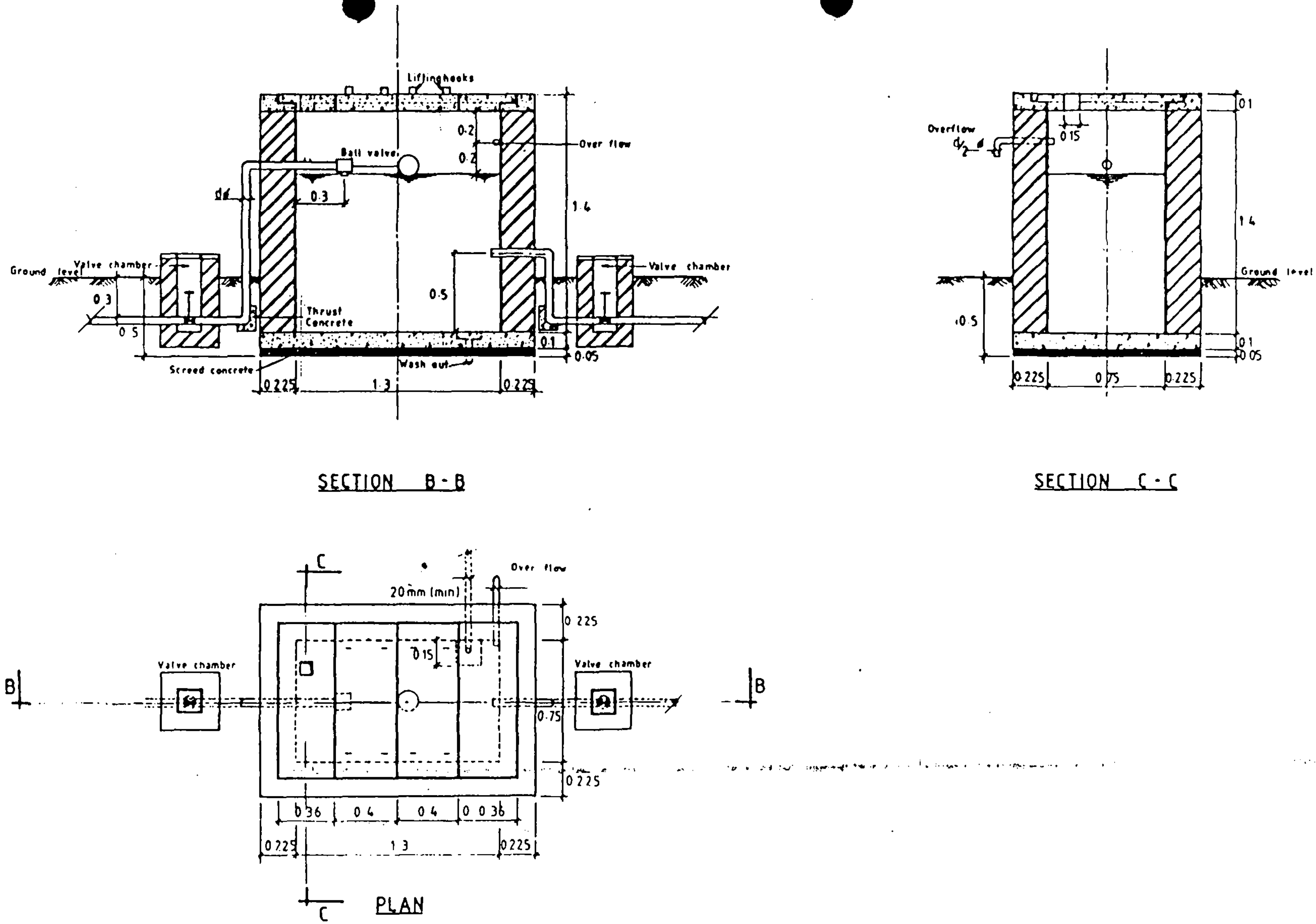
Notes (1) At temp of 30°C

(2) At temp of 20°C (Reduced working pressure at higher temp - Ref CP 312 Part 2)

(3) Max sustained operating (or static) pressure must not exceed working pressure rating and max. internal design pressure including surge pressure must not exceed 1.1 times working pressure rating.

(4) Field hydrostatic test pressures

(5) 1 bar $= 10^2$ kN/m² = 10.1972 m H_2O
1 m H_2O = 9.80665 kN/m²
1 k Pa = 1 kN/m²
1 kgf/cm² = 98.0665 kN/m²



TYPICAL BREAK PRESSURE TANK

FIGURE 7.8

The number of hours the transmission main operates per day is another important factor. For a water supply with diesel engine or electric motor-driven pumps, the pumping period is often limited to 16 hours or less, due to constraints of cost, caretaking or source defficiency. In such cases, the design flow rate for the transmission main needs to be adjusted accordingly.

Pipelines laid in flat terrain should have a minimum grade of 1 in 500 to drain and expel air in the line.

Minimum earth cover to pipes should be 0.8 m except in roadways where the cover should be increased to at least 1.0 m. Where heavy traffic loads are expected, pipe thickness, earth cover, and bedding should be carefully checked.

Sketch the expected hydraulic grade line along the length or profile of proposed main route. Gravity mains should be designed with the hydraulic gradient above the ground surface for all rates of flow in order to prevent negative pressures. Nowhere should the operating head of water in the pipeline be less than 4 m. Air and scour valves should be provided at high and low points, respectively.

Pumping mains should be sized for the most economical combined cost of pumping main and pumping plant installation and operation. Power cost should be calculated for the demands during the design period. The decision to build a pipeline larger than is needed initially should be economically justified by considering the rate of increase of water demand at different time periods. Examples are given in Annex L.

Other design considerations are as follows:

- anchor pipe lines against thrusts resulting from test pressures, normally 150% of system operating pressure; consider all changes in direction;

- use joint harness welded across flexible pipe couplings to take thrust along line if thrust blocks are not used;

- do not use joint harness across flexible pipe couplings with one end clamped to pipe; there is no way that clamp will hold;

- consider all modes of operation of pipe line at inter connections, means of bypass, and maintenance of flow for direction of thrusts;

- provide support for pipes crossing from firn ground across backfilled ground to rigid structure, even though compaction of backfill is specified.

- provide short pieces of pipe connecting to rigid structures or manholes.

- allow for expansion and contraction of piping between installation and putting into service. Remember PVC pipe expansion is 8 mm/30 m/5°C; keep temperature difference below 8°C. Remember PE pipe expansion is 22 mm/30 m/5°C; use PVC pipe with gasketed joints, rather than solvent cemented joints;

- consider means of getting air out of pipelines, while putting line into service or testing;

- keep velocity when filling a pipeline to less than 0.30 m/s; the velocity should not exceed 0.60 m/s;

- use caution in selecting the location of air valves; remember that air valves are no good unless they are properly maintained;

- select air valves with fewest mechanical parts to go wrong;

- remember that air valve chambers do need venting in order to work.

7.4 Valves and Appurtenances

7.4.1 General

Mains should be divided into sections by the provision of stop valves so that the water may be shut off for repairs.

Air valves should be provided at pipeline summits and washouts at low points between summits unless adequate provision is made for the discharge of air and water by the presence of service connections, fire hydrants or standposts.

Large-orifice air valves will discharge displaced air when mains are being charged with water. When air is liable to collect at summits under ordinary conditions of flow, small-orifice air valves, which discharge air under pressure, may be required. "Double-acting" air valves having both large and small orifices should be provided where necessary. Air valve chambers should have adequate drainage to avoid the possibility of contamination.

Washouts should not discharge into a drain or sewer or into a manhole or chamber connected thereto. Where a washout discharges into a natural watercourse, the discharge should at all times be well above the highest possible water level in the watercourse. In some cases it may be necessary for the washout to discharge into a watertight sump which has to be emptied while in use by portable pumping equipment.

Mains need not be laid at unvarying gradients but may follow the general contour of the ground. They should, however, as far as possible, fall continuously towards the washouts and rise continuously towards the air valves.

Anchor blocks should be provided at every bend, branch and dead end in a main to resist the hydraulic thrust.

7.4.2 Stop Valves

These may be of several types of which the most common are:

- o sluice valves;
- o butterfly valves; and
- o screwdown plug valves.

Sluice valves, or gate valves, are the normal type of valves used for isolating or scouring. They seal well under high pressure and when fully open offer little resistance to fluid flow.

There are two types of spindles for raising the gate: A rising spindle which is attached to the gate and does not rotate with the handwheel, and a non-rising spindle which is rotated in a screwed attachment in the gate. The rising spindle type is easy to lubricate.

The gate may be parallel-sided or wedge shaped. The wedge gate seals best, but may be damaged by grit. For low pressures, resilient or gunmetal sealing faces may be used but for high pressures stainless steel seals are preferred.

Despite sluice valves' simplicity and positive action, they are sometimes troublesome to operate. They need a big force to unseat them against a high unbalanced pressure, and large valves take many minutes to turn open or closed. Some of the problems can be overcome by installing a valve with a smaller bore than the pipeline diameter. Valves of 400 mm and above should be provided with by-passes.

Butterfly valves are cheaper than sluice valves for large sizes and occupy less space. The sealing is sometimes not as effective as for sluice valves, especially at high pressures. They also offer a fairly high resistance to flow even in the fully open state, because the thickness of the disc obstructs the flow even when it is rotated 90 degrees to the fully open position. Butterfly valves, as well as sluice valves, are not suited for operation in partly open positions as the gates and seatings would erode rapidly. As both types require high torques to open them against high pressures, they often have geared handwheels or power-driven actuators.

Screwdown plug valves, include the usual types of small diameter bibtaps and stopcocks. They are susceptible to wear and consequent leakage at the seating, and can cause considerable loss of pressure.

7.4.3 Non-return Valves

Reflux, or non-return, or check valves as they are also known, are used to stop flow automatically in the reverse direction. Under normal flow conditions the gate is kept open by the flow, and when the flow stops, the horizontally hinged gate closes by gravity or with the aid of springs. A counterbalance can be fitted to the gate spindle to keep the gate fully open at practically all flows, or it could be used to assist in rapid closing when the flow does stop. Springs are also sometimes used to assist closing. Larger non-return valves may have multi-gates, in which case the thickness of each gate will partially obstruct the flow, and swabbing of the pipeline may be difficult. Mounted in horizontal pipes, the gates of some types of non-return valves tend to flutter at low flows, and for this reason an offset hinge which clicks the gate open is sometimes used.

Small double check valve assemblies are recommended on water service lines as backflow prevention devices.

7.4.4 Control Valves

These may be used to reduce or sustain pressure, maintain a constant flow rate, or set water levels. Often with automatic control, they can be utilised for a variety of conditions, but require careful setting and maintenance.

Pressure Reducing Valves (PRV): automatically reduce a higher inlet pressure to a constant lower downstream pressure, regardless of changing flow rate and/or varying inlet pressure.

PRVs should be selected and sized for required system flow rates, not by main line size. PRVs are suitable for velocities to 4:5 m/s (continuous) and 8 m/s (intermittent) service, therefore the correct PRV, in most cases, will be at least one pipe size smaller than the main line size.

PRVs are generally installed in pairs, but sometimes only a single valve may be used on stations under 100 mm, or even three are used where there is a wide variation in flow. The number of PRVs to be used will depend on conditions of service.

When using two or more PRVs in parallel, they should open at varying pressures to prevent excess throttling of the valves. The smallest valve is set to a slightly higher pressure to handle low flows. To handle high flow and peak loads, the larger valve(s) open in sequence as downstream pressure drops. The valves in reverse order with the smallest valve closing last.

With parallel installation any single valve can be taken out of service for repairs or adjustment without shutting down the line. Valves are normally installed in subsurface vaults with ready access and adequate space around the valves for maintenance.

Many reducing problems result from improper sizing. Factors of initial pressure, reduced pressure, minimum and maximum pressure drop and flow rates should be considered instead of simply using line size valves. Regulators should be protected from foreign material - install a strainer ahead of each PRV, unless the system is adequately protected elsewhere. Pressure gauges should be installed on the upstream and downstream side of the PRVs for setting purposes and checking operation of valves under service conditions.

It is good insurance to install a pressure and surge relief valve on the downstream side of the PRV station to protect the distribution system from excess pressure in the event of a PRV failure.

For maximum accuracy of regulation, on large main line multi-PRV stations, pilot controls should be connected to an external upstream supply and downstream sensing lines connected to the large diameter main line.

There are several important auxiliary control features that can be added to pressure reducing valves, as follows:

o Downstream surge control - this control will override PRV control to correct valve position in the event of quick rejection of flow down-stream of the PRV or malfunction of the PRV Control System.

o Excess flow shut down - in the event of main line break.

Pressure relief valves: maintain a constant upstream pressure by relieving excess pressure to drain or to a lower pressure zone. They are used on:

o <u>Pump systems</u> -

a) to provide protection against high surges when pumps are shut down.

b) to maintain constant pump dischage pressure by bypassing excess back to suction. Constant pressure is maintained automatically as flow rate decreases and increases.

c) to maintain constant back pressure on pumps to prevent excess flow rates due to high demand or line break.

o <u>Pressure reducing stations</u> - to provide downstream system protection against over pressure due to malfunction of PRV or quick rejection of flow due to fast closing valves or hydrants.

- <u>Distribution systems</u> - when installed in a line between and upper zone and lower area of heavy demand, the valve acts to maintain desired upstream pressure to prevent "robbing" of the upper zone. Water in excess of pressure setting, flows to an area of heavy demand.

Liquid level controls: available in 2 basic types:

Float valves
Altitude valves

Float valves are controlled by a float riding on the free surface of the water in the tank. They provide the most economical installation,and are simple and easy to service.

Altitude Valves are controlled by a pressure sensitive pilot control with a constant head loading balanced against the variable tank level. They are commonly used for controlling tank and reservoir levels in many major municipal systems elsewhere.

