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MANUAL ON WATER SUPPLY AND TREATMENT

THIRD EDITION - REVISED AND UPDATED

Prepared by

THE EXPERT COMMITTEE

Constituted by

THE GOVERNMENT OF INDIA

CENTRAL PUBLIC HEALTH
AND ENVIRONMENTAL ENGINEERING ORGANISATION

MINISTRY OF URBAN DEVELOPMENT, NEW DELHI

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FOREWORD

Water is a basic need. The provision of safe and adequate drinking water to the burgeoning urban population continues to be one of the major challenging tasks. Lack of safe drinking water could undermine the health and well being of the people, particularly, the urban poor and economically weaker sections.

The Central Public Health & Environmental Engineering Organisation (CPHEEO) in this Ministry had brought out the 3rd edition of the Manual on Water Supply and Treatment in March, 1991 with a view to provide valuable guidelines to the Public Health Engineering Departments, Water Boards and municipal bodies on the basic norms, standards and latest developments in this field. Subsequent to the publication of the said Manual, the CPHEEO had received comments and suggestions from field practitioners and manufacturers for revising and updating certain aspects, such as water quality, per capita water supply norms, water conservation, metering and availability of various kinds of pipes. A two member Expert Committee was set up by this Ministry in November, 1997 to look into these aspects.

I am pleased to acknowledge the contribution made by the Expert Committee in further revising and updating the Manual, which I am sure would be of considerable help to the Public Health Engineers and field practitioners for proper planning, design and quality control of water supply systems.

(ASHOK PAHWA)
SECRETARY TO THE GOVERNMENT OF INDIA
MINISTRY OF URBAN DEVELOPMENT

PREFACE

The "Manual on Water Supply and Treatment" brought out by the Government of India in 1962 was revised and updated in the late seventies. However, in order to provide the field engineers with information about the latest development in this field during the intervening period, the then Ministry of Urban Development, Government of India constituted an Expert Committee towards the end of 1985 to further update the Manual. The composition of the Committee was:-

1.	Shri V. Venugopalan Adviser(PHEE),Central Public Health & Environmental Engineering Organisation, Ministry of Urban Development, New Delhi	-	Chairman
2.	Shri M.R. Parthasarathy, Deputy Adviser(PHE),Central Public Health & Environmental Engineering Organisation, Ministry of Urban Development, New Delhi	-	Member
3.	Shri S.D. Mundra Director, Geo-Miller & Co. Pvt. Ltd., New Delhi	-	Member
4.	Shri B.P.C. Sinha, Chief Hydrogeologist & Member, Central Ground Water Board, New Delhi Alternate Shri A.R. Bakshi, Scientist, Central Ground Water Board, New Delhi	-	Member
5.	Shri G. Raman Director(Civil Engineering), Bureau of Indian Standards, New Delhi	-	Member

Alternate

Shri A.K, Awasthi,
Deputy Director(Civil Engineering),
Bureau of Indian Standards,New Delhi

6.	Shri R. Paramasivam Scientist & Head,Water Engineering Division, National Environmental Engineering Research Institute, Nagpur.	-	Member
7.	Dr. I.C. Agarwal, Profesor, Deptt. of Civill Engineering, Motional Nehru Regional Engineering College, Allahabad	-	Member
8.	Dr. A.G. Bhole, Professor, Deptt. of Civil Engineering, Visvesaraya Regional Engineering College, Nagpur.	-	Member
9.	Shri B.V. Rotkar Member Secretary, Maharashtra Water Supply & Sewerage Board, Bombay.	-	Member
10.	Prof. V. Chandrashekar, Chartered Engineer & Consultant, Mysore.	-	Member
11.	Dr. S.K. Biswas Deputy Adviser(RWS),Department of Rural Development, New Delhi.	-	Member
12.	Shri S.A. Jagadeesan, Engineering Director, Tamilnadu Water Supply & Drainage Board, Madras.	-	Member

13.	Dr. D.M. Mohan, Superintending Engineer, Hyderabad Metro Water Supply & Sewerage Board, Hyderabad.	-	Member
14.	Dr. R. Pitchai, Professor & Head, Centre for Environmental Studies, Anna University, Guindy, Madras.	-	Member
15.	Shri A.R. Mir, Secretary, Irrigation & Public Health Engineering Department, Government of Jammu & Kashmir	-	Member
16.	Shri S.L. Abhayankar, Technical Adviser, Indian Pump Manufacturers Association, Bombay.	-	Member
17.	Shri V.B. Rama Prasad, Deputy Adviser(PHE), Central Public Health & Environmental Engineering Organisation, Ministry of Urban Development, New Delhi	-	Member
18.	Shri B.B. Uppal, Assistant Adviser(PHE) Central Public Health & Environmental Engineering Organisation, Ministry of Urban Development, New Delhi	-	Member
19.	Dr. S.R. Shukla Deputy Adviser(PHE), Central Public Health & Environmental Engineering Organisation, Ministry of Urban Development, New Delhi	-	Member Secretary

The Committee held 11 meetings under the Chairmanship of Shri V. Venugopalan between April 1986 and August, 1989, and has drawn freely from all available literature, in finalising the revision of the Manual. The Committee wishes to thank the Union Ministry of Urban Development for rendering all the help needed for successfully revising and updating the Manual. The Committee also wishes to thank National Environmental Engineering Research Institute, Nagpur and the Maharashtra Water Supply & Sewerage Board for the arrangements made for the meetings of the Committee outside Delhi.

The Committee wishes to place on record their deep sense of appreciation for the unsparing and diligent efforts of Shri M.R. Parthasarthy and Dr. S.R. Shukla who ensured that the meetings were held regularly to enable the Committee to complete its work in spite of their heavy normal duties. The Committee also places on record its appreciation of the services rendered by the various officers and staff of Central Public Health and Environmental Engineering Organisation and Public Health Engineering Section of the Ministry without whose cooperation and active participation the enormous task assigned to the Committee could not have been accomplished.

A Sub-Committee for editing the draft Manual was constituted comprising of the Members Dr. I.C. Agarwal, Shri M.R. Parthasarathy and Dr. D.M. Mohan. The Sub Committee finalised the editing of the draft Manual in four sittings between September and December, 1989. The Committee also wishes to thank the Members of the Editing Sub Committee for the devoted and sincere work without which the final draft of the Manual would not have been completed.

The 3rd edition of the Manual was brought out by the CPHEEO in March, 1991 for the benefit of Public Health Engineers, Consultants, Water Supply Departments/Boards, Local Bodies, Educational Institutions. However, subsequent to the publication of the said Manual, a few suggestions, observations and comments have been received from various product manufacturers, field engineers, consultants, etc. for revising and updating certain aspects, such as, water quality guidelines, per-capita water supply norms, water conservation measures, metering, availability of various pipes, selection of pipes under different field conditions, etc.

Accordingly, a two member Expert Committee, comprising Dr. I.C. Agarwal, Director, Bundelkhand Institute of Engineering and Technology, Jhansi, Uttar Pradesh and Dr. D.M. Mohan, Retired Director (Technical), Hyderabad Metropolitan Water Supply & Sewerage Board, was set up by the Ministry in February, 1998 with definite terms of reference to update certain chapters by including latest developments in the field of water supply, treatment and distribution.

Both the Expert Committee Members had completed their job to the satisfaction of the Ministry. The draft Manual (revised portion) was discussed thoroughly in the Chief Engineers' Conference held in November, 1998 at Chandigarh. The Expert Committee had reviewed the suggestions made by the Chief Engineers' Conference and modified the draft Manual accordingly so as to make it comprehensive and more useful to the user agencies dealing with water supply sector. The contribution made by Shri V.B. Rama Prasad, Shri R. Sethuraman, Shri B.B. Uppal, Deputy Advisers, Shri M. Sankaranarayanan, Shri M. Dhinadhayalan, Shri. N.N. Hotchandani, Assistant Advisers and Shri Sukanta Kar, Scientific Officer, for successful completion of the task is duly appreciated. The services extended by Dr. I.C. Agarwal, Director, Dr. Rajiv Srivastava, Shri Pankaj Rana and other staff of Bundelkhand Institute of Engineering & Technology, Jhansi, for computerising the said manual is duly acknowledged.

Last but not the least, the whole-hearted cooperation of the Ministry in completing this exercise is greatly acknowledged.

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C.P.H.E.E.O.
Ministry of Urban Development
Government of India

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CHAPTER 1
INTRODUCTION

Water constitutes one of the important physical environments of man and has a direct bearing on his health. There is no gainsaying that contamination of water leads to health hazards. Water is precious to man and therefore WHO refers to "control of Water Supplies to ensure that they are pure and wholesome as one of the primary objectives of environmental sanitation". Water may be polluted by physical, chemical and bacterial agents. Therefore, protected water supply is a sine qua non of public health of a community.

The population of India is likely to be around a thousand million by the end of the century. The urban population would be around four hundred million by that time. This means a very large demand on the civic amenities including water supply for domestic purposes and in addition more water would be needed for purposes such as irrigation, industry, etc., which have to keep pace with the increasing demands of rising population. Therefore, identification of sources of water supply, their conservation and optimal utilization is of utmost importance. Even the present scale of water supply to urban and rural population is grossly inadequate and not all communities are provided with safe water supply, let alone piped water system; hardly any metropolitan city has a continuous water supply; and very few cities could boast of providing adequate water supply to meet their growing demands at adequate pressure.

Many facets are involved in tackling the problem of providing protected water supply to all communities at the minimum cost and in the shortest possible time. Emphasis has to be laid on both the aspects of the system namely, planning and management technical and financial. At present a number of decisions, both at policy and technical levels, are being based on empirical considerations and divergent practices are in vogue in the country in so far as designing the system itself is concerned. The Manual would have to attempt at the unification of these practices and help to inculcate rationale to policy and managerial decisions apart from giving guidance to the public health engineers in achieving the target of providing safe water to all communities economically and expeditiously.

Obviously, it would be in the interest of public health engineers to have a standard manual in public health engineering and a code of practice which could serve as a guide in their day to day practice. This Manual would discuss the basic principles such as planning, identification of source of supply, development and transmission, water treatment, distribution system, testing and other related administrative aspects and also explain in detail the proper approach to each problem.