Both types are available with on-off or modulating control. Each type has its use in a municipal system.

Careful consideration must be given to the choice of on-off or modulating control. On-off control is recommended for the following services:

- Gravity supply systems when chlorination is required. The on-off type of control valve supplies water at a constant flow rate and permits the use of a simple advance type of chlorinator.

- Pump systems where the pump starts when tank level is low and fills the tank to high water level and then stops. On-off control is recommended for this type of system to assure that the pump does not run for long periods against a partially open valve.

Modulating control is recommended for the following services:

- Reservoir level control in large distribution network systems where sudden increases and decreases in flow rates must be avoided. Modulating control automatically adjusts the inflow to the reservoir to match outflow, and therefore maintains a constant reservoir level regardless of system demands.

- o Level control to a reservoir that is sitting on the line to maintain constant system pressure with reserve for periods of peak demands.

Pump control valves: pump systems play a very important part in municipal distribution systems. In many cases, they are the prime source of water. In other cases, they are used to boost pressure to reach remote areas at the far end of a system during periods of peak demand.

In higher head installations it is important to provide good start-up and shut-down control as well as power failure protection against the harmful effects of water hammer.

The basic principle of pump control is to do everything slowly. To achieve this, there are 2 types of control available:

- o bypass control
- o in-line control

In both cases, the valve does automatically what experienced pump operators do manually, to prevent starting and stopping surges.

The bypass type of control system is the most economical method and is generally used in conjunction with vertical turbine pumps. This type of valve is open when the pump starts and stops. On start up, the total capacity of the pump is bypassed to drain, or back to suction. The valve closes slowly to put flow on-stream smoothly. On shut down, the valve first opens slowly to take flow off-stream smoothly before the pump is stopped.

The in-line type of control system requires a full discharge line sized valve and is generally used on centrifugal booster type pumps. This type of valve is closed when the pump starts and stops. On start up, the valve opens slowly to put flow on stream smoothly. On shut down, the valve first closes slowly to take flow off stream smoothly before the pump is stopped.

For power failure protection, a surge control valve should be used, with 2 basic functions. It should open on the low pressure wave of the water hammer cycle to readily dissipate the return high surge; it should be equipped to open on over-pressure due to high pressure surges caused by malfunction of the control system.

7.4.5 Air Valves

When a pipeline is filled, air could be trapped at peaks along the profile, thereby increasing head losses and reducing the capacity of the pipeline. Air vent valves are normally installed at peaks to permit air in the pipe to escape when displaced by the fluid. They also let air into the pipeline during scouring or when emptying the pipeline. Without air valves, vacuums may occur at peaks and the pipe could collapse, or it may not be possible to drain the pipe-line completely. It is also undesirable to have air pockets in the pipe as they may cause water hammer pressure fluctuations during operation of the pipeline.

The three basic types of air valves are single small orifice, single large orifice and double orifice.

The small orifice valve is designed to release air which has come out of solution in the system during pressured operation. As the air collects in the valve body, it depresses the water level until buoyancy is lost and the float falls away from the outlet orifice letting air out. The rising water level then shuts the outlet again.

Valves are available for pressures up to 25 bar and are normally of 25 mm nominal size with a ferrule for screwing into the pipeline or a flange for bolting to a branch. An isolating cock can be incorporated.

The large orifice valve is designed to allow large volumes of air to exhaust from the pipeline during initial filling and free entry of air during emptying. The air inflow rate must be sufficient to permit dewatering or scouring to be carried out without a risk of pipeline collapse through excessive vacuum.

The valve element consists of a cylindrical or spherical float, which remains open during filling or emptying and which should only close through water rising into the valve body and "floating" the valve onto its seat. A pipeline pressure about 0.1 bar above atmospheric should hold the valve closed even when air has displaced the water from the valve body.

The double orifice valve essentially combines a single small orifice unit and a single large orifice unit in one assembly and performs the functions of both. It is available in two forms: with the large orifice unit forming the main assembly and the small orifice unit attached with no integral isolation valve; or with the valve built around a central screwdown isolating valve which has the small and large orifice units disposed about it.

The latter arrangement creates turbulent air flow through the large unit so that stability and capacity are compared with the other simple "straight-through" type. Stability is very important, particularly since the discharging air can reach sonic velocity when the pressure difference is 0.9 bar. The "straight-through" arrangement can be isolated by a gate or butterfly valve mounted between the air valve and the pipe branch. This also allows the air valve to be removed from the pipe without interrrupting flow.

When considering location of air valves the following points should be remembered: (See Fig. 7.9).

- o Double orifice valves are essential at all peak points which must be defined with reference to the hydraulic gradient at maximum flow as well as to the horizontal. A peak may be considered as any pipe section which slopes up towards the hydraulic gradient or runs parallel to it.

- o Air tends to get trapped at points where the upward gradient decreases, or the downward gradient increases. Here a single small orifice or a double orifice valve should be used depending on the extent of the change.

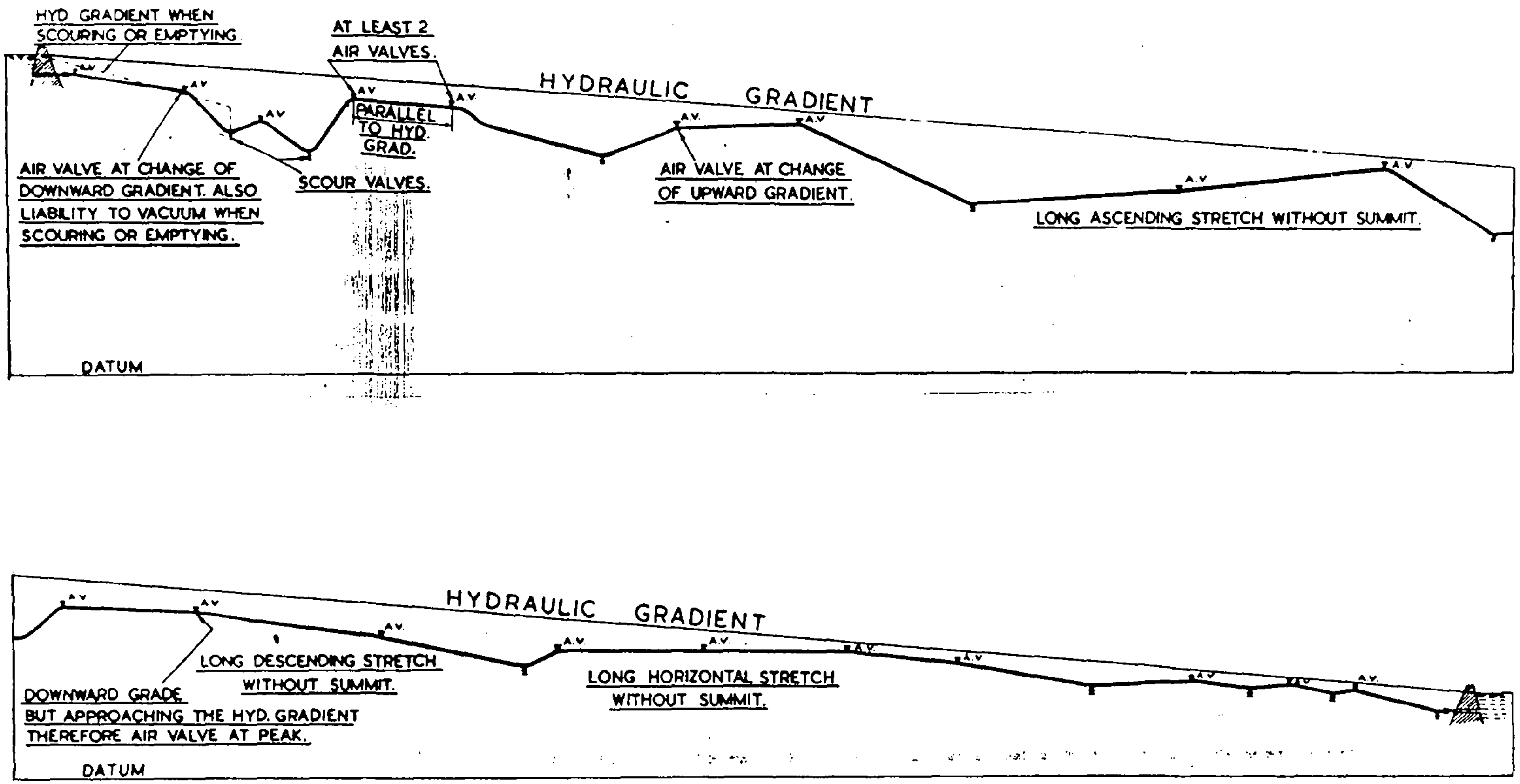
- o Valves will also be necessary at intermediate points on a long incline to limit the interval between valves to 500m. They may be single small orifice supplemented by double orifice at 1 km intervals.

- o Long declines should be considered on a similar basis to inclines except that double orifice valves should be used.

- o Truly horizontal sections or those with gradients of less than 0.2% are difficult to ventilate, particularly if the fluid velocity is less than 2m/s. Double orifice valves should be fitted at intervals not longer than 500m.

Generally, more air valves are required at the beginning of a system and while precise rules cannot be given, the maximum interval between valves should not exceed 500m in these early sections, whereas this can be increased to 750m towards the end. It is also advantageous to over-ventilate high-pressure sections, because air trapped in these sections will be more readily taken into solution so increasing the amount of air released in the lower pressure sections.

[redacted]



CRITICAL AIR VALVE LOCATION POINTS

FIGURE 7.9

Some features require special attention: an additional airvalve is often an advantage on a pump discharge, particularly in a suction-lift application; artificial pockets such as the bonnet section of a large gate valve or a pipeloop may require a single small orifice valve; and there is merit in fitting a small valve downstream of fittings such as pressure reducing valves, orifice plates, or taper pipes which are likely to cause pressure changes.

When selecting valve sizes, it should be remembered that small orifice valves must operate when the pipeline is pressurised, so it is vital that the maximum pressure condition is correctly identified for each location. Maximum pressure will occur under low flow conditions and selection on the basis of pressure will ensure that valves operate under all working conditions.

While sizes tend to be empirical, the discharge rate can be calculated from the formula:

$$Q = 0.0079\, d^2\, P_1$$

Where Q = rate of flow of free dry air (m^3/min.)
d = orifice diameter (mm)
P_1 = absolute pressue at valve inlet (bar)

Large orifice valves are required to admit or release air during filling or emptying of the pipeline and should be capable of passing free air at the same rate as the water enters or leaves the system. Once again, size selection tends to be empirical, but new high-performance designs have been introduced which have a different relationship from conventional valves.

Where actual air flow requirements are known the valve size can be calculated from the formula:

$$Q = C\sqrt{\Delta P \times P_1}$$

for inflow and for outflows up to a differential pressure of 0.9 bar.

Q = rate of flow of free dry air (m^3/s).

C = flow coefficient
ΔP = differential pressure (bar)
P_1 = absolute pressure at valve inlet (bar).

Where the outflow differential pressure is in excess of 0.9 bar, the formula becomes:

$$Q = C \times P_1$$

The flow coefficient C is dependent on valve size and configuration and is available from manufacturers; it approximates to $0.008d^2$, d being the orifice diameter in mm.

Double orifice valves are sized by applying the rules for both single small and large valves.

As a general guide, preliminary to the detailed evaluation above, the sizes of air valves to be installed on mains are to be as follows:

Main - dia. (mm)	Valve - dia. (mm)
Up to 450	75
500 to 600	100
700 to 800	150
900	2 x 100
1000 to 1200	2 x 150

Air valves can also be adapted for a number of special applications in addition to ventilation.

Surge alleviation: Air valves cannot relieve pressure surges in a water system, but surge effect can be minimised by removal of the air and hence the elasticity from the system. However, there are operating situations when air valves can be adapted to alleviate surge. In particular, pipelines which may be subject to column separation, possibly as a result of pump stoppage, benefit from free admission of air at the separation, provided that the redischarge of this air can be controlled. A vented non-return valve produces this characteristic, converting the pipeline into an air vessel to cushion the columns as they rejoin.

Low pressure zones: In certain systems a condition may arise where the hydraulic gradient passes below a peak point. A conventional air valve will admit air under these circumstances and in order to ventilate these peaks an "inflow" check valve must be incorporated in both of the air valve orifices. This allows air to be released without the embarrassment of air entering when not required.

Ref: Gilbert, S. "Air valves can protect your pipeline," World Water, Nov. 1979.

7.4.6 Washouts

These, used for emptying a main or removing stagnant or dirty water, are located at all low points on a transmission pipeline route preferably near drains or streams. Usually 100 mm to 225 mm in size, the level invert tee outlet should be carefully protected to prevent scour around the pipeline due to the high velocity discharge, and the discharge channel to the receiving drain should similarly be protected.

As a general guide, the sizes of washouts to be provided are as follows:

Main dia. (mm)	Washout dia. (mm)
Up to 500 mm	75 mm
600 to 800	100 mm
900	150 mm
1000 to 1200	200 mm

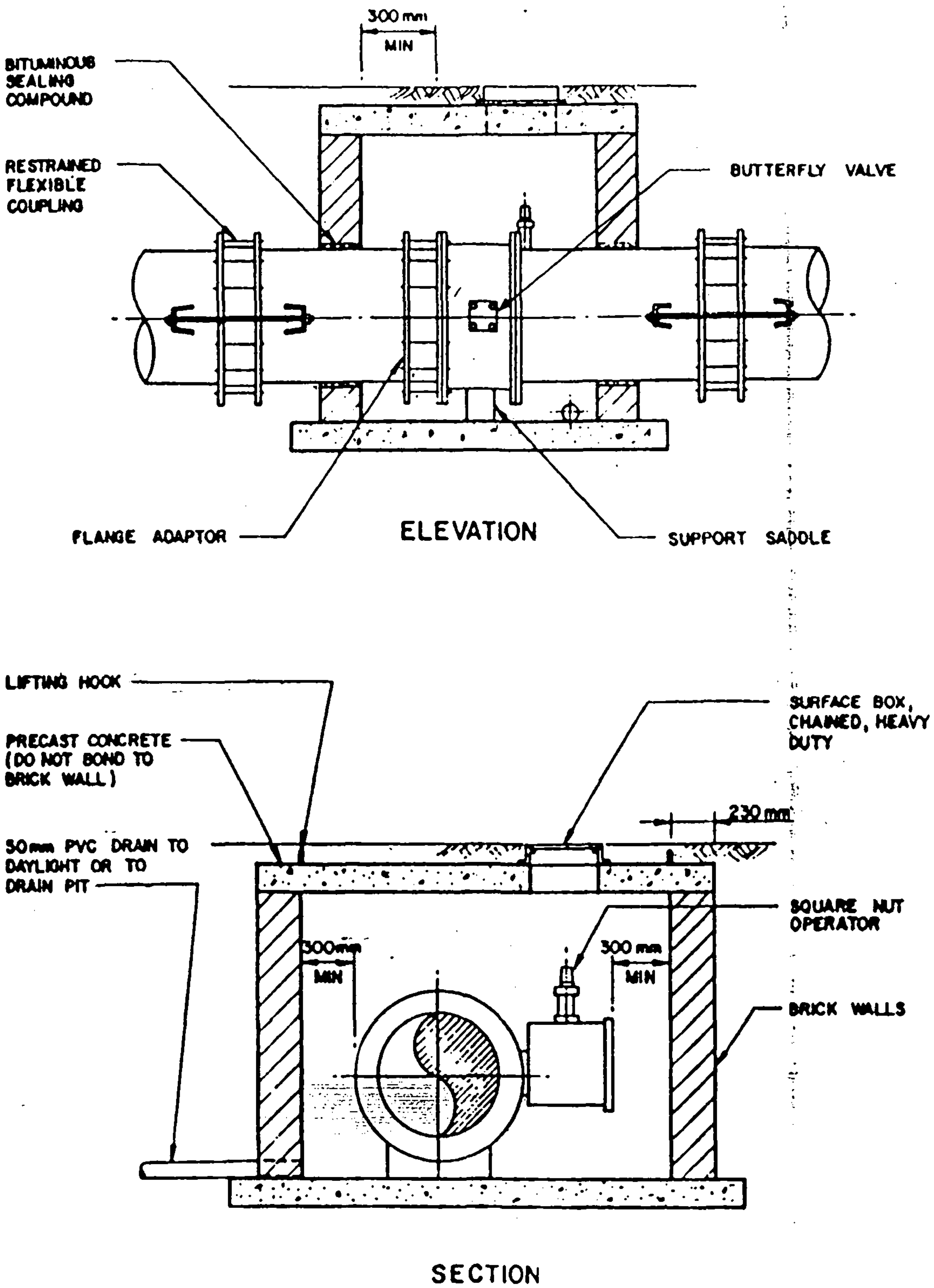
7.4.7 Valve Chambers

Typical valve chambers are shown in in Figs 7.10 and 7.11. On larger mains of 900 mm and above, manholes 500 mm in diameter should be provided near stop valves and air valves.

7.4.8 Thrust Blocks

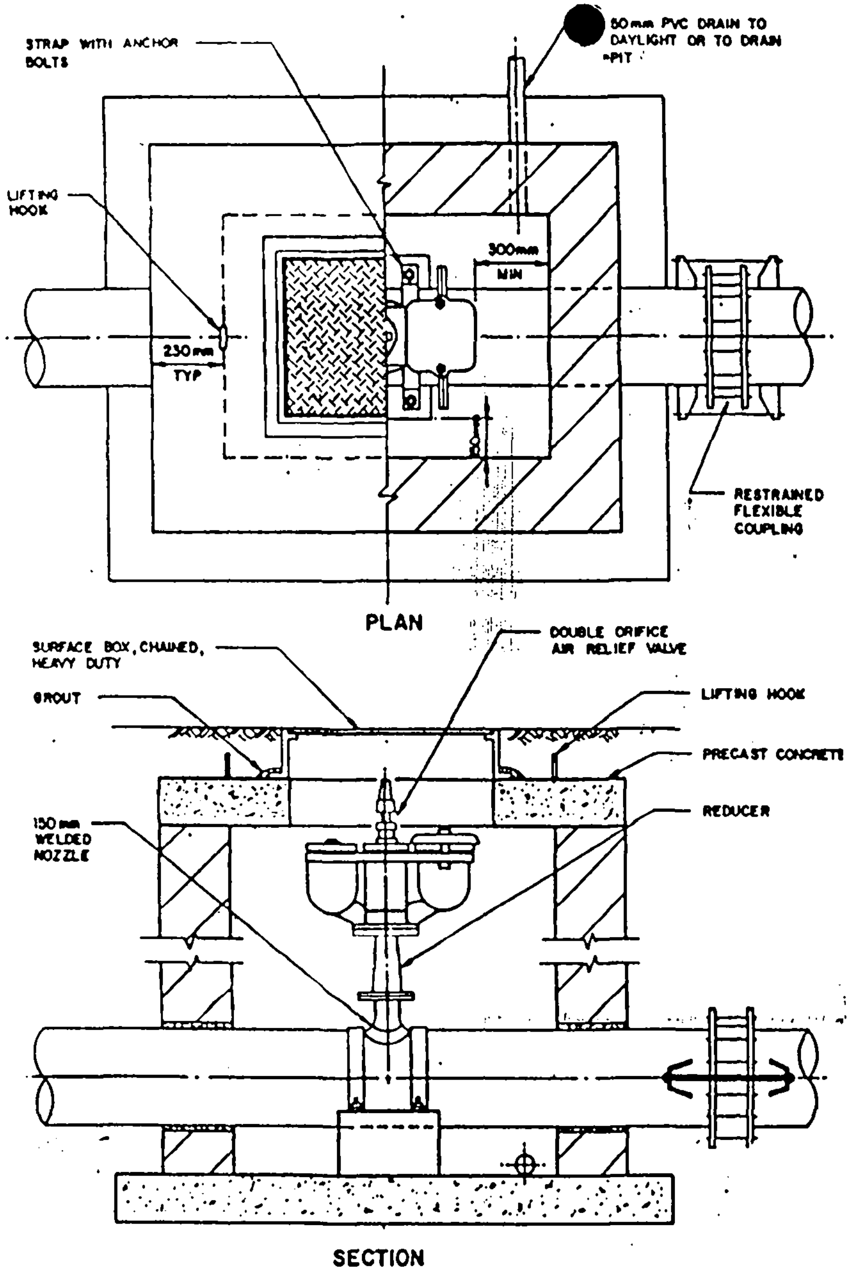
Refer to Table 7.3 and Figure 7.13. Typical details at river crossings are given in Figure 7.12.

For more details see Ref. 20.

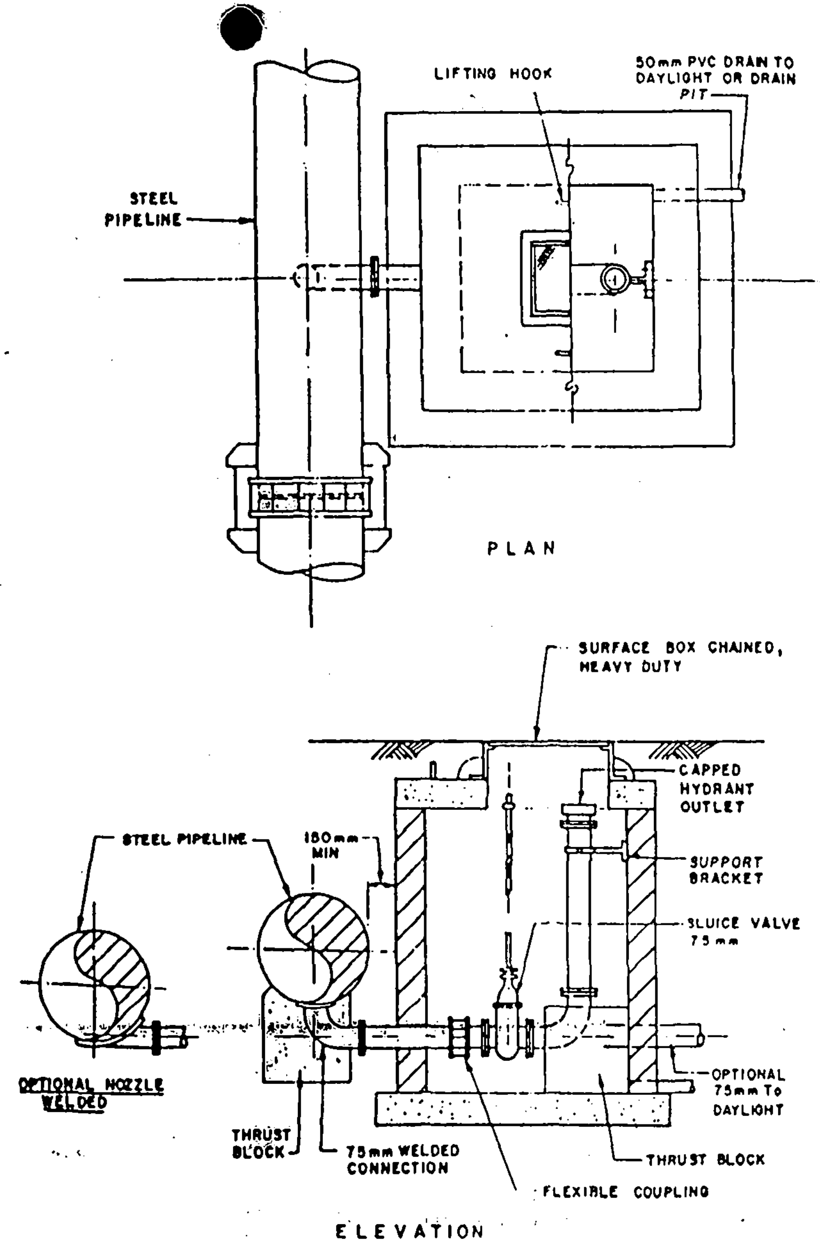


TYPICAL BUTTERFLY VALVE CHAMBER

FIGURE 7.10



TYPICAL AIR RELIEF VALVE CHAMBER



TYPICAL WASHOUT CHAMBER

FIGURE. 7.11

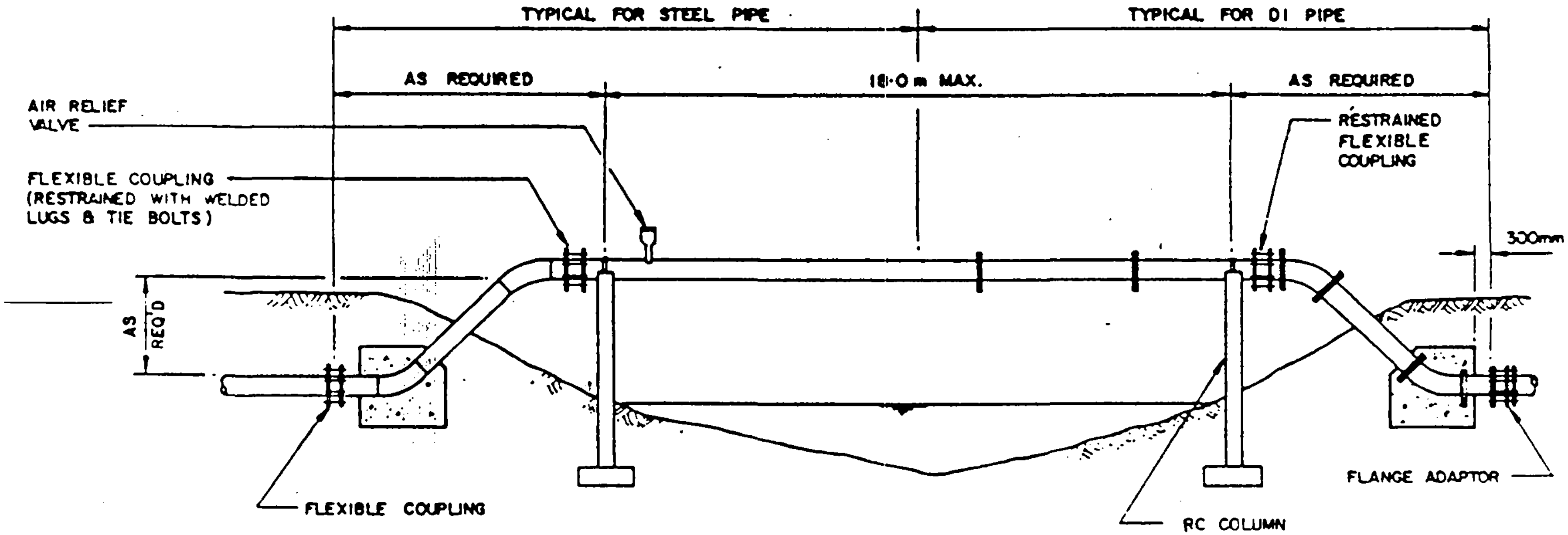
Table 7.3

End & Radial Thrusts for 1 Bar (100 kPa) Internal Pressure

Nom Internal Diameter (mm)	Blank Ends and Junctions (N)	90° Bends (Newtons)	45° Bends (Newtons)	22½° Bends (Newtons)	11¼° Bends (Newtons)
50	374	529	284	148	71
75	716	1013	548	277	142
100	1168	1652	897	458	232
125	1761	2490	1348	690	348
150	2471	3497	1890	961	484
175	3290	4652	2516	1284	645
200	4232	5987	3239	1652	832
225	5271	7452	4032	2058	1032
250	6426	9090	4916	2510	1258
300	9374	13258	7174	3658	1839
350	12523	17710	9587	4884	1458

Source: Public Utilities Board, Singapore - Code of Practice on Water Services.