This Revised Manual has taken into account the recent technical advances and trends in the development of protected water supply systems, some of the major changes and additions as highlighted in the following areas:

- Ground water potential and its development in hard rock regions;
- Well development, failure of wells and remedial measures;
- Ground water abstraction through radial wells;
- Measurement of flow;
- Minimum requirements for domestic, non-domestic, institutional, fire fighting and industrial needs;
- Minimum residual pressure and quality standards including virological aspects,
- Concept of unit operations;
- Chemical handling and feeding;
- Recent concepts of coagulation and flocculation;
- Advances in filtration;
- Operation and maintenance problems in various unit operations involved in water supply, from source development to the actual supply;
- Pumping stations and equipment;
- Hydraulic network analysis, direct design of networks and computer programming;
- Preventive maintenance including detection and prevention of wastage;
- Protection against pollution and freezing;
- Corrosion and its prevention;
- Water hammer problems;
- House service connections;
- Optimal design of water treatment systems;
- Instrumentation & controls in water treatment plants;
- Financing and management;
- Legal aspects;
- Laboratory tests and procedures with special reference to the classification of the water works laboratories.

In keeping with the changeover to the metric system, the various units of measurements, operational parameters and design criteria have all been confined to the metric system only, with deliberate omission of equivalents in the British System generally furnished alongside. This has been felt necessary, since there is, still an apathy on the part of the field engineer to break away from the conventional, in which he feels at home, since tradition dies hard.

However, a table of conversion factors has been appended to facilitate the verification of any of the parameters by conversion to the units he is accustomed to.

This Manual also contains a set of appendices furnishing useful information helpful in solving day to day problems which the practicing engineer is likely to encounter. Model problems have been worked out which have a relevance in design. Useful references of the Bureau of Indian Standards, are also listed in a separate appendix. Charts for Hazen Williams formula as well as Manning's formula, which are frequently used, are presented in the metric system in separate appendices.

A companion Manual on Sewerage and Sewage Treatment has been brought out by the erstwhile Union Ministry of Works and Housing (Central Public Health and Environmental Engineering Organization) which has been revised and published in 1993. The recommendations of this Manual and the provisions of the Water (Prevention and Control of Pollution) Act, 1974 should be followed wherever applicable.

CHAPTER 2
PLANNING

2.1 OBJECTIVE

The objective of a public protected water supply system is to supply safe and clean water in adequate quantity, conveniently and as economically as possible. The planning may be required at national level for the country as a whole, or for the state or region or community. Though the responsibility of the various organizations incharge of planning of water supply systems in each of these cases is different, they still have to function within the priorities fixed by the national and state governments, taking into consideration, the areas to be provided with water supply and the most economical way of doing it, keeping in view the overall requirements of the entire region.

The water supply projects formulated by the various state authorities and local bodies at present do not contain all the essential elements for appraisal and when projects are assessed for their cost benefit ratio and for institutional or other funding, they are not amenable for comparative study and appraisal. Also, different guidelines and norms are adopted by the central and state agencies; for example, assumptions regarding per capita water supply, design period, population forecast, measurement of flow, water treatment, specifications of materials, etc. Therefore, there is a need to specify appropriate standards, planning, and design criteria to avoid empirical approach.

2.2 BASIC DESIGN CONSIDERATIONS

Engineering decisions are required to specify the area and population to be served, the design period, the per capita rate of water supply, other water needs in the area, the nature and location of facilities to be provided, the utilization of centralized or multiple points of treatment facilities and points of water supply intake and waste water disposal. Projects have to be identified and prepared in adequate detail in order to enable timely and proper implementation. Optimization may call for planning for a number of phases relating to plant capacity and the degree of treatment to be provided by determining the capacities for several units, working out capital cost required, interest charges, period of repayment of loan, water tax and water rate. Uncertainties in such studies are many, such as the difficulties in anticipating new technology and changes in the investment pattern, the latter being characterized by increasing financing costs.

2.2.1 WATER QUALITY AND QUANTITY

The waters to be handled may vary both in quantity and quality and in the degree of treatment required, seasonally, monthly, daily and sometimes even hourly. The public health engineer may use his ingenuity to mitigate the variations in quantity by provision of storage, which may be drawn upon during peak demand. Variations in quality can be managed by provision for the introduction of suitable process adjustments in the water treatment plant.

2.2.1.1 Water Conservation

Rising demand for water in urban communities due to population increase, commercial and industrial development and improvement in living standards is putting enormous stress on easily and economically exploitable water resources. Not only the quantity of extractable fresh water resources is being depleted but also the quality is deteriorating. Ground waters may be chemically contaminated, for example, due to excessive fluorides, total dissolved solids, iron and manganese and even arsenic in some cases. Due to over abstraction of ground waters for agricultural and industrial uses, this problem is further aggravated. Surface water bodies, being indiscriminately used for discharge of municipal and industrial wastewaters, may have quality parameters which may require application of advanced water treatment processes. It has therefore, become essential to initiate measures for effective and integrated approach for water conservation.

Water conservation may be possible through (1) optimal use of available water resources; (2) prevention and control of wastage of water and (3) effective demand management.

2.2.1.2 Increasing The Water Availability And Supply& Demand Management

The measures required to increase the water availability involve augmentation of water resources by storing rainwater on the surface or below the surface. Surface storage is usually contemplated either in natural ponds, reservoirs and lakes or artificially created depressions, ponds, impounding reservoirs or tanks. Subsurface storage of water is effected by constructing subsurface dykes, artificial recharge wells, etc. For storing subsurface water in rocky areas, several techniques have been developed indigenously like Jacket Well Technique, Bore Blast Techniques, Fracture Seal Cementation etc. These techniques have been deployed to improve porosity, storage volume as well as interconnectivity between fractures/fissures and other types of pores. Artificial recharge of ground water may be contemplated in some areas.

Water supply management aims at improving the supply by minimizing losses and wastage and unaccounted for water (UFW) in the transmission mains and distribution system (Reference may also be made to section 10.10.) The unaccounted-for-water constitutes a significantly higher fraction of total water supplied in poorly managed water transmission and distribution systems. Measures like detection, control and prevention of leakage, metering of water supply, installation of properly designed waste-not-taps and prompt action to repair and maintain distribution system components should be adopted.

Water demand management involves measures which aim at reducing water demand by optimal utilization of water supplies for all essential and desirable needs. It focuses on identification of all practices and uses of water in excess of functional requirement. Use of plumbing fixtures, such as low volume and dual flushing cisterns in place of conventional 12.5 litre capacity cisterns which conserve water may be encouraged. Practices like reuse and recycling of treated wastewater may be promoted for which references may be made to chapter 19 & 20 of the Manual on Sewerage and Sewage Treatment.

2.2.2 Plant Siting

Though the distribution lay out and the sources of supply and their development methods are important in siting the different units for optimal and economical utilization, factors like topography, soil conditions, and physical hazards should also be taken into consideration. Hillside construction may have an advantage in accommodating the headloss in the plant without excessive excavation and may permit ground level entrance to several floors in service buildings. Wet sites must be dewatered and structures may have to be designed to overcome the hydrostatic uplift. On soils having low bearing capacities, structures may need to be placed on piles or rafts. Rocky sites may require costly excavation.

Flooding is a common hazard for the treatment plants and pumping stations located near rivers. The highest flood level observed at the site selected should be taken into account and the treatment plant and pumping station structures may be built above the high water mark, or may be surrounded by dykes, to prevent damage due to flooding. Contact should be maintained with the Irrigation Department for the use of the flood warning system.

2.2.3 Mechanization

Mechanization, instrumentation, and automation are becoming more and more common in water works and this should also be taken into account in planning the system, subject to local availability and maintenance facilities.

Mechanization replaces and serves the functions that cannot be performed efficiently by manual operations such as the removal of the sludge from sedimentation tanks. Instrumentation involves installation of various kinds of devices and gauges for monitoring and recording of plant flows and performance. Automation combines instrumentation and mechanization to effect head loss control for back washing and specific control for turbidity, colour, dissolved oxygen, pH, chlorine residual, conductivity and float controls for pumping.

2.2.4 Service Building

Considerable attention is to be given to the service building required at treatment works and pumping stations such as houses, offices and laboratories, washing room and store rooms, chemical house, pump house etc.. In mild climates operating structures need to be protected against rain and sun while in adverse climates complete protection against such weather is advisable.

2.2.5 Other Utilities

Provision needs to be made for facilities such as electricity, water supply and drainage, roadways, parking areas, walkways, fencing, telephone facilities and other welfare services such as housing for operation and maintenance personnel.

2.2.6 Design Period

Water Supply projects may be designed normally to meet the requirements over a thirty-year period after their completion. The time lag between design and completion of the project should also be taken into account which should not exceed two years to five years depending on the size of the project. The thirty year period may however be modified in regard to certain components of the project depending on their useful life or the facility for carrying out extensions when required and rate of interest so that expenditure far ahead of utility is avoided. Necessary land for future expansion/duplication of components should be acquired in the beginning itself. Where expensive tunnels and large aqueducts are involved entailing large capital outlay for duplication, they may be designed for ultimate project requirements. Where failure such as collapse of steel pipes under vacuum put the pipe line out of commission for a long time or the pipe location presents special hazards such as floods, ice, and mining etc., duplicate lines may be necessary.

Project components may be designed to meet the requirements of the following design periods:

Sl. No.	Items	Design period in years
1.	Storage by dams	50
2.	Infiltration works	30
3.	Pumping:	
	i. Pump house (civil works)	30
	ii. Electric motors and pumps	15
4.	Water treatment units	15
5.	Pipe connection to several treatment units and other small appurtenances	30
6.	Raw water and clear water conveying mains	30
7.	Clear water reservoirs at the head works, balancing tanks and service reservoirs (overhead or ground level)	15
8.	Distribution system	30

2.2.7 POPULATION FORECAST

2.2.7.1 General Considerations

The design population will have to be estimated with due regard to all the factors governing the future growth and development of the project area in the industrial, commercial, educational, social and administrative spheres. Special factors causing sudden emigration or influx of population should also be foreseen to the extent possible.

A judgement based on these factors would help in selecting the most suitable method of deriving the probable trend of the population growth in the area or areas of the project from out of the following mathematical methods, graphically interpreted where necessary.

a) Demographic Method of Population Projection

Population change can occur only in three ways (i) by births (population gain) (ii) by deaths (population loss) or (iii) migration (population loss or gain depending on whether movement out or movement in occurs in excess). Annexation of an area may be considered as a special form of migration. Population forecasts are frequently obtained by preparing and summing up of separate but related projections of natural increases and of net migration and is expressed as below.

The net effect of births and deaths on population is termed natural increase (natural decrease, if deaths exceed births).

Migration also affects the number of births and deaths in an area and so, projections of net migration are prepared before projections for natural increase.

This method thus takes into account the prevailing and anticipated birth rates and death rates of the region or city for the period under consideration. An estimate is also made of the emigration from and immigration to the city, growth of city area wise, and the net increase of population is calculated accordingly considering all these factors, by arithmetical balancing.

b) Arithmetical Increase Method

This method is generally applicable to large and old cities. In this method the average increase of population per decade is calculated from the past records and added to the present population to find out population in the next decade. This method gives a low value and is suitable for well-settled and established communities.

c) Incremental Increase Method

In this method the increment in arithmetical increase is determined from the past decades and the average of that increment is added to the average increase. This method increases the figures obtained by the arithmetical increase method.

d) Geometrical Increase Method

In this method percentage increase is assumed to be the rate of growth and the average of the percentage increases is used to find out future increment in population. This method gives much higher value and mostly applicable for growing towns and cities having vast scope for expansion.

e) Decreasing Rate Of Growth Method

In this method it is assumed that rate of percentage increase decreases and the average decrease in the rate of growth is calculated. Then the percentage increase is modified by deducting the decrease in rate of growth. This method is applicable only in such cases where the rate of growth of population shows a downward trend.

f) Graphical Method

In this approach there are two methods. In one, only the city in question is considered and in the second, other similar cities are also taken into account.

(i) Graphical Method Based On Single City

In this method the population curve of the city (i.e. the Population vs. Past Decades) is smoothly extended for getting future value. This extension has to be done carefully and it requires vast experience and good judgement. The line of best fit may be obtained by the method of least squares.

(ii) Graphical Method Based On Cities With Similar Growth Pattern

In this method the city in question is compared with other cities which have already undergone the same phases of development which the city in question is likely to undergo and based on this comparison, a graph between population and decades is plotted.

g) Logistic Method

The 'S' shaped logistic curve for any city gives complete trend of growth of the city right from beginning to saturation limit of population of the city.

h) Method of Density

In this approach, trend in rate of density increase of population for each sector of a city is found out and population forecast is done for each sector based on above approach. Addition of sector-wise population gives the population of the city.

2.2.7.2 Final Forecast

While the forecast of the prospective population of a projected area at any given time during the period of design can be derived by any one of the foregoing methods appropriate to each case, the density and distribution of such population within the several areas, zones or districts will again have to be made with a discerning judgement on the relative probabilities of expansion within each zone or district, according to its nature of development and based on existing and contemplated town planning regulations.

Wherever population growth forecast or master plans prepared by town planning or other appropriate authorities are available, the decision regarding the design population should take into account their figures. Worked out examples for estimation of the future population by some of the methods are given in Appendix 2.1.

2.2.8 PER CAPITA SUPPLY

2.2.8.1 Basic Needs

Piped water supplies for communities should provide adequately for the following as applicable:

(a) Domestic needs such as drinking, cooking, bathing, washing, flushing of toilets, gardening and individual air conditioning

(b) Institutional needs

(c) Public purposes such as street washing or street watering, flushing of sewers, watering of public parks

(d) Industrial and commercial uses including central air conditioning

(e) Fire fighting

(f) Requirement for livestock; and

(g) Minimum permissible UFW(Ref. Table. 2.1)

2.2.8.2 Factors Affecting Consumption

a) Size of City

Larger the size, more the consumption.

b) Characteristics of Population and Standard of Living

In the high value residential area of the city or in a suburban community, per capita consumption is high. Slum areas of large cities have low per capita consumption. A person staying in an independent bungalow consumes more water compared to a person staying in a flat. Habit of person also affects consumption; the type of bath i.e. tub bath or otherwise and material used for ablution etc. also affect per capita consumption.

c) Industries and Commerce

The type and number of different industries also affect consumption. Commercial consumption is that of the retail and wholesale mercantile houses and office buildings.

d) Climatic Conditions

In hot weather, the consumption of water is more compared to that during cold weather.

e) Metering

The consumption of water when supply is metered is less compared to that when the water charges are on flat rate basis.

2.2.8.3 Recommendations

The Environmental Hygiene Committee suggested certain optimum service levels for communities based on population groups. In the Code of Basic Requirements of Water Supply, Drainage and Sanitation (IS: 1172-1983) as well as the National Building Code, a minimum of 135 lpcd has been recommended for all residences provided with full flushing system for excreta disposal. Though the Manual on Sewerage and Sewage Treatment recommends a supply of 150 lpcd wherever sewerage is existing/contemplated, with a view to conserve water, a minimum of 135 lpcd is now recommended.

It is well recognised that the minimum water requirements for domestic and other essential beneficial uses should be met through public water supply. Other needs for water including industries etc. may have to be supplemented from other systems depending upon the constraints imposed by the availability of capital finances and the proximity of water sources having adequate quantities of acceptable quality which can be economically utilised for public water supplies.

Based on the objectives of full coverage of urban communities with easy access to potable drinking water in quantities recommended to meet the domestic and other essential non-domestic needs, the following recommendations are made:

a) Domestic and non-domestic needs

The recommended values for domestic and non-domestic purposes are given in Table 2.1

TABLE 2.1

RECOMMENDED PER CAPITA WATER SUPPLY LEVELS FOR DESIGNING SCHEMES

Sl. No.	Classification of towns/cities	Recommended Maximum Water Supply Levels (lpcd)
1.	Towns provided with piped water supply but without sewerage system	70
2.	Cities provided with piped water supply where sewerage system is existing/contemplated	135
3.	Metropolitan and Mega cities provided with piped water supply where sewerage system is existing/ contemplated	150

Note:

(i) In urban areas, where water is provided through public standposts, 40 lpcd should be considered;

(ii) Figures exclude "Unaccounted for Water(UFW)" which should be limited to 15%

(iii) Figures include requirements of water for commercial, institutional and minor industries. However, the bulk supply to such establishments should be assessed separately with proper justification.

b) Institutional Needs

The water requirements for institutions should be provided in addition to the provisions indicated in (a) above, where required, if they are of considerable magnitude and not covered in the provisions already made. The individual requirements would be as follows:

Sl.No.	Institutions	Litres per head per day
1.	Hospital (including laundry)	
	(a) No. of beds exceeding 100	450 (per bed)
	(b) No. of beds not exceeding 100	340 (per bed)
2.	Hotels	180(per bed)
3.	Hostels	135
4.	Nurses' homes and medical quarters	135
5.	Boarding schools / colleges	135
6.	Restaurants	70(per seat)
7.	Air ports and sea ports	70

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Sl.No.	Institutions	Litres per head per day
8.	Junction Stations and intermediate stations where mail or express stoppage (both railways and bus stations) is provided	70
9.	Terminal stations	45
10.	Intermediate stations (excluding mail and express stops)	45 (could be reduced to 25 where bathing facilities are not provided)
11.	Day schools / colleges	45
12.	Offices	45
13.	Factories	45 (could be reduced to 30 where no bathrooms are provided)
14.	Cinema, concert halls and theatre	15

c) Fire Fighting Demand

It is usual to provide for fire fighting demand as a coincident draft on the distribution system along with the normal supply to the consumers as assumed. A provision in kilolitres per day based on the formula of $100\sqrt{p}$ where, p = population in thousands may be adopted for communities larger than 50,000. It is desirable that one third of the fire fighting requirements form part of the service storage. The balance requirement may be distributed in several static tanks at strategic points. These static tanks may be filled from the nearby ponds, streams or canals by water tankers wherever feasible. The high rise buildings should be provided with adequate fire storage from the protected water supply distribution as indicated in 10.3.2.

d) Industrial Needs

While the per capita rates of supply recommended will ordinarily include the requirement of small industries (other than factories) distributed within a town, separate provisions will have to be included for meeting the demands likely to be made by specific industries within the urban areas. The forecast of this demand will be based on the nature and magnitude of each such industry and the quantity of water required per unit of production. The potential for industrial expansion should be carefully investigated, so that the availability of adequate water supply may attract such industries and add to the economic prosperity of the community. As can be seen from the tabulation, the quantities of water used by industry vary widely. They are also affected by many factors such as cost and availability of water, waste disposal problems, management and the types of processes involved. Individual studies of the water requirement of a specific industry should, therefore, be made for each location, the values given below serving only as guidelines. In the context of reuse of water in several industries, the requirement of fresh water is getting reduced considerably.

Industry	Unit of production	Water requirement in Kilolitres per unit
Automobile	Vehicle	40
Distillery	(Kilolitre Alcohol)	122-170
Fertilizer	Tonne	80-200
Leather	100 Kg (tanned)	4
Paper	Tonne	200-400
Special quality paper	Tonne	400-1000
Straw board	Tonne	75-100
Petroleum Refinery	Tonne(crude)	1-2
Steel	Tonne	200-250
Sugar	Tonne (Cane crushed)	1-2
Textile	100 Kg (goods)	8-14

e) Pressure Requirements

Piped water supplies should be designed on continuous 24 hours basis to distribute water to consumers at adequate pressure at all points. Intermittent supplies are neither desirable from the public health point of view nor economical. For towns where one-storeyed buildings are common and for supply to the ground level storage tanks in multi-storeyed buildings, the minimum residual pressure at ferrule point should be 7m for direct supply. Where two-storeyed buildings are common, it may be 12m and where three-storeyed buildings are prevalent 17 m or as stipulated by local byelaws. The pressure required for fire fighting would have to be boosted by the fire engines.