Note 1 bar = 10.1972 mH_2O.

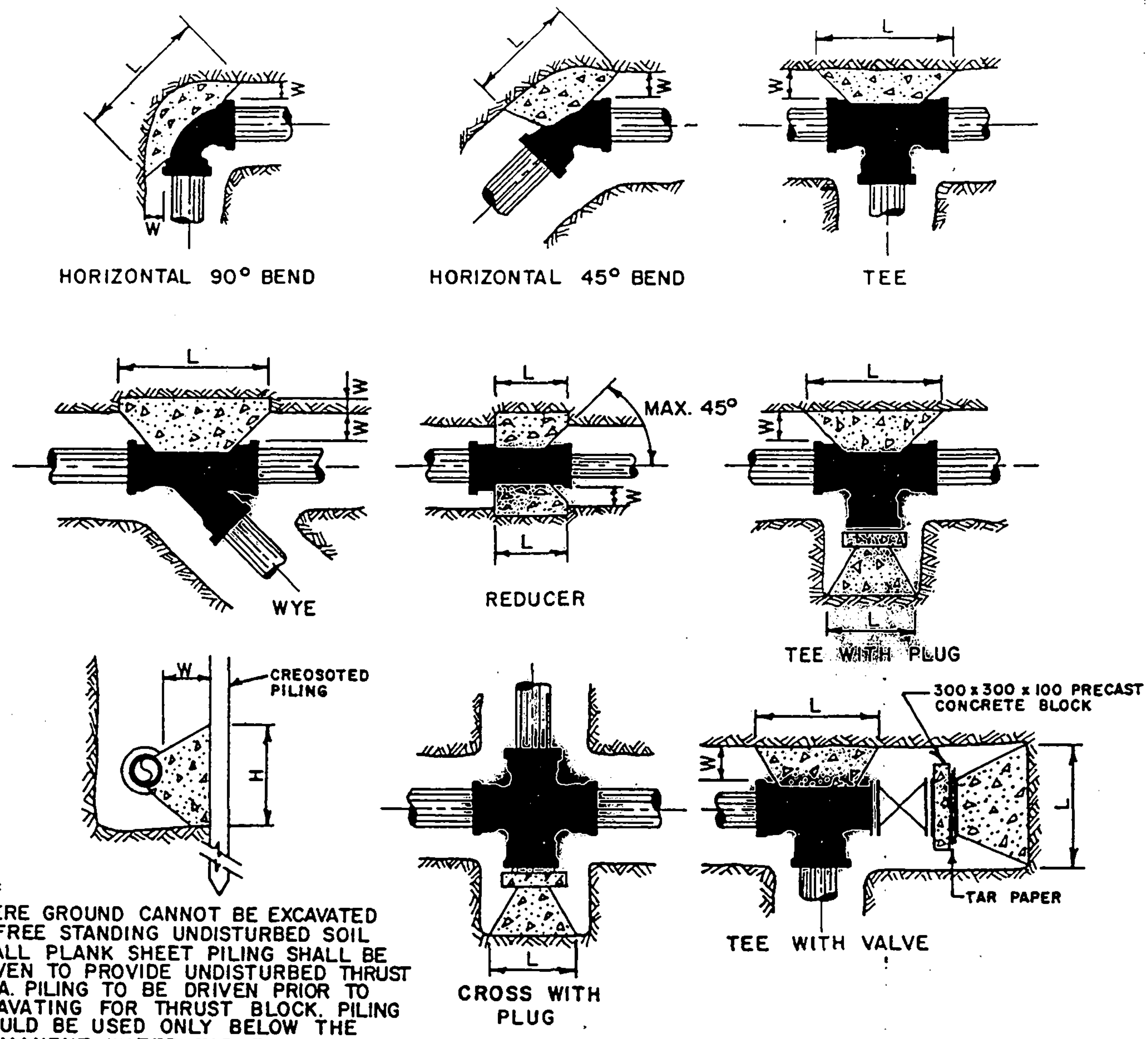


NOTES:

1. ALL RC COLUMNS & PILES TO BE LOCATED SO THERE IS MINIMAL OBSTRUCTION OF MAIN CHANNEL.
2. PIPELINE TO BE LOCATED SO THERE IS 600 mm MINIMUM CLEARANCE FROM MAXIMUM WATER LEVEL.

PIPELINE RIVER CROSSING DETAILS

FIGURE 7.12



NOTES:

1. WHERE GROUND CANNOT BE EXCAVATED TO FREE STANDING UNDISTURBED SOIL SMALL PLANK SHEET PILING SHALL BE DRIVEN TO PROVIDE UNDISTURBED THRUST AREA. PILING TO BE DRIVEN PRIOR TO EXCAVATING FOR THRUST BLOCK. PILING SHOULD BE USED ONLY BELOW THE PERMANENT WATER TABLE.
2. ALL DIMENSIONS ARE GIVEN IN millimetres UNLESS OTHERWISE INDICATED.

MINIMUM THRUST AREAS FOR FITTINGS AT 1030 kPa PRESSURE AND FOR SOILS WITH MIN. BEARING OF 960 kPa
(NOT TO BE USED FOR SOFT CLAY, MUCK, PEAT ETC.)

TYPE OF FITTING	FITTING SIZE	OUTSIDE OF FITTING TO BEARING FACE	RECESS IN TRENCH WALL	LENGTH	HEIGHT	TYPE OF FITTING	FITTING SIZE	OUTSIDE OF FITTING TO BEARING FACE	RECESS IN TRENCH WALL	LENGTH	HEIGHT
	D	W	W	L	H		D	W	W	L	H
90° bend	150 200 250 300	300 350 380 400		920 1070 1450 1650	460 610 760 920	cross	150 200 250 300	300 350 380 400		610 760 990 1220	460 610 760 920
45° bend	150 200 250 300	300 350 380 400		460 610 760 920	460 610 760 920	45° wye	150 200 250 300	300 350 380 400	300 400 500 600	460 610 760 920	460 610 760 920
22 ½° bend	150 200 250 300	300 350 380 400		460 610 840 920	230 300 460 460	reducer*	150 200 250 300	300 350 380 400	150 200 250 300	460 510 760 920	460 610 760 920
tee	150 200 250 300	300 350 380 400		610 760 990 1220	460 510 760 920	caps and plugs (if not bolted)	150 200 250 300	300 350 380 400		460 610 760 920	460 610 760 920

*DIMENSIONS APPLY TO THE LARGER DIAMETER END OF FITTING

THRUST BLOCK DETAILS

FIGURE. 7.13

8. DISTRIBUTION

8.1 Design Considerations

8.1.1 Pipe Network

For a larger urban system, pipelines should be considered in 4 categories, as follows:

a) Transmission mains
b) Primary mains
c) Secondary mains
d) Tertiary mains

a) Transmission (or supply) mains (dealt with in Section 7) carry water from the source or treatment plant to service reservoirs or the edge of the supply area. They should have no cross connections, services or hydrants, and line valves, air-valves and washouts should be provided.

b) Primary (or trunk) mains (usually 250 mm and above) form the main grid and primary ring main system supplying water from the service reservoirs to each distribution zone or pressure zone. Pressure will be governed by the zone service reservoirs. Primary mains should be connected with all crossing primary and secondary mains. No services or hydrants should be allowed, and air valves and washouts should be provided.

c) Secondary (or feeder) mains (usual size 150-300 mm) supply water to consumers either directly or via tertiary mains. Cross connections with all other secondary and tertiary mains should be provided, together with hydrants. Service connections are allowed; air valves and washouts are not required.

d) Tertiary (or network) mains (usually 75-200 mm) provide the minor distribution to serve all consumers in the system not adjacent to secondary mains.

In general a looped, inter-connected ring main system of primary mains is desirable for optimum system design and flexibility. However, dead-end mains usually cannot be avoided.

8.1.2 Location

Adopt the following criteria for positioning distribution mains:

- The criteria governing the depth of mains below ground will be easy maintenance, avoidance of excessive earth pressure and protection from live load due to traffic.

- Mains laid in trenches should have a minimum cover of 0.8 m for pipes 400 mm and less and 1.0 m for larger pipes. Where mains will be subject to traffic loading, minimum cover should be 1.0 m.

- The depth of cover should be increased as necessary where the ground level is to be changed in future for construction of a road, where increased depth is needed to maintain minimum slope in the pipeline where this will eliminate the need for an air valve, or where other special requirements call for greater depth.

- Wherever a suitable route is available, mains will preferably be laid on the side of the carriageway, in verges, footpaths, pavements or green strips.

- Mains proposed to be laid under roads are to be located at a minimum distance of 1 m from the edge of the road or the roadside drain.

- Mains crossing railways shall be laid either in open cut or in tunnel as may be required by the authorities concerned.

- Mains crossing under a railway, highway or road shall be at a minimum depth of 1 m below the surface.

- The provisions for protecting mains in the above cases shall be one of the following:

  a) Concrete surround 150 mm thick for road crossings;

  b) Steel pipe casing having a diameter 150 mm larger than the main, for railway crossings.

- Along major highways, rider mains should be installed along each side of the road to avoid the necessity to lay service connections across it.

Position the main to avoid other services with the following criteria:

- Mains should be laid with a horizontal clearance of at least 3 m from any sewer. Where this is not possible the bottom of the water main should be at least 0.5 m above the top of the sewer or the sewer should be constructed of watertight materials and joints equivalent to water main standards.

- Mains running parallel to underground cables are to be located at a minimum distance of 1 m away from the cable.

- Mains crossing underground cables shall generally be laid under such cables at a minimum depth of 1 m below them.

- Mains running parallel to or near overhead lines shall have a clearance of at least 1.5 m between the base of supporting poles and the wall of main trench.

- The position of the main relative to overhead lines shall allow easy access for maintenance and repair the main.

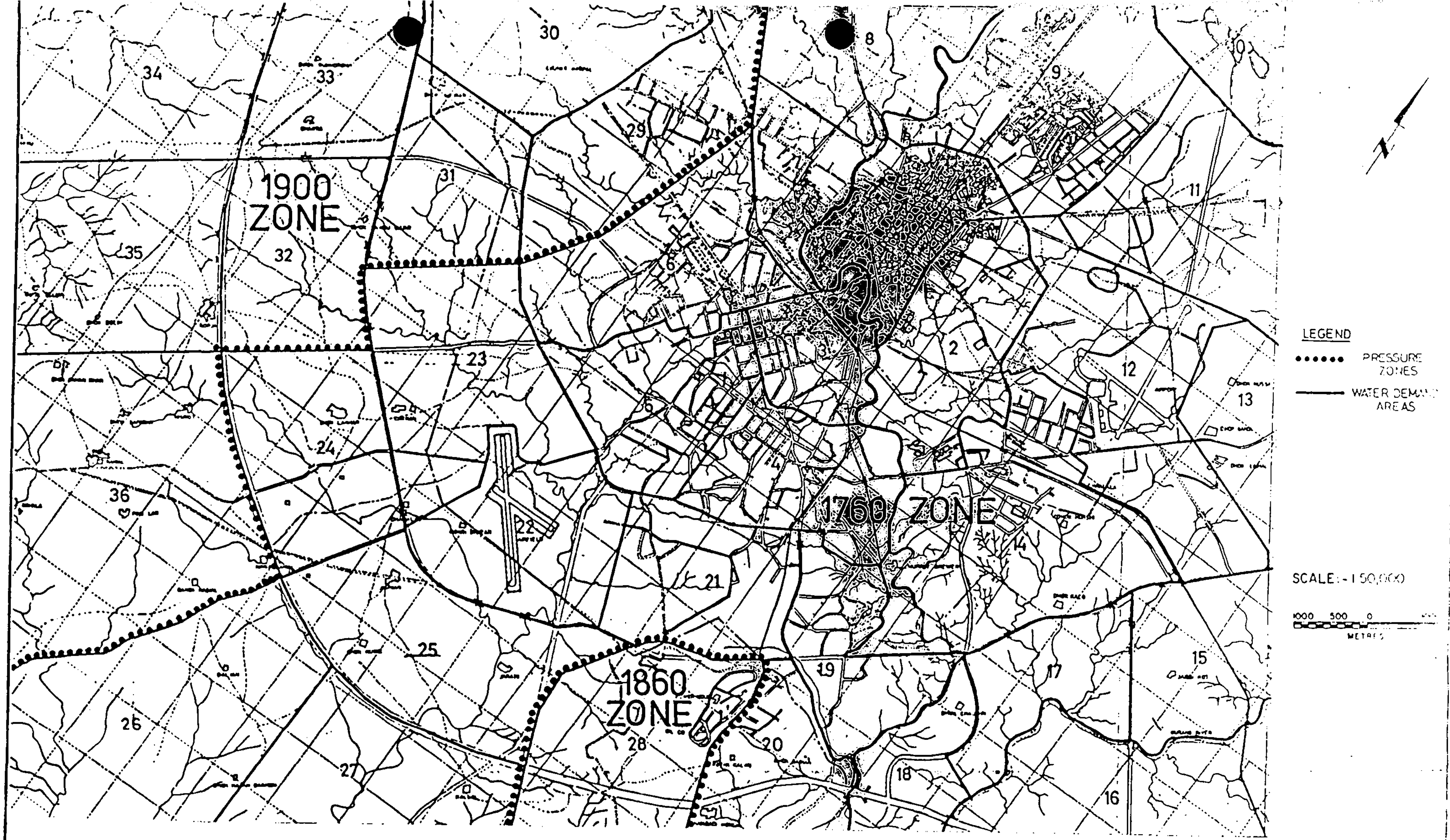
- Mains to be laid alongside open drains shall be laid at a distance of at least 1 m from the nearest side of the drain.

8.1.3 Water Demand

Section 3.2 includes basic data for calculation of water demands, and a procedure for systematically surveying and calculating the water demand in each section of the distribution system is given in Annex C.

This detailed survey procedure would be too detailed for use in a larger urban system, where it is more usual to calculate domestic demands on an area basis using population densities, and add on factors or specific amounts for non-domestic consumptions. Each area of urban development would contribute to the demand at an adjacent demand node in the idealised system. Different rates of per capita demand (litres/capita day) could be applied to different values of population density (persons/hectare) depending on the various housing classifications or zones in the urban area concerned; e.g. high, medium and low density residential. An example of such a water demand projection is given in Table 8.1, for the city shown in Figure 8.1.

Schemes should be designed for 24 hours supply to consumers. Typical peaking factors to apply for water demand, for transmission and distribution works, are given in Section 3.2.5.



EXAMPLE OF WATER DEMAND AREAS AND PRESSURE ZONES

FIGURE 8.1

Table 8.1

Example Calculation of Water Demands by Distribution Zone

Pressure Zone and Water Consumption Area	1980 Population	1980 Water Demand (ML/d)	1990 Population	1990 Water Demand (ML/d)	2000 Population	2000 Water Demand (ML/d)	2030 Population	2030 Water Demand (ML/d)
1970 Zone								
Area 1	160,760	21.5	165,600	25.7	165,600	32.7	165,600	50.9
2	104,100	13.9	116,920	18.2	128,220	25.3	128,220	39.4
3	49,050	6.6	50,050	7.8	50,050	9.9	50,050	15.4
4	34,460	4.6	39,350	6.1	43,740	8.6	45,000	13.8
5	45,840	6.1	59,220	9.2	66,900	13.2	66,900	20.6
6	19,180	2.6	33,710	5.2	43,590	8.6	43,590	13.4
7	40,260	5.4	52,680	8.2	52,680	10.4	52,680	16.2
8	37,730	5.1	56,250	8.7	56,250	11.1	56,250	17.3
9	78,320	10.5	128,160	19.9	142,400	28.1	142,400	43.8
10	20,240	2.7	34,160	5.3	37,950	7.5	37,950	11.7
11	70,070	9.4	114,660	17.8	127,400	25.2	127,400	39.2
12	-	3.9	-	7.9	-	11.8	-	15.8
13	2,660	0.3	7,980	1.2	15,960	3.2	15,960	4.9
14	35,200	4.7	44,000	6.8	52,800	10.4	52,800	16.2
15	-	-	-	4.2	-	6.4	-	8.5
16	5,690	0.8	11,380	1.8	22,760	4.5	34,140	10.5
17	-	-	-	-	-	-	-	-
18	400	0.1	540	0.1	670	0.1	3,350	1.0
19	-	-	-	-	-	-	-	-
20	4,440	0.6	6,120	0.9	9,420	1.9	18,820	5.8
21	7,760	1.0	10,780	1.7	17,240	3.4	25,860	8.0
22	8,930	1.2	12,400	1.9	19,840	3.9	29,760	9.1
23	6,600	0.9	13,200	2.0	29,700	5.9	39,600	12.2
24	4,110	0.6	8,220	1.3	18,500	3.6	24,660	7.6
25	11,430	1.5	15,880	2.5	25,400	5.0	38,100	11.7
26	5,620	0.7	7,490	1.2	9,360	1.8	46,800	14.4
27	3,190	0.4	4,250	0.7	5,310	1.1	26,550	8.2
Total 1760 Zone	756,040	105.1	993,000	166.3	1141,740	243.6	1272,440	415.6
1860 Zone								
Area 28	9,430	1.3	13,050	2.0	20,470	4.1	35,850	11.0
1900 Zone								
Area 29	17,050	2.3	29,970	4.7	40,530	8.0	40,530	12.5
30	12,430	1.7	32,210	5.0	59,330	11.7	73,450	22.6
31	6,800	0.9	16,100	2.5	31,740	6.3	40,390	12.4
32	6,070	0.8	12,140	1.9	27,320	5.4	36,420	11.2
33	10,360	1.4	26,850	4.2	49,460	9.8	61,230	18.8
34	4,890	0.6	6,520	1.0	8,150	1.6	40,750	12.5
35	5,990	0.8	7,980	1.2	9,980	2.0	49,900	15.3
36	6,120	0.8	8,160	1.3	10,200	2.0	51,000	15.7
Total 1900 Zone	69,710	9.3	139,930	21.8	236,710	46.8	393,670	121.0
Total all Zones	835,180	115.7	1145,980	190.1	1398,920	294.5	1701,960	547.6

Notes:
1. Non domestic consumption and losses are distributed throughout the area based on population. Demands in Areas 12 and 15 are on the basis of 2 ML/d per 100 ha in year 2030.
2. Water consumption area and pressure zone boundaries are given on Figure 8.1
3. Water demands are for average day.

Projected Water Demand by Pressure Zones and Water Consumption Areas

- 6 -

8.1.4 Valving

Isolating valves on transmission mains should be installed at intervals of about 1.5 km, and about 0.5 km intervals on primary mains to suit requirements of air valves and washouts. On secondary mains, isolating valves should be provided at every branch connection, street junction and where required by special circumstances.

At every pipe intersection the number of isolating valves should be (n-1) where n is the number of pipe arms at the intersection. One isolating valve should be provided at all of the following points:

- o on by passes;
- o at hydrant connections;
- o at washouts;
- o at air-valves;

Valves and hydrants in a system should close the same way, usually clockwise closing. Valves on mains of 300 mm and smaller should be of the same size as the main. For economy, isolating valves on larger mains may be smaller than the main as follows:

350-400 mm main	- 300 mm valve
450-500	400
600-700	500
800-900	700

Isolating valves on branches for air valves, hydrants, washouts and by-passes should be of the same size as the branch pipe. By passes should be provided on all valves of 400 mm size and over.

For smaller rural or semi urban systems, where it is required to miminize the number of valves in a system, cross-connections between branch lines and loops should be considered where possible and emergency shut-off valving should be by service zones rather than for individual lines.

8.1.5 Supply Pressures and Zoning

Give careful consideration to dividing the system into separate pressure or distribution zones supplied from service reservoirs at different elevations. This can usually be done initially on the basis of topgraphic contours at say, 60 m, intervals. Figure 8.1 shows an example of water demand areas and pressure zones.

Design for the following minimum residual pressures throughout the system at peak hourly flow:

	Minimum Residual Pressure
Rural or semi rural systems	6 m
Small urban systems	9-12 m
Medium urban systems (dwellings (up to 3 storey height)	20 m
Large urban systems	30 m

Maximum static pressures will depend on the topography, being higher in hilly areas - try to maintain a maximum of 60-80 m in flat areas and 80-100 m in hilly areas.

If necessary, use pressure reducing valves or break pressure tanks for low lying areas where static pressures would be excessive.

8.1.6 Standposts

A single tap public standpost supply should be used for about 100 persons and the walking distance should, whenever possible, be limited to 150m. In cases where supply is limited, cistern standposts for 700-800 persons each may be provided at distances up to 500 m. The number of users per tap should be limited to about 100 and in order to avoid crowding near a standpost, the number of taps should be limited to 2 taps per standpost. Discharge at a tap should be between 0.2 and 0.3 l/s.

The location of standposts should be selected in consultation with the local authority. In order to derive revenues to operate the water system, standpost supplies should be limited to low-income users who cannot afford house connections. In commercial areas, for instance, where store-owners could afford house connections, standpost locations should be discouraged. Standposts in residential areas should be readily accessible and at a spacing of up to 300 m or less as required by population density. The standpost location should be protected from vehicular traffic and at a safe distance from the roadway.

Proper drainage should be provided to prevent ponding of water around the standpost, erosion damage, or creation of an unhealthy environment. If required, provide a drain pipe to convey waste water to a suitable location, in addition to a curb around the apron.
(Ref: Rural Water Manual D1).

8.1.7 Hydrants

Hydrants are required in urban schemes and should be of the below-ground type to British standards. Hydrants should normally be spaced at intervals of 300 m in suburban residential areas and 200 m in downtown cores and suburban centres.

Factors determining the location of fire hydrants will include the following:

- o easy access at street intersections, lengths of fire hose lines not to exceed 150 m, value of property and degree of fire risk;

- o 2 hydrants per 1,000 persons should be provided for smaller urban schemes, with up to 4 in larger urban schemes;

- o if fire protection has to be given, this will generally be found to have an overriding influence on the size of the main necessary. The rate of flow required in the main and the pressure required at the hydrants will vary according to the fire risk involved and the local Fire Department should be consulted. The minimum rate of flow required to deal with a substantial fire in a small house is about 15 l/s. If mobile fire pumps are to be used, the flow required to supply one pump is from 25 to 75 l/s.

8.2 Methods of Analysis

The procedure for calculating or estimating the demand at each street (or part thereof) of the supply area has been described in Section 8.1.3. A schematic representation of the pipe system should be drawn up, with numbered pipes and demand nodes (see Figure 8.2).

In a simple system or zone served by one reservoir, the reservoir will be the input node at a fixed head. The next step is to make a rough estimate, using charts, of the flow in each pipe so that approximate pipe sizes can be assigned. This can be done by starting at the part of the system furthest from the reservoir and making assumptions regarding the contribution from each pipe to each nodal demand.

Next, tabulate pipe data as follows:

- o length (m);
- o internal diameter (mm);
- o Hazen-Williams C factor.

and node data as follows:

- o demand or input (l/s, or other units);
- o elevation above given datum.

EXAMPLE OF SYSTEM MODEL

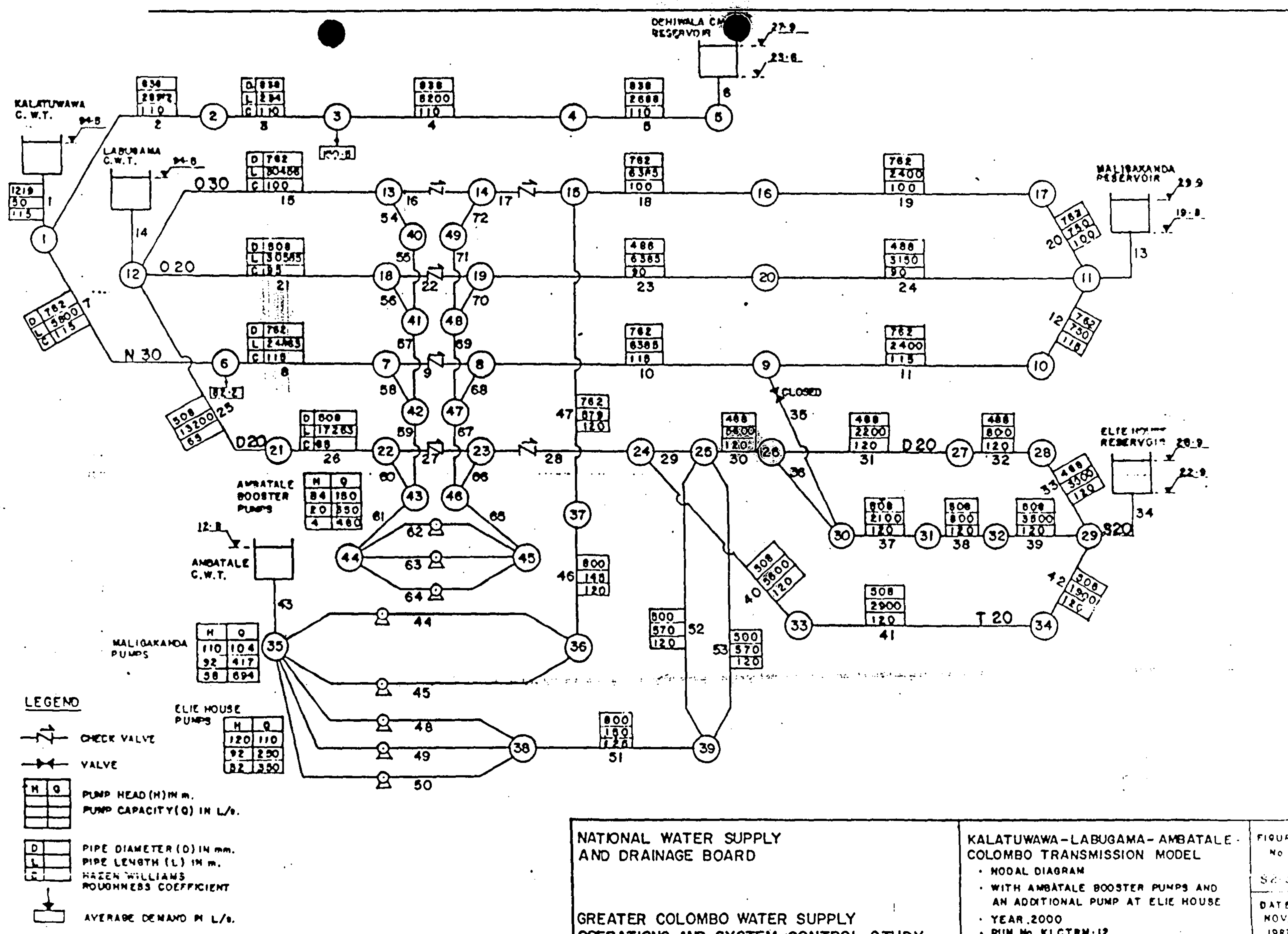


FIGURE 8.2

Note that the total demand from the system must equal the total input. For reservoirs, the top and bottom water elevations should be known, and if possible for pumping stations and booster stations, the pump characteristic curves. Some software programmes, such as KPIPES are able to model fairly complicated system configurations including pumps, valves and check valves, head restrictions, pressure regulating valves and reservoirs, and to perform step by step simulations of system performance over a period of time. Such programmes are suitable for modelling performance of existing systems.