2.2.9 QUALITY STANDARDS

The objective of Water Works Management is to ensure that the water supplied is free from pathogenic organisms, clear, palatable and free from undesirable taste and odour, of reasonable temperature, neither corrosive nor scale forming and free from minerals which could produce undesirable physiological effects. The establishment of minimum standards of quality for public water supply is of fundamental importance in achieving this objective. Standards of quality form the yardstick within which the quality control of any public water supply has to be assessed.

Sanitary inspections are intended to provide a range of information and to locate potential problems. The inspections allow for an overall appraisal of many factors associated with a water supply system, including the water works and the distribution system. Moreover such an appraisal may later be verified and confirmed by microbiological analysis, which will indicate the severity of the problem. Sanitary inspections thus provide a direct method of pinpointing possible problems and sources of contamination. They are also important in the prevention and control of potentially hazardous conditions, including epidemics of water borne diseases. The data obtained may identify failures, anomalies, operator errors and any deviations from normal that may affect the production and distribution of safe drinking water. When the inspections are properly carried out at appropriate regular intervals and where the inspector has the knowledge necessary to detect problems and suggest technical solutions, the production of good quality water is ensured.

The evolution of standards for the quality control of public water supplies has to take into account the limitations imposed by local factors in the several regions of the country. The Environmental Hygiene Committee (1949) recommended that the objective of a public water supply should be to supply water "that is absolutely free from risks of transmitting diseases, is pleasing to the senses and is suitable for culinary and laundering purposes" and added that "freedom from risks is comparatively more important than physical appearance or hardness" and that safety is an obligatory standard and physical and chemical qualities are optional within a range. These observations are relevant in the development of a country-wide programs of protected water supply systems for communities big and small, making use of the available water resources in the different regions, with a wide variation in their physical, chemical and aesthetic qualities, that can be achieved by communities in due course within the limits of their financial resources. The immediate need is for minimum standards, consistent with the safety of public water supplies. Considering the standards prescribed in the earlier Manual and further development in the international standardization and the conditions in the country, the following guidelines are recommended.

a) Physical And Chemical Quality Of Drinking Water

The physical and chemical quality of drinking water should be in accordance with the recommended guidelines presented in Table 2.2.

TABLE 2.2

RECOMMENDED GUIDELINES FOR PHYSICAL AND CHEMICAL PARAMETERS

Sl. No.	Characteristics	*Acceptable	**Cause for Rejection
1.	Turbidity (NTU)	1	10
2.	Colour (Units on Platinum Cobalt scale)	5	25
3.	Taste and Odour	Unobjectionable	Objectionable
4.	pH	7. 0 to 8.5	<6.5 or >9.2
5.	Total dissolved solids (mg/l)	500	2000
6.	Total hardness (as $CaCO_3$) (mg/l)	200	600
7.	Chlorides (as Cl) (mg/l)	200	1000
8.	Sulphates (as SO_4) (mg/l)	200	400
9.	Fluorides (as F) (mg/l)	1.0	1.5
10.	Nitrates (as NO_3) (mg/l)	45	45
11.	Calcium (as Ca) (mg/l)	75	200
12.	Magnesium (as Mg) (mg/l)	≤ 30	150

Sl. No.	Characteristics	*Acceptable	**Cause for Rejection
If there are 250 mg/l of sulphates, Mg content can be increased to a maximum of 125 mg/l with the reduction of sulphates at the rate of 1 unit per every 2.5 units of sulphates			
13.	Iron (as Fe) (mg/l)	0.1	1.0
14.	Manganese (as Mn) (mg/l)	0.05	0.5
15.	Copper (as Cu) (mg/l)	0.05	1.5
16.	Aluminium (as Al) (mg/l)	0.03	0.2
17.	Alkalinity (mg/l)	200	600
18.	Residual Chlorine (mg/l)	0.2	>1.0
19.	Zinc (as Zn) (mg/l)	5.0	15.0
20.	Phenolic compounds (as Phenol) (mg/l)	0.001	0.002
21.	Anionic detergents (mg/l) (as MBAS)	0.2	1.0
22.	Mineral Oil (mg/l)	0.01	0.03
	TOXIC MATERIALS		
23.	Arsenic (as As) (mg/l)	0.01	0.05
24.	Cadmium (as Cd) (mg/l)	0.01	0.01
25.	Chromium (as hexavalent Cr) (mg/l)	0.05	0.05
26.	Cyanides (as CN) (mg/l)	0.05	0.05
27.	Lead (as Pb) (mg/l)	0.05	0.05
28.	Selenium (as Se) (mg/l)	0.01	0.01
29.	Mercury (total as Hg) (mg/l)	0.001	0.001
30.	Polynuclear aromatic hydrocarbons (PAH) (μg/l)	0. 2	0. 2
31.	Pesticides (total, mg/l)	Absent	Refer to WHO guidelines for drinking water quality Vol 1. – 1993
	RADIO ACTIVITY+		
32.	Gross Alpha activity (Bq/l)	0.1	0.1
33.	Gross Beta activity (Bq/l)	1.0	1.0

NOTES

* The figures indicated under the column 'Acceptable' are the limits upto which water is generally acceptable to the consumers.

** Figures in excess of those mentioned under 'Acceptable' render the water not acceptable, but still may be tolerated in the absence of an alternative and better source but upto the limits indicated under column "Cause for Rejection" above which the sources will have to be rejected.

\+ It is possible that some mine and spring waters may exceed these radio activity limits and in such cases it is necessary to analyze the individual radio-nuclides in order to assess the acceptability or otherwise for public consumption.

b) Bacteriological Guidelines

The recommended guidelines for bacteriological quality are given in Table 2.3.

TABLE 2.3
BACTERIOLOGICAL QUALITY OF DRINKING WATER[a]

Organisms	Guideline value
All water intended for drinking	
E.coli or thermotolerant coliform bacteria[b,c]	Must not be detectable in any 100-ml sample
Treated water entering the distribution system	
E.coli or thermotolerant coliform bacteria[b]	Must not be detectable in any 100-ml sample
Total coliform bacteria	Must not be detectable in any 100-ml sample
Treated water in the distribution system	
E.coli or thermotolerant coliform bacteria[b]	Must not be detectable in any 100-ml sample
Total coliform bacteria	Must not be detectable in any 100-ml sample. In case of large supplies, where sufficient samples are examined, must not be present in 95% of samples taken throughout any 12 month period.

Source : WHO guidelines for Drinking Water Quality Vol.1 – 1993.

[a] Immediate investigative action must be taken if either *E.coli* or total coliform bacteria are detected. The minimum action in the case of total coliform bacteria is repeat sampling; if these bacteria are detected in the repeat sample, the cause must be determined by immediate further investigation.

[b] Although *E.coli* is the more precise indicator of faecal pollution, the count of thermotolerant coliform bacteria is an acceptable alternative. If necessary, proper confirmatory test must be carried out. Total coliform bacteria are not acceptable indicators of the sanitary quality of rural water supplies, particularly in tropical areas where many bacteria of no sanitary significance occur in almost all untreated supplies.

[c] It is recognized that, in the great majority of rural water supplies in developing countries, faecal contamination is widespread. Under these conditions, the national surveillance agency should set medium term targets for progressive improvement of water supplies, as recommended in volume 3 of W.H.O. *guidelines for drinking-water quality1993.*

c) Virological Quality

Drinking water must essentially be free of human enteroviruses to ensure negligible risk of transmitting viral infection. Any drinking-water supply subject to faecal contamination presents a risk of a viral disease to consumers. Two approaches can be used to ensure that the risk of viral infection is kept to a minimum: providing drinking water from a source verified free of faecal contamination, or adequately treating faecally contaminated water to reduce enteroviruses to a negligible level.

Virological studies have shown that drinking water treatment can considerably reduce the levels of viruses but may not eliminate them completely from very large volumes of water. Virological, epidemiological, and risk analysis are providing important information, although it is still insufficient for deriving quantitative and direct virological criteria. Such criteria can not be recommended for routine use because of the cost, complexity, and lengthy nature of virological analysis, and the fact that they can-not detect the most relevant viruses.

The guideline criteria shown in Table 2.4 are based upon the likely viral content of source waters and the degree of treatment necessary to ensure that even very large volumes of drinking water have negligible risk of containing viruses.

Ground water obtained from a protected source and documented to be free from faecal contamination from its zone of influence, the well, pumps, and delivery system can be assumed to be virus-free. However, when such water is distributed, it is desirable that it is disinfected, and that a residual level of disinfectant is maintained in the distribution system to guard against contamination.

TABLE 2.4

RECOMMENDED TREATMENT FOR DIFFERENT WATER SOURCES TO PRODUCE WATER WITH NEGLIGIBLE VIRUS RISK[a]

Type of Source	Recommended Treatment
Ground water	
Protected, deep wells; essentially free of faecal contamination	Disinfection[b]
Unprotected, shallow wells; faecally contaminated	Filtration and disinfection
Surface water	
Protected, impounded upland water; essentially free of faecal contamination	Disinfection
Unprotected impounded water or upland river; faecal contamination	Filtration and disinfection

Type of Source	Recommended Treatment
Unprotected lowland rivers; faecal contamination	Pre-disinfection or storage, filtration, disinfection
Unprotected watershed; heavy faecal contamination	Pre-disinfection or storage, filtration, additional treatment and disinfection
Unprotected watershed; gross faecal contamination	Not recommended for drinking water supply

[a] For all sources, the median value of turbidity before terminal disinfection must not exceed 1 nephelometric turbidity unit(NTU) and must not exceed 5 NTU in single sample.

Terminal disinfection must produce a residual concentration of free chlorine of ≥ 0.5 mg/litre after atleast 30 minutes of contact in water at pH < 8.0, or must be shown to be an equivalent disinfection process in terms of the degree of enterovirus inactivation($>99.99\%$).

Filtration must be either slow sand filtration or rapid filtration (sand, dual, or mixed media) preceded by adequate coagulation-flocculation (with sedimentation or floatation). Diatomaceous earth filtration or filtration process demonstrated to be equivalent for virus reduction can also be used. The degree of virus reduction must be $>90\%$.