For most new system analysis such sophistication is not required and more straightforward programmes such as LOOP and FLOW are adequate to model the basics of the system. There is a danger, in trying to develop too complicated a system model, that too much confidence will be placed in the results. Remember that the purpose of the model is to aid design - if the system you develop is too complicated to be modelled adequately using LOOP and FLOW then perhaps the system will be difficult to operate in practice, and should be simplified by zoning, avoidance of pumping directly into the system, or other means. A simple distribution system supplied by gravity from service reservoirs is straightforward to model, and to operate. (See also Section 9.3 for alternative system and reservoir configurations).

Having tabulated the basic data, these must then be input to the computer (in the format applicable to the software programme in use) and the model analysed for various conditions which will depend on the actual system configuration. Generally, it is necessary to model the following cases for the design year and for any interim construction stage years in which new distribution pipes are added:

Case 1 Maximum day flows, to show the normal system operation and pressures.

Case 2 Peak hour flows, to show the maximum demand conditions and low pressures.

Case 3 Minimum night flows, to show the minimum demand conditions and high or static pressures.

Case 4 Maximum day + fire flows, to show typical fire emergency conditions (if required).

Consideration should be given to the service reservoir level to be adopted for each case - it would be unrealistic for most sytems to assume constantly empty or constantly full reservoirs and the following could be adopted:

Case 1 Reservoir full

Case 2 Reservoir half full - the probable water level at the end of the maximum supply period.

Case 3 Reservoir full - the probable water level at the completion of night-time replenishment.

Case 4 Reservoir at 1 m above empty - the severest condition during a major fire.

The assignment of fire flow locations and quantities will have to be ascertained depending on local Fire Department requirements. Fire flow demands may well necessitate an increase in pipe sizes over those required for peak hour flow. The locations chosen for fire demands will normally be at strategic points in each zone. (See Section 8.1.7)

It may also be necessary to consider variations in the C-values of pipes to represent deterioration in carrying capacity over time.

The preliminary pipe diameters and system configuration should be modified after running each set of cases to obtain an optimum system design by trial and error, which meets the design criteria for all cases. For a straightforward system this can usually be done after only 2 or 3 trials.

8.3 Leak Detection and Repair

This is a subject which is often overlooked as being unimportant, or bypassed in favour of development of new sources to meet increasing demands. However, it is a fact that in most systems 20% or more - even up to 50% - of the supply is lost through leakage and waste from mains, valves, reservoirs, pumping stations, service connections and on consumers' premises. In a correctly operated and maintained system, such losses should not exceed 10-15% of production.

This loss can amount to a considerable sum of money whether in terms of the cost of production (usually Rs.2 to Rs.5/m^3) or in terms of loss of revenue had this water been available for sale to consumers.

Waste of water on consumers' premises after the meter is in theory not a monetary loss, but is a waste of a usually scarce resource which should be minimised - often, however the loss becomes monetary if the customer meter is defective or if the customer successfully contests his high water bill: in either case the situation is unacceptable.

When a scheme reports a shortage of supply it should automatically become the practice to send out a leak detection crew to locate and repair leaks in the system, either before or as a part of any rehabilitation or extension work. The reduction of system leakage and waste could well result in an extension or augmentation project being deferred: this would usually be of economic benefit. The economics of leakage control are not discussed here, but have been well documented (Refs. 26, 27).

The starting point in any survey is to know how much supply is entering the system, preferably zone by zone, and this will usually require reading existing meters or carrying out pitometer or insertion flow meter surveys. An analysis of billings should readily show actual consumption and indicate the level of unaccounted-for water (UFW). Note, however, that this also will include "billing losses" due to meter inaccuracies, meter reading errors and billing faults, all of which may amount to a significant proportion of total UFW.

The actual leakage/waste surveys would include the following:

- o loss surveys at treatment plants - measure leakage, filter washwater, overflows, etc;

- o leak sounding surveys and inspections of transmission mains (eg air valves, washouts, unauthorised connections);

- o surveys of overflow and leakage from service reservoirs (study operation, perform leakage test);

- o survey standposts for use and correct metering;

- o survey pump stations for leakage;

- o leak sounding surveys and inspections of distribution mains;

- o house inspections for detecting leakage on consumers' premises.

Methods of leakage control applicable in Sri Lanka are summarised as follows:

- o Visual observation, sounding (preferably at night) and repair: in dry weather periods, inspectors can readily observe leakage points due to presence of water, seepage or green vegetation along pipeline routes. Sounding, with sounding rods, on all stopcocks, valves and hydrants listening for the characteristic noise of leaking water is also effective. Necessary repairs should be carried out without delay.

- o district or zone metering: records are kept of the flow into each distribution zone - in the event of an unexpected increase in zone consumption, special surveys could be carried out in that zone to identify the cause.

More sophisticated methods, such as waste metering, and instruments such as leak noise correlators are not suitable. Waste metering involves a fair amount of input in the form of manpower, equipment and distribution facilities (meter chambers, isolating valves etc) and is suited more to non-metered systems. Electronic leak detectors are of limited value and the effectiveness of surface sounding is limited by traffic noise, the depth of pipes, and low system pressures. Sounding at night is preferable due to increased pressures and reduced traffic noise. More details of leakage control methods and equipment are given in Annex O.

Other important points should be noted:

- o high system pressures will lead to increased pipe bursts, and increased leakage rates - pressures should be minimised as much as possible while providing an acceptable supply.

- o accurate mapping of the distribution system is a necessary requirement prior to commencing a leakage survey.

- o leak noise does not travel so well in PVC and other non-metallic pipe, as it does in steel or iron pipe. Nor can such pipes be located with metal pipe locators, which highlights the need for accurate mapping. Special equipment is available for PVC pipe location.

8.4 Rehabilitation of Mains

It may sometimes be cost effective to clean and reline old mains rather than replacing or augmenting them with new mains.

The condition of a main can be evaluated by carrying out a pressure and flow test, as follows:

- o select known length of main to be tested;

- o install pressure tappings at each end of pipe length;

- o install accurate pressure gauges (the gauges should be checked before and after each test by a deadweight tester);

- o survey accurately the difference in elevation between the 2 pressure points;

- o install a flow measuring device - eg. bulk meter, pitometer, insertion flow meter;

- o measure instantaneous pressure loss over test length under measured (constant) flow rate;

- o repeat with changed flow rate if possible;

- o calculate Hazen Williams C-factor.

If possible, inspect the inside of the pipeline by removing a section or valve, for example. Try to relate the actual condition of the internal surfaces to the calculated C-factor.

If the C-factor is inexplicably low, it may be that there is some obstruction or partly closed valve in the pipeline - it may then be necessary to repeat the test over shorter sections, to isolate the problem area.

Mains can be cleaned or relined using various techniques. Pipelines frequently build up sediment deposits which may be removed by periodic flushing. When flushing is not adequate, a spring-loaded instrument (sometimes called a pig) with steel scrapers can be forced through the line by water pressure or by a winch. Sometimes several spongy polyurethane foam balls of increasing size are sent through a pipe for cleaning. During cleaning, temporary service should be provided to customers with a pipe laid above ground.

Tuberculation and scale deposits can be removed by the use of mechanical or hydraulic scrapers. The scrapers remove the tuberculation and existing coatings by a honing action. Care must be exercised after each scraping to ensure that deposits removed are flushed away and not allowed to enter service connections.

After scraping, pipes can be lined in place by the application of a cement mortar lining. A thin lining of 6 mm or less is preferred in order not to reduce the inner diameter of the pipe too seriously. There are three ways of applying the lining: centrifugal method, reinforced centrifugal method, and mandrel method. Pipe that has been scraped is almost always lined afterward because, after scraping, tuberculation will occur at a greater rate than before. Cleaning and lining of pipe should result in lower pumping costs, increased flow, decreased pressure drop and therefore, increased water pressure to the customer. The process is normally carried out by specialist contractors.

Tests to calculate the Hazen Williams C-factor should be carried out before and after the cleaning/relining process to evaluate the improvement achieved.

8-14

9. SERVICE RESERVOIRS.

9.1 Function

Service reservoirs provide a suitable reserve of treated water to minimise interruptions to supply due to failure of power, supply mains, pumps, etc. They also enable the system to meet the widely fluctuating demands when supply is by constant or intermittent pumping. Service reservoirs are therefore helpful in reducing the size of transmission mains which would otherwise be necessary to meet the peak rates of demand.

The two main functions are therefore

- o emergency storage;
- o equalizing storage.

9.2 Storage Capacity

The amount of emergency storage to be provided depends on the relative importance of maintaining distribution flow in the event of a breakdown, and the likely extent or duration of such breakdowns. An amount of emergency storage is necessary in most urban schemes, but is of less importance in smaller urban and rural schemes. In the event that, in order to economize on the capacity of distribution storage, only equalizing storage is provided initially, land acquisition at storage sites should allow for future storage requirements.

The amount of equalizing storage depends on the rate of inflow to the system and the consumptive pattern throughout the day. Local patterns of water used will produce hourly variations in the rate of drawoff. A mass diagram should be constructed showing the draw off pattern against the inflow rate to determine the volume of equalising storage required. For most schemes, however, the drawoff pattern is not known or is affected by shortages, low pressures or restricted supply hours: in such cases an estimate must be made of the demand pattern.

In the absence of actual field data, the following pattern of consumption may be assumed until data becomes available:

Time	% of Daily Demand	Time	% of Daily Demand
5- 6 am	5	2- 3 pm	3
6- 7	12	3- 4	4
7- 8	10	4- 5	8
8- 9	8	5- 6	8.5
9-10	4	6- 7	8
10-11	4	7- 8	3
11-12 noon	4.5	8- 9	2
12- 1 pm	8	9-10	1
1- 2	7	10 pm - 5 am	0

For smaller urban and rural schemes, the period of supply inflow should be carefully considered to minimize storage. It should be made to coincide preferably with the drawoff period especially during high consumption hours. As a general guide and pending field data the following Table may be used in estimating storage requirements in rural and small urban communities for different periods of supply.

Daily supply inflow period (hours)	Storage as % of maximum day demand
18–24	50
12–18	33
9–12	25

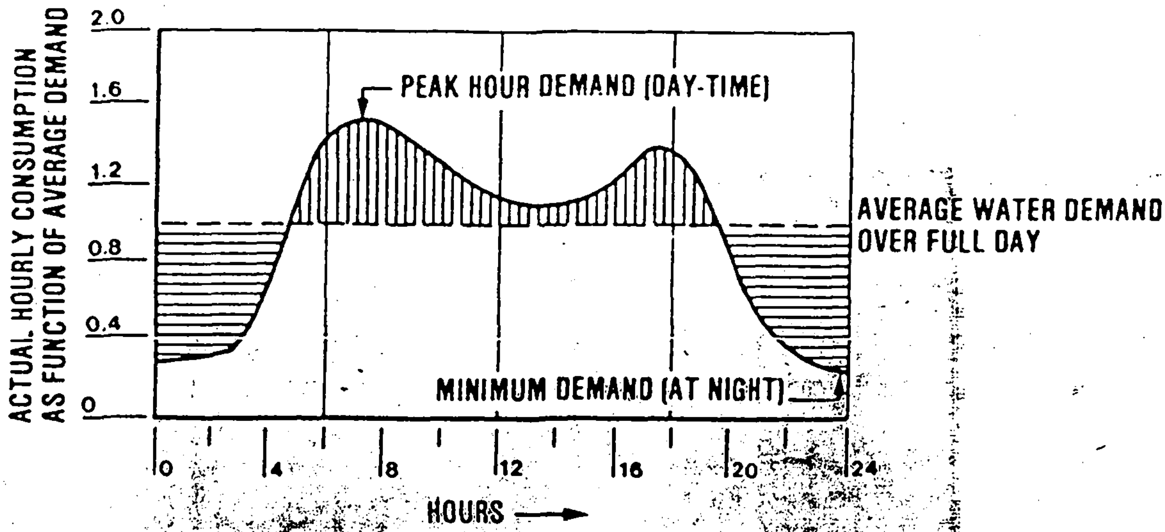
In pumped supplies, the cost of storage must be balanced against the cost and reliability of pumping. For instance, if water is pumped over a 24 hour period, storage will be required to store water pumped in the night when there should be virtually no consumption. If water can be pumped during the consumption periods at a higher rate than the average rate for the day, storage and pump operation costs could be significantly reduced; but pump costs would be higher. In all schemes, the cost of supply mains and pumping plants should be balanced against the cost of storage. Where the supply main is short, it may be economical to lay a larger capacity main provided there is sufficient yield from the source. Storage at the source is often required when the yield of the source is insufficient to meet a higher pumping rate which has been selected to minimize the pumping period.

Notwithstanding the above, most schemes involving treatment or having a limited source will require continuous 24-hour operation in order to optimise the treatment process or obtain the necessary source yield. Slow sand filters, in particular, should not operate on an intermittent basis.

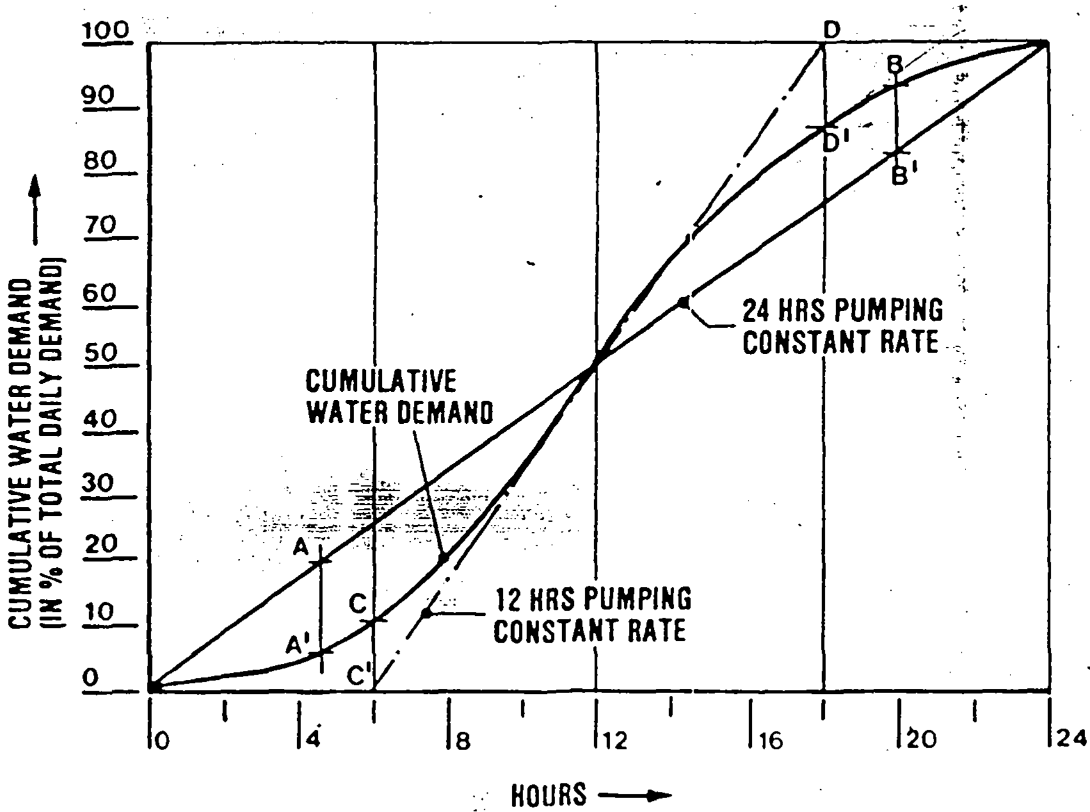
In general, the larger the system, the less hourly fluctuation there will be in distribution system flows, and the less equalising storage will be required, but this would normally be offset by the additional need for emergency storage.

An example calculation of equalising storage is given in Figure 9.1. For a constant rate supply, 24 hours a day, the required storage is represented by A-A' plus B-B', about 28% of the total peak day demand. If the supply capacity is so high that the daily demand can be met with 12-hours pumping a day, the required storage is found to be C-C' plus D-D', about 22% of the total peak day demand.

For larger systems, a rule of thumb guide for storage capacity would be to provide a minimum of 8 hours demand on a maximum day, (or 33%) increasing up to 24 hours demand (100%) where emergency storage is important.



VARIATION OF WATER DEMAND DURING THE DAY



GRAPHICAL DETERMINATION OF REQUIRED RESERVOIR STORAGE

SERVICE RESERVOIR CAPACITY

FIGURE 9.1

9.3 Position and Elevation

If the storage is to be of maximum value as a safeguard to the community against breakdown, then it should be positioned as near as possible to the area of demand. From it, the distribution system should spread directly, with such arrangement of mains that if a breakdown of any one main occurs, a supply could be maintained by re-routing the water. Storage must be situated at a higher elevation than any part of distribution area. If such a site is available only at some distance, the reservoir should be placed there.

In flat areas where no suitable high points for ground reservoirs are available, water towers or elevated tanks have to be used. In practice, water towers or elevated tanks have relatively small volumes because they are more costly to construct than a ground reservoir. Reservoirs, depending on the relative locations and elevations of source and demand area, may either supply the distribution by gravity or 'balance' on a pumped system (see Figure 9.2). The former method is preferred - See Figure 9.3 for the advantages and disadvantages of each.

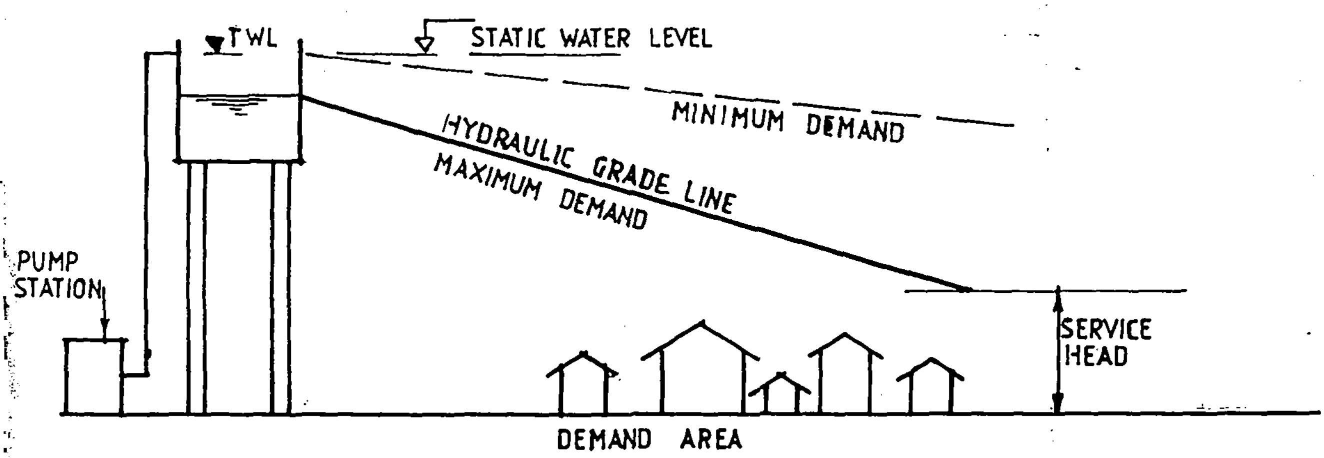
In large systems, or when the source or treatment plant are distant, rechlorination of the water may be necessary at the distribution service reservoirs.

9.4 Design Considerations

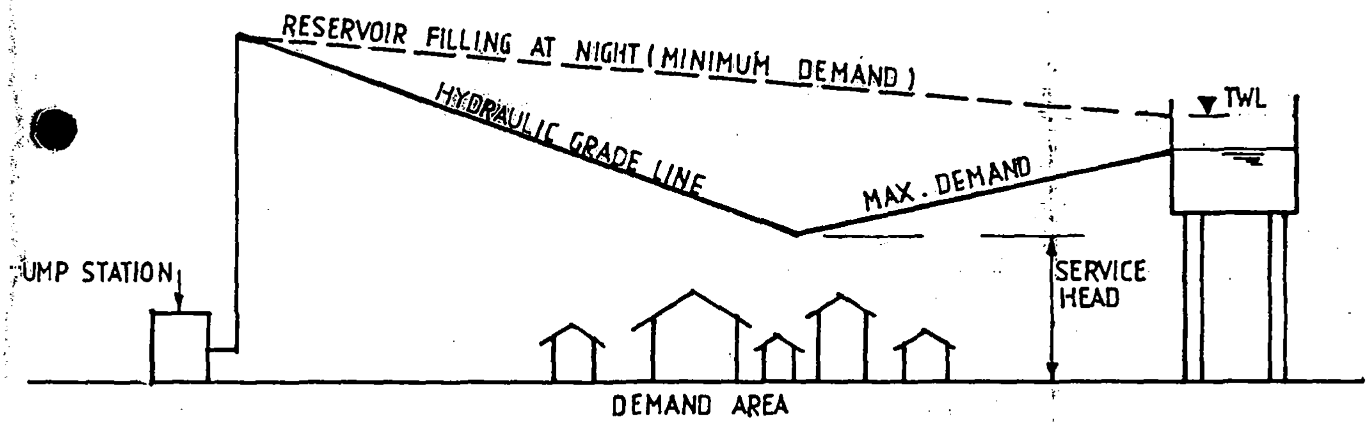
For considerations of shape, size, type and cost refer to report of Research/Design Study on Service Reservoirs by University of Moratuwa, 1989. (not yet available in March 1989)

Other factors should be considered, as follows:

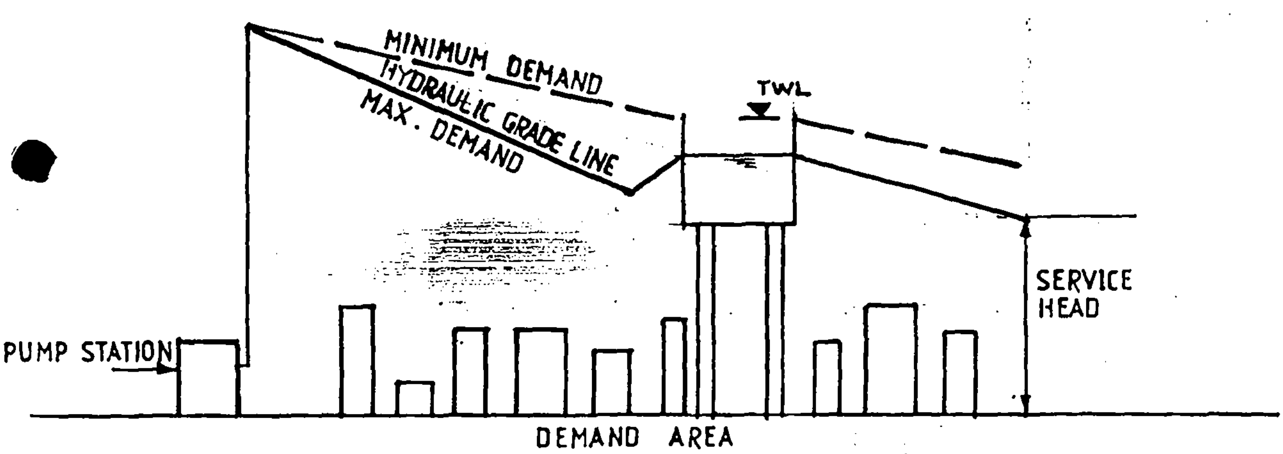
- o water depth is normally 2.5 - 5.5 m;
- o provide bell-mouth draw off pipe a few cm off floor provided with strainer (preferably of perforated cast iron);
- o provide ball/float valve to shut off inlet when tank full;
- o provide adequately sized outflow and drain to discharge maximum flow that can be delivered to reservoir;
- o provide a fall on floor to drain sump and access to drain to permit easy flushing-out of accumulated sediment on reservoir floor;
- o position inlet and outlet to avoid short circuiting and stagnation of water;
- o provide bypass piping around reservoir;
- o provide 2 compartments if possible, each with separate inlet and outlet. Scour and overflow from each may be connected to a single line;



STORAGE NEAR SOURCE



STORAGE OPPOSITE SOURCE

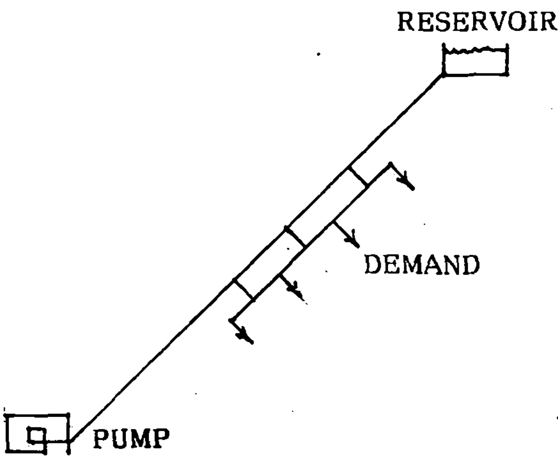


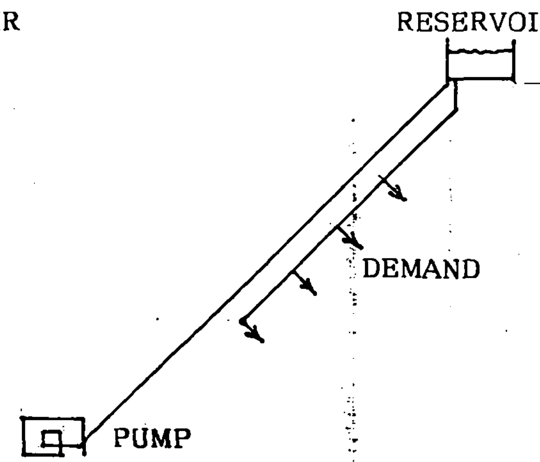
STORAGE IN DEMAND AREA

STORAGE LOCATION

FIGURE 9.2

ALTERNATIVE A	ALTERNATIVE B
System Supplied From Pumping Main	System Supplied From Gravity Main





ADVANTAGES

Alternative A	Alternative B
Lower pipeline cost. Slightly lower pumping cost.	Lower, more constant pressures - less leakage. Constant head on pumps and constant flow rate - simple operation. Supply line only vulnerable to pressure surges - no damage to distribution pipes or meters.

DISADVANTAGES

Alternative A	Alternative B
Higher more variable pressures - more leakage. Variable head on pumps and variable flow rate - less simple operation. Whole system vulnerable to pressure surges - possible damage to pipes, meters, etc.	Higher pipeline cost. Slightly higher pumping cost.

Comparison of Separate or Combined Pumping and Distribution Mains

FIG. 9.3

- side walls should be designed as partition walls when expansion of the reservoir in future is anticipated.

- land acquisition should be sufficient for future expansion of storage;

- provide screened ventilation to each compartment;

- provide adequately sized access manholes with raised curb and locking covers and ladders to each compartment for inspection and maintenance;

- a low level inlet with non-return valve is preferable where delivery head is limited;

- provide stop valves on inlet and outlet;

- consider differential settlement;

- provide the following instrumentation;

  - locally fabricated float operated board gauges for indicating reservoir level (units should be metres);

  - accurately calibrated pressure gauges for any remote indication of reservoir level. Units should be metres head of meter;

  - propeller meters for indicating totalised flow from reservoirs. (Rate of flow indication not necessary). Units should be cubic metres.