Additional treatment may consist of slow sand filtration, ozonation with granular activated carbon adsorption, or any other process demonstrated to achieve $> 99\%$ enterovirus reduction.

[b] Disinfection should be used if monitoring has shown the presence of *E. coli* or thermo-tolerant coliform bacteria.

SOURCE : W.H.O. guidelines for Drinking Water Quality - 1993.

CHAPTER 3
PROJECT REPORT

3.1 GENERAL

All projects have to follow distinct stages between the period they are conceived and completed. These various stages are:

- Pre-Investment Planning
  - Identification of a project
  - Preparation of project
- Appraisal and sanction
- Construction of facilities and carrying out support activities
- Operation and Maintenance
- Monitoring and feed back

3.1.1 PROJECT REPORTS

Project reports deal with all the aspects of pre-investment planning and establishes the need as well as the feasibility of projects technically, financially, socially, culturally, environmentally, legally and institutionally. For big projects economical feasibility may also have to be examined. Project reports should be prepared in three stages viz. (i) identification report; (ii) pre-feasibility report and (iii) feasibility report. Projects for small towns or those forming parts of a programme may not require preparation of feasibility reports. Detailed engineering, and preparation of technical specifications and tender documents are not necessary for taking investment decisions, since these activities can be carried out during the implementation phase of projects. For small projects, however, it may be convenient to include detailed engineering in the project report, if standard design and drawings can be adopted.

Since project preparation is quite expensive and time consuming, all projects should normally proceed through three stages; and at the end of each stage a decision should be taken whether to proceed to the next planning stage, and commit the necessary manpower and financial resources for the next stage. Report at the end of each stage should include a time table and cost estimate for undertaking the next stage activity, and a realistic schedule for all future stages of project development, taking into consideration time required for review and approval of the report, providing funding for the next stage, mobilising personnel or fixing agency (for the next stage of project preparation), data gathering, physical surveys, site investigations etc.

The basic design of a project is influenced by the authorities/organizations who are involved in approving, implementing and operating and maintaining the project. Therefore, the institutional arrangements through which a project will be brought into operation, must be considered at the project preparation stage. Similarly responsibility for project preparation may change at various stages. Arrangements in this respect should be finalized for each stage of project preparation. Sometimes more than one organisation may have a role to play in the various stages of preparation of a project. It is, therefore, necessary to identify a single entity to be responsible for overall management and coordination of each stage of project preparation. It is desirable that the implementing authority and those responsible for operation of a project are consulted at the project preparation stage.

3.2 IDENTIFICATION REPORT

Identification report is basically a "desk study", to be carried out relying primarily on the existing information. It can be prepared reasonably quickly by those who are familiar with the project area and needs of project components. This report is essentially meant for establishing the need for a project, indicating likely alternatives, which would meet the requirements. It also provides an idea of the magnitude of cost estimates of a project to facilitate bringing the project in the planning and budgetary cycle, and makes out a case for obtaining sanction to incur expenditure for carrying out the next stages of project preparation. The report should be brief and include the following information:

- Identification of project area and its physical environment
- Commercial, industrial, educational, cultural and religious importance and activities in and around the project area (also point out special activities or establishment like defence or others of national importance)
- Existing population, its physical distribution and socio-economic analysis
- Present water supply arrangements and quality of service in the project area, pointing out deficiencies, if any, in quality, quantity and delivery system
- Population projection for the planning period, according to existing and future land use plans, or master plans, if any
- Water requirements during planning period for domestic, industrial, commercial and any other uses
- Establish the need for taking up a project in the light of existing and future deficiencies in water supply services, pointing out adverse impacts of non-implementation of the project, on a time scale
- Bring out, how the project would fit in with the national/regional/sectoral strategies and with the general overall development in the project area
- Identify a strategic plan for long term development of water supply services in the project area, in the context of existing regional development plans, water resources studies and such other reports, indicating phases of development

- State the objectives of the short term project under consideration, in terms of population to be served, other consumers, if any, service standard to be provided, and the impact of the project after completion; clearly indicate the design period
- Identify project components, with alternatives if any; both physical facilities and supporting activities
- Preliminary estimates of costs (componentwise) of construction of physical facilities and supporting activities, cost of operation and maintenance, identify source for financing capital works, and operation and maintenance, work out annual burden (debt servicing + operational expenditure)
- Indicate institutions responsible for project approval, financing, implementation, operation and maintenance (e.g. National Government, State Government, Zilla Parishad, Local Body, Water Supply Boards)
- Indicate organisation responsible for preparing the project (pre-feasibility report, feasibility report), cost estimates for preparing project report, and sources of funds to finance preparation of project reports
- Indicate time table for carrying out all future stages of the project, and the earliest date by which the project might be operational
- Indicate personnel strength required for implementation of the project, indicate if any particular/peculiar difficulties of policy or other nature are likely to be encountered for implementing the project and how these could be resolved
- Recommend actions to be taken to proceed further.

The following plans may be enclosed with the report:

(a) An index plan to a scale of 1 cm = 2 km showing the project area, existing works, proposed works, location of community / township or institution to be served.

(b) A schematic diagram showing the salient levels of project components.

3.3 PRE-FEASIBILITY REPORT

After clearance is received, on the basis of identification report from the concerned authority and/or owner of the project, and commitments are made to finance further studies, the work of preparation of pre-feasibility report should be undertaken by an appropriate agency, which may be a central planning and designing cell of a Water Supply Department/ Board, Local Body, or professional consultants working in the water supply-sanitation-environmental areas. In the latter case, terms of references for the study and its scope should be carefully set out. Pre-feasibility study may be a separate and discrete stage of project preparation or it may be the first stage of a comprehensive feasibility study. In either case it is necessary that it proceeds with taking up of a feasibility study because the pre-feasibility study is essentially carried out for screening and ranking of all project alternatives, and to select an appropriate, alternative for carrying out detailed feasibility study. The pre-feasibility study helps in selecting a short-term project which will fit in the long term strategy for improving services in the context of overall perspective plan for development of the project area.

3.3.1 CONTENTS

A pre-feasibility report can be taken to be a preliminary Project Report, the structure and component of which are as follows:

- Executive summary
- Introduction
- The project area and the need for a project
- Long term plan for water supply
- Proposed water supply project
- Conclusions and recommendations
- Tables, figures/maps and annexes.

3.3.1.1 Executive Summary

It is a good practice to provide an Executive Summary at the beginning of the report, giving its essential features, basic strategy, approach adopted in developing the study project, and the salient features of financial and administrative aspects.

3.3.1.2 Introduction

This section explains the origin and concept of the project, how it was prepared and the scope and status of the report. These sub-sections may be detailed as under :

(a) Project Genesis

- Describe how the idea of the project originated, agency responsible for promoting the project, list and explain previous studies and reports on the project, including the project identification report, and agencies which prepared them,
- Describe how the project fits in the regional development plan, long term sector plan, land use plan, public health care, and water resources development etc.

(b) How Was The Study Organised

- Explain how the study was carried out, agencies responsible for carrying out the various elements of work, and their role in preparing the study,
- Time table followed for the study.

(c) Scope And Status Of The Report

- How the pre-feasibility report fits in the overall process of project preparation
- Describe data limitation.
- List interim reports prepared during the study,

- Explain if the pre-feasibility report is intended to be used for obtaining approval for the proposed project.

3.3.1.3 The Project Area And The Need For The Project

This section establishes the need for the project. It should cover. the following :

(a) Project Area

- Give geographical description of the project area with reference to map / maps, describe special features such as topography, climate, culture, religion, migration, etc. Which may affect project design, implementation and operation
- Map showing administrative and political jurisdiction
- Describe, if any, ethnic, cultural or religious aspects of the communities which may have a bearing on the project proposal.

(b) Pollution Pattern

- Estimate population in the project area, indicating the source of data or the basis for the estimate
- Review previous population data, historic growth rates and causes
- Estimate future population growth with different methods and indicate the most probable growth rates and compare with past population growth trends
- Compare growth trends within the project area, with those for the region, state and the entire country
- Discuss factors likely to affect population growth rates
- Estimate probable densities of population in different parts of the project area at future intervals of time e.g. five, ten, and twenty years ahead
- Discuss patterns of seasonal migration, if any, within the area
- Indicate implication of the estimated growth pattern on housing and other local infrastructure.

(c) Economic and Social Conditions

- Describe present living conditions of the people of different socio-economic and ethnic groups
- Identify locations according to income levels or other indications of socio-economic studies
- Show on the project area map locationwise density of population; poverty groups and ethnic concentrations, and the present and future land uses (as per development plan)
- Information on housing conditions and relative proportions of owners and tenants

- Provide data on education, literacy and unemployment by age and sex
- Provide data and make projection on housing standards, and average household occupancy in various parts of the project area
- Describe public health status within the project area, with particular attention to diseases related to water and sanitary conditions; provide data on crude maternal and infant mortality rates, and life expectancy
- Discuss the status of health care programmes in the area, as well as other projects which have bearing on improvements in environmental sanitation.

(d) Sector Institutions

- Identify the institutions (Government, Semi-Government, Non-Government) which are involved in any of the stages of water supply and sanitation project development in the area, (planning, preparing projects, financing, implementation, operation and maintenance, and evaluation).
- Comment on roles, responsibilities and limitation (territorial or others) of all the identified institutions, in relation to water supply and sanitation (This may also be indicated on a diagram).

(e) Available Water Resources

- Summarise the quantity and quality of surface and ground water resources, actual and potential, in the project area and vicinity (give sources of information)
- Indicate studies carried out or being carried out concerning development of potential sources, and their findings
- Mention the existing patterns of water use by all sectors (irrigation, industrial energy, domestic etc.), comment on supply surplus or deficiency and possible conflicts over the use of water, at present and in future
- Comment on pollution problems, if any, which might affect available surface and ground water resources.
- Mention the role of agencies/authorities responsible for managing water resources, their allocation and quality control.