10. COST ESTIMATES AND ANALYSIS

10.1 Cost Items

Cost estimates are required to ascertain the amount of capital investment initially needed and the recurrent operation and maintenance charges after commissioning the scheme. Rate structures which would generate sufficient revenues are based on the amount of debt or loan repayment and the cost of O&M. Accurate cost estimates are essential for making sound economic comparisons of engineering alternatives and for selecting the most cost-effective scheme for implementation.

10.1.1 Capital Costs

Preliminary and final construction cost estimates for each component of a water supply scheme (e.g. intake, transmission line, reservoir, distribution, etc) should be itemized in detail. Materials and labour should be listed separately. Cost schedules should include type of materials to be used, size, quantity and the unit rate. In addition, for pipeline construction, permanent road re-instatement and disinfection are listed as separate items. Where electric power supply is required the cost of the sub-station and/or transmission line should be obtained from Ceylon Electricity Board. In addition to the estimated construction cost (ECC) the following items where applicable need to be included in the cost estimate:

Preliminaries: This item includes temporary office, stores, and transport of materials. Allow ½ to 1% of ECC, the larger percentage used for smaller schemes.

Land acquisition: Land cost prevailing at the District level should be included, if to be acquired for the scheme.

Landscaping: Allow an adequate amount for clearing up and landscaping site including provision of grass, trees and shrubs.

Maintenance of schemes for 2 years upon completion: This cost includes labour, energy and materials such as chemicals and equipment to start-up and operate the scheme prior to handing over to the local authority (if so decided between the Government and the local authority).

CIF cost: Cost of purchase, insurance and freight of imported materials.

For customs duty: See Costing Section bulletin.

10.1.2 O&M Costs

In order to establish a rate structure, the cost components of the loan repayment and operation and maintenance must be as accurate as possible. Capital cost estimates must reflect the actual cost of construction. O&M costs should include labour, electricity and/or fuel, chemicals, repairs and a contingency amount. The following general guidelines are given in allocating these cost components.

Labour: As a minimum, a full-time caretaker/operator is included for each scheme. Where the operation is in excess of 12 hours per day, additional caretakers/ operators would be required. To assure an operator at all times, a replacement for a period of one month per year is also included. One casual labourer, one week per month would be needed for maintenance of each scheme except for slow sand filter plants which would require 4 labourers for a period of about two weeks every two months. Additional labour inputs would be required depending on the scope of the scheme. Consult O&M Dept. for more detailed information on specific schemes, and refer to Annex P, which shows typical staff cadres for different types of scheme.

Electricity and fuel: The cost of energy to operate a water supply system can be estimated by using the prevailing tariffs of the Electricity Board and the Petroleum Corporation. For electricity costs, a demand charge using a power factor of 0.85 has to be included in addition to the unit charge. Lubrication costs for engines would be included under this item.

Chemicals: The costs of chemicals for chlorination and for treatment processes should be as accurate as possible as they are often the most significant cost items. The chemical prices should be current and should include transportation and handling costs. For disinfection by chlorination, a dosage of 2 mg/l may be assumed.

Minor repairs: For mechanical equipment such as pumps and motors, it is recommended that about 1% of the installed equipment cost be put aside for repairs and maintenance. Where equipment needs to be replaced on a periodic basis, sufficient funds would have to be set aside by the local authority for this purpose. In addition, a suitable sum should be put aside for miscellaneous repairs to piping, valves and structures.

Contingency: In order to meet unforeseen expenditure, 10% of the above total costs is recommended as a contingency amount.

10.2 Format of Estimate

The Costing Section of Planning & Designs Department prepares periodic bulletins of unit costs to be adopted for all projects.

In general, cost estimates should be to the following format:

Capital Costs (at base year)

1.	Line items to include contractor's overhead and profit @	35%
2.	Include 2 years O&M cost and recoverable connection costs if applicable	
3.	Subtotal of Line Items	
4.	Engineering design and construction supervision* (Investigations & Design - 6% Supervision - 2%)	8% of (3)
5.	Contingencies (physical) Feasibility Stage	10%-15% of (3)
	Pre-Feasibility Stage	20% of (3)
6.	BTT	3% of (3)
7.	Total Capital Cost	
8.	Price escalation for funding, relative to to base year	10%-15% p.a.

O&M Costs (at base year)

1.	Line items to include NWSDB/LA overhead @	35%
2.	Projected costs for 20 years	

Present worth analysis should be on base year costs without price escalation.

Ensure that the following facilities have been provided for in the design contract documents and cost estimates:

- o Landscaping
- o Security and fencing
- o Laboratory services and equipment;
- o Communication facilities, telephone, radio;
- o Domestic water supply, electricity, site access roads;
- o Quarters

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* See also GM's circular on Construction Services to Clients, August 25, 1988.

- o Transport service, if site is remote;
- o Chemicals for start-up;
- o Access/safety facilities (ladders, hand rails);
- o Protective equipment/clothing, first aid kits, gloves, boots, etc.
- o Spare parts;
- o Office furniture, stationery, etc.
- o Storeroom.

10.3 Comparative Cost Analysis

For a valid comparative cost analysis, the capital costs and recurrent costs have to be converted into comparable units. Two methods can be used for this purpose.

- present value;
- equivalent annual cost.

10.3.1 Present Value Method

The present value of future recurrent costs can be calculated by discounting them at an appropriate rate, the discount rate. If there is no price inflation, the prevailing interest rate, if not regulated by governmental control measures, may be used as the discount rate. For example, to cover a cost of Rs.990 for maintenance in the following year, it is sufficient to set aside Rs.900 invested at an interest rate of 10% and if the expenditure of Rs.990 is required after three years. Rs.740 invested at 10% would be adequate. Present value calculations are more complicated where price inflation occurs, because then it is necessary to adjust the prevailing interest rate for inflation in order to arrive at a realistic discount rate.

The present value of a cost (C), incurred in the year (n) from the present, can be calculated with the equation:

$$PV = \frac{C}{(1+r)^n}$$

where:

PV	=	present value of cost
C	=	nominal amount of cost incurred n years from present
n	=	number of years
r	=	discount rate

The present values of all recurrent costs to be incurred over the expected lifetime are added to the initial capital cost to obtain a total figure, which can be compared for various altenatives. See Annex N for example.

10.3.2 Equivalent Annual Cost Method

The capital cost is converted into a series of equivalent annual costs over the total lifetime of the scheme. The sum of the annual equivalents will be greater than the initial capital cost of the system, because interest on the remaining debt is included in each annual installment. The equivalent annual cost of a capital investment (C), over an expected lifetime of (n) years, can be calculated with the equation:

$$EAC = C \times \frac{r\,(1 + r)^n}{(1 + r)^n - 1}$$

where:

EAC = equivalent annual cost
C = initial capital cost
n = expected lifetime
r = discount rate

The total annual cost is obtained by adding the annual recurrent costs to the equivalent annual cost, to give a figure which can be compared for various alternatives. This method is recommended for comparative cost analysis of water schemes for the following reasons. Recurrent costs are difficult to estimate accurately, and it may well be necsssary to adjust the cost calculations from time to time. This is more easily done in this method than for the discounted amounts in present value calculations. Another advantage is that the total equivalent annual cost divided by 365 gives the daily cost of the scheme which divided by the daily water output, yields the unit cost per cubic metre of water produced.

The discount rate to be used differs from country to country. In general, the World Bank applies discount rates in the range of 8 to 15% for developing countries. When sufficient data are not available, it is advisable to make calculations for several discount rates to get an indication of the effect of the selected discount rate on the results of the comparative cost analysis.

10.4 Financial Viability

A scheme would be financially viable if it can generate enough cash flow to meet the direct costs and pay back the loan instalments and interest, and costs of replacements necessary during the designed life period of the scheme. In some cases the loan repayment period exceeds the life period of the scheme (e.g loan repayment of 38 years and scheme life of 20 years as in the case of LLD loans). The position is reversed in the case of some schemes funded by international funds on-lent by the Treasury to NWSDB; in others the time spans are approximately the same.

Where the loan term is more than the scheme's life period, the unpaid loan commitment would be added on to the replacement scheme's new loan commitment. Care should be exercised when estimating the salvage value of the component parts of the scheme. Generally it would be very low, except for equipment replaced and used only for a few years. Obsolescence would reduce salvage value and therefore, in practice, salvage value could be zero.

Any grant made by the Government or other institution should not qualify for a return. Economically there should be a return on such funds invested, but for purposes of small community water schemes this factor may be ignored in calculating financial viability. The returns in social benefits would cover this aspect.

Loan repayment terms should be ascertained from the lending agency, and in particular any grace period for interest and/or capital repayment. Interest would be on the reducing balance of the loan and the total capital plus interest payment would normally reduce annually. Some lending agencies provide equal capital + interest payments where, with decreasing interest, there is an equivalent increase in capital repayment.

Loss making schemes should not be ruled out if other factors make it desirable to go ahead with the scheme. In such cases, contributions to decentralised budgets and Treasury grants or increased revenue from other sources should be negotiated by the local authority, before the work commences.

Tariff fixing - Although this is necessarily related to costs of production and distribution, a certain flexibility should be followed. At the present time, NWSDB uses an Island-wide tariff rather than a scheme-specific one, although specific tariffs are being investigated for the ADB project schemes. Bulk tariffs are usually scheme specific. For evaluation of individual tariffs, the total expected revenue may be allowed to be lower than the cash outflow in the initial years, until consumption increases. Cross subsidising a water scheme within the local authority's resources may be justifiable in certain cases, while in others the water scheme's surplus cash may subsidise another activity. These aspects should be discussed with the local authority and a strategy developed in each case. In determining the tariff structure, the tariffs operational in other comparable areas should be considered and rates that are affordable should be adopted. This may be referred to as the principle of "what the traffic can bear."

All direct operating expenses, loan capital and interest repayments and costs of replacements should be included in the cost of production and distribution. It is necessary to avoid socially desirable schemes being loaded with excessive indirect costs and thereby classified as financially not viable, and for that reason not commenced. The principle of incremental costing should be used for determining the indirect costs so as to minimise the load. To determine the amount of indirect costs of the regional offices and head office on this basis, the question should be asked "what extra costs would these offices incur because the subject scheme becomes operational?" For each of the schemes in a region no, or very little additional costs may be incurred. But cumulatively, for the 10th and subsequent schemes, the regional offices and head offices will have to incur extra costs, eg. additional supervisory engineers. To allow for this, 5% of direct operational expenses should be included as indirect costs.

Provision for inflation should be made at rates currently used by Treasury Planning Division. The question of tariff increases to cover inflation should be discussed with the local authorities and only practicable increases should be included.

Cash Flow Statement - Having regard to the above, the estimated annual cash inflow, outflow, annual cash deficit/surplus and cumulative deficit/surplus should be prepared with a column for each year. The data may then be varied to suit various probabilities, and print outs of various combinations obtained very easily. This method is understood more easily by most clients especially local authority administrators and policy makers. It is not proposed that the annual surpluses are reduced to present values as no return over and above the capital and interest repayment is required.

In most cases, if revenues are predicted using the existing NWSDB tariff, projects will not meet the criterion of financial viability as defined (Manual P1 Section 2.2) and the following less rigorous criteria should be considered assuming current costs, NWSDB tariff and current Government financing terms:

a) For schemes with 100% GSL or donor grant funding: (See Table 10.1)

   i) Viability taking into account O&M costs and revenues only, neglecting capital repayment. (viable in terms of O&M cost recovery only).

b) For foreign loan funded schemes with a proportion of loan funding on-lent by GSL (See Table 10.1).

   i) Viability taking into account O&M costs and revenues, actual cost of repaying amount on-lent by GSL.

   ii) Viability taking into account O&M costs and revenue only, neglecting capital repayment (viable in terms of O&M cost recovery only).

Assumptions

1. Sensible projections will need to be made on an individual project basis with regard to the proportion of population served by public standposts and by private connections, as this factor has a marked effect on costs and revenues. These projections must tie in with the scheme production, and with affordability - it is no use predicting 90% house connections if the affordability criterion is not met.

2. The collection ratio, or percentage of bad debts must not be projected too pessimistically. Although in many existing schemes this ratio does not exceed 50%, there is usually a good reason for this, based on problems with the service, such as poor quality or inadequate quantity or pressure of water provided. Assuming a good service of good quality water, and an effective disconnection policy for non-payment, then the collection ratio should rise to 90% by 1995.

Tariffs

Bulk and individual tariffs should be proposed, based on the affordability criterion of 4% of household income as a maximum amount for water. Until such time as consumers' attitudes change the "willingness to pay" should be reviewed seperately from "ability to pay" and the local authority advised on its consequences. In some cases these tariffs will need to be several times the NWSDB tariff. Further developments on future tariffs are proceding, along with the concept of regional (or provincial) bulk tariffs charged to Local Authorities.

Table 10.1

CURRENT GOVERNMENT FINANCING TERMS FOR WATER SCHEMES

For financial evaluation of projects, adopt the following financing terms for capital costs of water schemes:

Funding Source	Urban (VC/MC)	Rural Piped	Rural Non-Piped and Sewerage/ Sanitation
1. GSL Consolidated Fund grants			
Total Capital Cost	100% grant	100% grant	100% grant
2. Foreign donor's funds on-lent by Treasury			
Total Capital Cost	50% grant 50% loan	85% grant 15% loan	100% grant

Notes: 1) Current Government on-lending interest rate to NWSDB is 12% p.a. regardless of terms of donor loan to GSL except in the case of donor commercial loans which may be higher.

2) Based on Cabinet paper No.116 of 1986 (continuation 32) dated February 9, 1986 and Cabinet decision of 8-10-86.

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