(f) Existing Water Supply Systems and Population served

- Describe each of the existing-water supply systems in the project area, indicating the details as under
- Source of water, quantity and quality available in various seasons, components of the system such as head works, transmission mains, pumping stations, treatment works, balancing/service reservoirs, distribution system, reliability of supply in all seasons
- Areas supplied, hours of supply, water pressures, operating problems, bulk meters, metered supplies, un-metered supplies, supply for commercial use, industrial use, and domestic use

- Private water supply services such as, wells, bores, water vendors etc
- Number of people served according to water supply systems of the following category:
  - Unprotected sources like shallow wells, rivers, lakes, ponds, etc.
  - Protected private sources like wells, bores, rain water storage tanks etc., Piped water system
- Number of house connections, number of stand pipes
- Consumers opinion about stand-pipe supply, (e.g. Distance, hours of supply, waiting time etc.)
- How many people obtain water from more than one source, note these sources, and how their waters are used, e.g., Drinking, bathing, washing etc. and reasons for their preferences
- Explain un-accounted for water, probable causes and trends and efforts made to reduce losses
- Comment on engineering and social problems of existing systems and possible measures to resolve these problems and the expected improvement

(g) Existing Sanitation Systems and Population served

Even if the proposed project may be for providing a single service i.e. water supply and not sanitation, the existing sanitation arrangements should be described, giving details of the existing sanitation and waste disposal systems in the project area, and the number of people served by each system. Comment on the impact of existing system on drinking water quality and environment.

(h) Drainage and Solid Wastes

Briefly describe existing systems of storm water drainage and solid waste collection and disposal. This discussion should be focussed in terms of their impact on water supply and environment.

(i) Need for a Project

Comment as to why the existing system cannot satisfy the existing and projected demands for services with reference to population to be served and the desired service standards, other demands like commercial and industrial. Describe the consequences of not taking up a project (which may include rehabilitation/augmentation of the existing system and/or developing a new system), indicate priorities to improvement of existing system, expansion of system, construction of new system, supply for domestic use, industrial and commercial use; assessment of the need for consumer education in hygiene; and comments on urgency of project preparation and implementation.

3.3.1.4 Long Term Plan for Water Supply

(a) Improvement in water supply services has to be planned as a phased development program and any near-term project should be such as would fit in the long term strategy. Such a long term plan or the strategic plan should be consistent with the future overall development plans for the areas. A long term plan may be prepared for a period of 25 to 30 years, and alternative development sequences may be identified to provide target service coverage and standards at affordable costs. From these alternative development sequences, a priority project to be implemented in near-term can be selected. It is this project which then becomes the subject of a comprehensive feasibility study.

(b) Alternative development sequences should be identified in the light of the service coverages to be achieved during the planning period in phases. This calls for definition of the following:

- Population to be covered with improved water supply facility
- Other consumers to be covered (industrial, commercial, government, institutions, etc.)
- Service standards to be provided for various section of population (e.g. House connections, yard-taps, public stand post and point sources)
- Target dates by which the above mentioned service coverage would be extended within the planning period, in suitable phases

(c) It must be noted that service standards can be upgraded over a period of time. Therefore, various options can be considered for different areas. While selecting service standard, community preferences and affordability should be ascertained through dialogue with intended beneficiaries. Only those projects which are affordable to the people they serve, must be selected. This calls for careful analysis of the existing tariff policies and practices, cost to the users for various service standards and income of various groups of people in the project area.

(d) Having determined the service coverage in stages over a planned period, requirements of water can be worked out for each year (or in suitable stages), adopting different standards; at different stages. To this may be added the demand for industrial, commercial and institutional users. Thus, water for the projected needs throughout the planned period can be quantified, (duly considering realistic allowances for unaccounted for water and the daily and seasonal peaking factors) for alternative service standards, and service coverage. These demands form the basis for planning and providing system requirements.

The annual water requirements should also take into consideration water demands for upgrading sanitation facilities, if proposals to that effect are under consideration. Consistency and coordination has to be maintained between projections for both water supply and sanitation services.

(e) It must be noted that availability of funds is one of the prime factors which will ultimately decide the scope and scalc of a feasible project.

(f) Selection of a Strategic Plan

Each of the alternative development sequences, which can overcome the existing deficiencies and meet the present and future needs, consists of a series of improvements and expansions to be implemented over the planned period. Since all needs cannot be satisfied in immediate future, it is necessary to carefully determine priorities of target groups for improvement in services and stages of development and thus restrict the number of alternatives.

(g) Planning For System Requirement Includes Consideration Of The Following

- Possibilities of rehabilitating and/or de-bottlenecking the existing systems
- Reduction in water losses which can be justified economically, by deferring development of new sources
- Alternative water sources, surface and ground water with particular emphasis on maximising the use of all existing water sources
- Alternative transmission and treatment systems and pumping schemes
- Distribution system including pumping station and balancing reservoirs
- Providing alternative service standards in future, including upgrading of existing facilities and system expansion

(h) Need Assessment For Supporting Activities

It may also be necessary to ascertain if supporting activities like health education, staff training and institutional improvements etc. are necessary to be included as essential components of the project. All the physical and supporting input need to be carefully costed (capital and operating) after preparing preliminary designs of all facilities identified for each of the alternative development sequences. These alternatives may then be evaluated for least cost solution by net present value method; which involves

- Expressing all costs (capital and operating) for each year in economic terms;
- Discounting future costs to present value;
- Selecting the sequence with the lowest present value.

(i) Costings And Their Expressions

As stated above, costs are to be expressed in economic terms and not in terms of their financial costs. This is because the various alternatives should reflect resource cost to the economy as a whole at different future dates. Costing of the selected project may however, be done in terms of financial costs, duly considering inflation during project implementation.

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3.3.1.5 Proposed Water Supply Project

(a) Details Of The Project

The project to be selected are those components of the least cost alternative of development sequence, which can be implemented during the next 3-4 years. Components of the selected project may be as follows:

- Rehabilitation and de-bottlenecking of the existing facilities
- Construction of new facilities for improvement and expansion of existing systems
- Support activities like training, consumer education, public motivation etc. Equipment and other measures necessary for operation and maintenance of the existing and expanded systems
- Consultancy services needed (if any) for conducting feasibility study, detailed engineering, construction supervision, socio-economic studies, studies for reducing water losses, tariff-studies, studies for improving accounts support activities

(b) Support Documents Required

All project components should be thoroughly described, duly supported by documents such as:

- Location maps
- Technical information for each physical component, and economic analysis where necessary
- Preliminary engineering designs and drawings in respect of each physical component, such as head works, transmission mains, pumping stations, treatment plants, balancing reservoirs, distribution lines

(c) Implementation Schedule

A realistic implementation schedule should be presented, taking into consideration time required for all further steps to be taken, such as conducting feasibility study, appraisal of the project, sanction to the project, fund mobilisation, implementation, trial runs and commissioning. In preparing this schedule, due consideration should be given to all authorities/groups whose inputs and decisions can affect the project and its timing.

(d) Cost Estimates

Cost estimates of each component of the project should be prepared and annual requirement of funds for each year should be worked out, taking into consideration the likely annual progress of each component. Due allowance should be made for physical contingencies and annual inflation. This exercise will result in arriving at total funds required annually for implementation of the project.

(e) Environment And Social Impact

The pre-feasibility report should bring out any major environment and social impact the project is likely to cause and if these aspects will affect its feasibility.

(f) Institutional Responsibilities

The pre-feasibility report should identify the various organisations/departments/agencies who would be responsible for further planning and project preparation, approval, sanction, funding, implementation and operation and maintenance of the project and indicate also the strength of personnel needed to implement and later operate and maintain the project. It should also discuss special problems likely to be encountered during operation and maintenance, in respect of availability of skilled and technical staff, funds, transport, chemicals, communication, power, spare parts etc. Quantitative estimates of all these resources should be made and included in the project report.

(g) Financial Aspects

The capital cost of a project is a sum of all expenditure required to be incurred to complete design and detailed engineering of the project, construction of all its components including support activities and conducting special studies. After estimating component-wise costs, they may also be worked out on annual basis, throughout the implementation period, taking into consideration construction schedule and allowances for physical contingencies and inflation. Basic item costs to be adopted should be of the current year. Annual cost should be suitably increased to cover escalation costs during the construction period. Total of such escalated annual costs determines the final cost estimate of the project. Financing plan for the project should then be prepared, identifying all the sources from which funds can be obtained, and likely annual contribution from each source, until the project is completed. The possible sources of funds include:

- Cash reserves available with the project authority
- Cash generated by the project authority from sale of water from the existing facilities
- Grant-in-aid from government
- Loans from government
- Loans from financing institutions like Life Insurance Corporation, Banks, HUDCO etc.
- Open market borrowings
- Loans/grants from bilateral/international agencies
- Capital contribution from voluntary organisations or from consumers

(h) If the lending authority agrees, interest payable during implementation period can be capitalised and loan amount increased accordingly.

(i) The next step is to prepare recurrent annual costs of the project for the next few years (say 10 years) covering operating and maintenance expenditure of the entire system (existing and proposed). This would include expenditure on staff, chemicals, energy, spare parts and other materials for system operation, transportation, up-keep of the systems and administration.

The annual financial burden imposed by a project comprises the annual recurring cost and payment towards loan and interest (debt servicing). This has to be met from the operational revenue, which can be realised from sale of water. The present and future tariff for sale of water should be identified and a statement showing annual revenue for ten years period, beginning with the year when the project will be operational, should be prepared. if this statement indicates that the project authority can generate enough revenue to meet all the operational expenditure as well as repayment of loan and interest, the lending institution can be persuaded to sanction loans for the project.

(j) Every State Government and the Government of India have programmes for financing water supply scheme in the urban and rural areas, and definite allocations are normally made for the national plan periods. It will be necessary at this stage to ascertain if and how much finance can be made available for the project under consideration, and to estimate annual availability of funds for the project till its completion. This exercise has to be done in consultation with the concerned department of the Government and the lending institutions, who would see whether the project fits in the sector policies and strategies, and can be brought in an annual planning and budgetory cycle taking into consideration the commitments already made in the sector and the overall financial resources position. The project may be finally sanctioned for implementation if the financing plan is firmed up.

3.3.1.6 Conclusions And Recommendations

(a) Conclusions

This section should present the essential findings and results of the pre-feasibility report. It should include a summary of:

- Existing service coverage and service standards
- Review of the need for the project
- Long-term development plans considered
- The recommended project, its scope in terms of service coverage and service standards and components
- Priorities concerning target-groups and areas to be served by the project
- Capital costs and tentative financing plan
- Annual recurring costs and debt servicing
- Tariffs and projection of operating revenue
- Urgency for implementation of the project
- Limitation of the data/information used and assumptions and judgements made; need for indepth investigation, survey, and revalidation of assumption and judgements, while carrying out feasibility study.

The administrative difficulties likely to be met with and risks involved during implementation of the project should also be commented upon. These may pertain to boundary question for the project area, availability of water, sharing of water sources with other users, availability of land for constructing project facilities, coordination with the various agencies, acceptance of service standards by the beneficiaries, tenancy problems, acceptance of recommended future tariff, shortage of construction materials, implementation of support activities involving peoples' participation, supply of power, timely availability of funds for implementation of the project and problems of operation and maintenance of the facilities.

(b) Recommendations

(i) This should include all actions required to be taken to complete project preparation and implementation, identifying the agencies responsible for taking these actions. A detailed time table for actions to be taken should be presented. if found necessary and feasible, taking up of works for rehabilitating and/or de-bottlenecking the existing system should be recommended as an immediate action. Such works may be identified and costed so that detailed proposals can be developed for implementation.

(ii) It may also be indicated if the project authority can go ahead with taking up detailed investigations, data collection and operational studies, pending undertaking feasibility study formally.

(iii) In respect of smaller and medium size projects, the pre-feasibility report can be considered sufficient for obtaining investment decision for the project if :

- The results of the pre-feasibility study are based on adequate and reliable data/ information
- Analysis of the data and situation is carried out fairly intensively
- No major environmental and social problems are likely to crop up that might jeopardise project implementation
- No major technical and engineering problems are envisaged during construction and operation of the facilities

(iv) In that case the pre-feasibility study with suitable concluding report, should be processed for obtaining investment decision for the project. The feasibility study, can then be taken up at the beginning of the implementation phase and results of the study if noticed to be at variance with the earlier ones, suitable modification may be introduced during implementation.

(v) In respect of major projects however, and particularly those for which assistance of bilateral or international funding agencies is sought for, comprehensive feasibility study may have to be taken up before an investment decision can be taken.

3.4 FEASIBILITY REPORT

Feasibility study examines the project selected in the pre-feasibility study as a nearterm project, in much greater details, to see if it is feasible technically, financially, economically, socially, legally, environmentally and institutionally. Enough additional data/ information may have to be collected to examine the above mentioned aspects, though the details necessary for construction of project components may be collected during execution of works.

It is a good practice to keep the authority responsible for taking investment decision, informed of the stage and salient features of the project. if there are good prospects of the project being funded immediately after the feasibility study is completed, detailed engineering of priority components may be planned simultaneously.

3.4.1 CONTENTS

The feasibility report may have the lowing sections :

- Background
- The proposed project
- Institutional and financial aspects
- Conclusion and recommendations

3.4.1.1 Background

In this section describe the history of project preparation, how this report is related to other reports and studies carried out earlier and in particular its setting in the context of a pre-feasibility report. It should also bring out if the data/information and assumption made in the pre-feasibility report are valid, and if not, changes in this respect should be highlighted. References to all previous reports and studies should be made.

In respect of the project area, need for a project and strategic plan for water supply, only a brief summary of the information covered in pre-feasibility report, should be presented, highlighting such additional data/ information, if any, collected for this report. The summary information should include planning period, project objectives, service coverage, service standards considered and selected for long-term planning and for the project, community preferences and affordability, quantification of future demands for services, alternative strategic plans, their screening and ranking, recommended strategic plan and cost of its implementation.

3.4.1.2 The Proposed Project

This section describes details of the project recommended for implementation. Information presented here is based on extensive analysis and preliminary engineering designs of all components of the project. The detailing of this section may be done in the following sub-sections:

(a) Objectives

Project objectives may be described in terms of general development objectives such as health improvements, ease in obtaining water by consumers, improved living standards, staff development and institutional improvements; and also terms of specific objectives such as service coverage and standards of service to be provided to various target groups.

(b) Project Users

Define number of people by location and institutions who will benefit and/or not benefit from the project area and reasons for the same, and users involvement during preparation, implementation and operation of the project.

(c) Rehabilitation and De-bottlenecking of the Existing Water Supply Systems

In fact rehabilitation, improvements and de-bottlenecking works, if necessary, should be planned for execution prior to that of the proposed project. If so these activities should be mentioned in the feasibility report. if, however, these works are proposed as components of the proposed project, necessity of undertaking the rehabilitation/improvement/ de-bottlenecking works should be explained.

(d) Project Description

This may cover the following items in brief:

- Definition of the project in the context of the recommended development alternative (strategic plan) and explanation for the priority of the project
- Brief description of each component of the project, with maps and drawings
- Functions, location, design criteria and capacity of each component
- Technical specification (dimension, material) and performance specifications
- Stage of preparation of designs and drawings of each component
- Method of financing and constructing in-house facilities, like plumbing and service connection etc.

(e) Support Activities

Need for and description of components such as staff training, improving billing and accounting, consumer education, health education, community involvement etc. and timing of undertaking these components and the agencies involved.

(f) Integration Of The Proposed Project With The Existing And Future Systems

Describe how the various components of the proposed project would be integrated with the existing and future works.

(g) Agencies Involved In Project Implementation And Relevant Aspects

- Designate the lead agency
- Identify other agencies including government agencies who would be involved in project implementation, describing their role, such as granting administrative approval, technical sanction, approval to annual budget provision, sanction of loans, construction of facilities, procurement of materials and equipment etc.

- Outline of arrangements to coordinate the working of all agencies
- Designate the operating agency and its role during implementation stage
- Role of consultants, if necessary, scope of their work, and terms of reference
- Regulations and procedures for procuring key materials and equipment, power, and transport problems, if any,
- Estimate number and type of workers and their availability
- Procedures for fixing agencies for works and supplies and the normal time it takes to award contracts
- List of imported materials, if required, procedure to be followed for importing them and estimation of delivery period
- Outline any legislative and administrative approvals required to implement the project, such as those pertaining to riparian rights, water quality criteria, acquisition of lands, permission to construct across or along roads and railways, high-tension power lines, in forest area and defence or other such restricted areas
- Comments on the capabilities of contractors and quality of material and equipment available indigenously

(h) Cost Estimates

- Outline basic assumption made for unit prices, physical contingencies, price-contingencies and escalation
- Summary of estimated cost of each component for each year till its completion and work out total annual costs, to know annual cash flow requirements
- Estimate foreign exchange cost if required to be incurred
- Work out per capita cost of the project on the basis of design population, cost per unit of water produced and distributed and compare these with norms, if any, laid down by government or with those for similar projects

(i) Implementation Schedule

Prepare a detailed and realistic implementation schedule for all project components, taking into consideration stage of preparation of detailed design and drawings, additional field investigations required, if any, time required for preparing tender documents, notice period, processing of tenders, award of works/supply contract, actual construction period, period required for procurement of material and equipment, testing, trials of individual component and commissioning of the facilities etc.

If consultants' services are required, the period required for completion of their work should also be estimated.

A detailed PERT diagram (ref. Appendix 3.1) showing implementation schedule for the whole project, as well as those for each component should be prepared, showing linkages and inter-dependence of various activities.

Implementation schedule should also be prepared for support-activities such as training, consumers' education etc. and their linkages with completion of physical components and commissioning of the project should be established.

(j) Operation And Maintenance Of The Project

Estimate annual operating costs, considering staff, chemicals, energy, transport, routine maintenance of civil works, maintenance of electrical/mechanical equipment, including normal cost of replacement of parts and supervision charges. Annual cost estimates should be prepared for a period of 10 years from the probable year of commissioning the project, taking into consideration expected out-put levels and escalation.

Proposal for monitoring and evaluating the project performance with reference to project objectives should be indicated.

(k) Environmental Impact

Brief description of the adverse and beneficial impacts of the project may be given covering the following aspects:

BENEFICIAL IMPACT	ADVERSE IMPACT
Ease and convenience in obtaining water by the consumers	Risk of promoting mosquito breeding, effect of with-drawing surface/ground water
Improvement in public reuse of water in household premises or by water authority.	Effect of disposal of backwash water and sludge from water treatment plant.
Effect of construction of storage reservoirs on flood moderation, navigation, ground water table, power generation etc.	Effects of construction of storage reservoirs on ground water table, down stream flow of the stream, the reservoir bed etc. and effects on ecology.

3.4.1.3 Institutional And Financial Aspects

(a) Institutional Aspects

It is necessary to examine capabilities of the organisations who would be entrusted with the responsibility of implementing the project and of operating the same after it is commissioned. The designated organisation(s) must fulfil the requirements in respect of organisational structure, personnel, financial, health and management procedures, so that effective and efficient performance is expected. This can be done by describing the following aspects :

- History of the Organisation, its functions, duties and powers, legal basis, organisational chart, (present and proposed), relationship between different functional groups of the organisation, and with its regional offices, its relation with government agencies and other organisations involved in sector development

- Public relations in general and consumer relations in particular, extension services available to sell new services, facilities for conducting consumer education programme, and settling complaints

- Systems for budgeting for capital and recurring expenditure and revenue, accounting of expenditure and revenue, internal and external audit arrangements, inventory management

- Present positions and actual staff, comments on number and quality of staff in each category, ratio of staff proposed for maintenance and operation of the project to the number of people served, salary ranges of the staff and their comparison with those of other public sector employees

- Staff requirement (category wise) for operating the project immediately, after commissioning, future requirements, policies regarding staff training, facilities available for training

- Actual tariffs for the last 5 years, present tariff, tariff proposed after the project is commissioned, its structures, internal and external subsidies, procedure required to be followed to adopt, new- tariff, expected tariff and revenues in future years, proposal to meet shortage in revenue accruals

- Prepare annual financial statements (income statements, balance sheets and cash flows) for the project operating agency, for three years after the project is commissioned, explain all basic assumptions for the financial forecast and the terms and conditions of tapping financial sources, demonstrate ability to cover all operating and maintenance expenditure and loan repayment, workout rate of return on net fixed assets and the internal financial rate of return of the project

(b) Financing Plan

Identify all sources of funds for implementation. of the project, indicating year-by-year requirements from these sources, to meet expenditure as planned for completing the project as per schedule; state how interest during construction will be paid, or whether it will be capitalised and provided for in the loan; explain the procedures involved in obtaining funds from the various sources.

3.4.1.4 Conclusions And Recommendations

This section should discuss justification of the project, in terms of its objectives, cost-effectiveness, affordability, willingness of the beneficiaries to pay for services and the effect of not proceeding with the project.

Issues which are likely to adversely affect project implementation and operation should be outlined and ways of tackling the same should be suggested. Effect of changes in the assumptions made for developing the project, on project implementation period, benefits, tariff, costs and demand etc. should be mentioned.

Definite recommendations should be made regarding time-bound actions to be taken by the various agencies, including advance action which may be taken by the lead agency pending approval and financing of the project.

CHAPTER 4
MEASUREMENT OF FLOW

4.1 POINTS OF MEASUREMENT

The measurement of flow in water supply systems is of importance in connection with assessment of source and its development, transmission, treatment, distribution, control of wastage and other factors.

The probable locations where flow measurement may be needed in a water supply system are:

(a) River flow gauging-upstream of intake-by floats and current meters or weirs and flumes or dilution methods.

(b) Measuring yield from wells (yield test) using the head differential through an orifice meter or venturi meter for pipe flows or by weirs or flumes for open channel flow.

(c) Intake structure-raw water input rate by venturi or orifice meter for pipe flows or by weirs or flumes for open channel flow.

(d) Flow at the entry to the treatment works (normally after aeration if it is practiced) by weirs or flumes.

(e) Filtrate flow from each filter by weirs or notches or orifice meters or venturi meters.

(f) Bulk flow measurements of water supplied from treatment plant and clear water reservoir by venturi meter.

(g) Bulk flow measurements (integrating and instantaneous) for supply to distribution zones, sub-zones or industries by bulk meters or venturi meters.

(h) Measurement of domestic water supply through service connections by domestic consumer water meters.

(i) Assessment of wastages and leakages in pipes and plumbing systems by waste flow measuring or recording meters.

There are several types of flow measurements of which the more common ones are described below with some detail. The choice of the particular type depends on the specific circumstances and desired accuracy.

4.2 MEASUREMENT IN OPEN CHANNELS

4.2.1. USE OF HYDRAULIC STRUCTURES

Several types of hydraulic structures like notches, weirs, flumes and drops are in use for measurement of flow in open channels.

4.2.1.1. Notches

These are cut from thin metal plates, the general forms being either triangular or trapezoidal.

(a) Triangular Notches

90^0 triangular notches are used for measuring small quantities of flows upto about 1.25 m^3/s

(i) Installation Requirements

The approach channel should be reasonably smooth, free from disturbances and straight for a length equal to at least 10 times the width. The structures in which the notch is fixed shall be rigid and water-tight and the upstream face vertical. The downstream level should be always at least 5 cm below the bottom-most portion of the notch (inverted apex) ensuring free flow.

(ii) Specification for Materials

The plate should be smooth and made of rust-proof and corrosion-resistant material. The thickness should not exceed 2 mm, with the downstream edge chamfered at an angle of not less than 45^0 with the crest surface.

(iii) Measurement of Head Causing the Water Flow

The head causing flow over the notch shall be measured by standard hook gauge upstream at a distance of 3 to 4 times the maximum depth of flow over the notch.

(iv) Discharge Equation

The discharge Q (in m^3/sec) for V-Notch is given by the expression :

$$Q = \frac{8}{15} C_e \sqrt{2g} \tan\frac{\theta}{2} h^{2.5} \tag{4.1}$$

where,

C_e	=	effective discharge coefficient
g	=	acceleration due to gravity (9.806 m/s^2)
θ	=	angle of the notch at the centre
h	=	measured head causing flow in m,

For 90^0 V-Notch which is generally used, the discharge is given by the expression

$$Q = 2.362\, C_e\, h^{2.5} \quad (4.2)$$

C_e values vary from 0.603 to 0.686 for values of head varying from 0.060 to 0.377m.

(v) Limitations

The triangular notches should be used only when the head is more than 60 mm.

(vi) Accuracy

The values obtained by the equation for triangular notches would vary from 97 to 103% of the true discharge for discharges from 0.008 to 1.25 m^3/s.

(b) Rectangular Notches

The installation requirements, specifications, head measurements, head limits and accuracy will be the same as for triangular notches. The width of notch should be at least 150 mm.

There are two types of rectangular notches viz. (i) with end contractions and (ii) without end contractions.

(i) With End Contractions

The contraction from either side of the channel to the side of the notch should be greater than 0.1 m.

The discharge (m^3/s) through a rectangular notch with end contractions is given by the equation:

$$Q = \frac{2}{3} C_e \sqrt{2g}\; b_e H^{1.5} \quad (4.3)$$

where,

b_e = effective width = actual width of the notch + k (value of k being 2.5 mm, 3 mm and 4 mm for b/B ranges of upto 0.4, 0.4 to 0.6 and 0.6 to 0.8 respectively);

$\frac{b}{B}$ = ratio of the width of the notch to the width of the channel;

H = effective head = actual head measured (h) + 1 mm;

g = acceleration due to gravity (9.806 m/s^2) ; and

C_e = varies from 0.58 to 0.70 for values of b/B from 0 to 0.8.

(ii) Without End Contractions

The discharge (m^3/s) through a rectangular notch without end contractions is given by the following expression:

$$Q = \frac{2}{3} C_e \sqrt{2g} b H^{1.5} \quad (4.4)$$

where,

b = width of the notch (m)

H = effective head = actual /measured head (h) + 1.2 mm

$C_e = 0.602 + 0.075\ h/p$

where,

p = height of the bottom of the notch from the bed of the channel

(c) Trapezoidal Notches (Cipoletti Notches)

The main advantage in a trapezoidal or Cipoletti notch is that as the flow passes over the weir, the end contractions are either eliminated or considerably reduced. The sides of the notch should have a slope of 1 : 4 such that the top width of discharge is equal to the bottom width of the notch (b) + half the head of water over the sill of the notch (1/2 h). Thus the loss of discharge due to end contractions is made good. Discharge equation $Q = 1.859\ bh^{3/2}$ where b is bottom width of notch and h is the head over the sill.

4.2.1.2 Weirs

These are similar to rectangular notches but the thickness in the direction of flow is considerable and therefore coefficient of discharge will be less. The installation conditions will be the same as for the notches.

(a) Without End Contractions (Suppressed Weirs)

The discharge equation to be used is:

$$Q = 0.5445 C_e \sqrt{g} b h^{1.5} \tag{4.5}$$

C_e varies from 0.864 to 1.0 depending upon the h/p (ratio of measured head to length of weir in the direction of flow) value from 0.4 to 1.6; for h/p values lower than 0.4, C_e may be taken as 0.864.

(b) With End Contractions

Same equation 4.5 is to be used replacing the 'b' by '(b-0.1 nh)' where n is the number of contractions.

(c) Limitations

The weirs should be used only when the head is more than 60 mm. Minimum width of the weir should be 300 mm.

(d) Accuracy

The discharge values obtained by weir measurements would vary from 95 to 105% of the true discharge.

4.2.1.3 Flumes (Free Flowing)

There are two types of flumes, namely:

- Standing wave flumes in which standing wave of hydraulic jumps is formed down stream.

- **Venturi flumes**

The installation conditions will be the same as for the notches.

(a) Standing Wave Flumes

(i) **Discharge equation** : The discharge equation for standing wave fumes is given by :

$$Q = \frac{2}{3}\sqrt{2g}C_f(B_0 - mb - 2C_c mH)H^{1.5} \quad (4.6)$$

Where,

Q = discharge in m^3/s

C_f = coefficient of friction having the following values

0.97 for Q = 0.05 to 0.3 m^3/s

0.98 for Q = 0.3 to 1.5 m^3/S

0.99 for Q = 1.5 to 15 m^3/s

1.00 for Q = 15m^3/s and above

B_0 = overall throat width including piers

m = number of piers

b = thickness of each pier

C_c = Coefficient of contraction, having a value of 0.045 for piers with round nose and 0.040 for piers with pointed nose and $H = D_1 + h_v$ = upstream head over sill corrected for velocity of approach

$$H = D_1 + \frac{V_a^2}{15.2}$$

Where,

D_1 = the depth upstream over sill of throat and

V_a = the mean velocity of approach. Effect of velocity of approach is greater than $V_a^2/2g$ because the velocity in the central portion will be higher than V_a. Therefore, the head due to velocity of approach should be taken as :

$$h_v = \frac{V_a^2}{15.2}$$

(ii) Limitations

Standing wave flumes should be used only when the head is more than 60 mm. Ratio of D_2/D_1 (Depth downstream above sill of throat/depth upstream over sill of throat) should always be greater than 0.5 for the application of standing wave flumes. If this ratio is less than 0.5, drop may be adopted.

Minimum width of the flumes should be 90 mm.

(iii) Accuracy

The discharge values obtained by measurements with standing wave flumes would vary from 95 to 105% of the true discharge.

Parshall Flume is a type of standing wave flume widely used. However, its use requires application of different equations, based on the throat size, if accuracy in results similar to other types of flumes is expected.

The approximate equation applicable for the entire range of its usage, namely, discharges varying from 0.001 m^3/s to 100 m^3/s (i.e. throat widths varying from 75 to 15,000 mm) is given by:

$$Q = 2.42\ W\ h^{2.58}$$

Where,

Q = discharge in m^3/s

W = throat width in m and

h = upstream gauged depth in m,

The numerical factors 2.42 and 2.58 are subject to 4% variation in extreme cases (less in case of smaller widths).

The minimum head and accuracy will be the same as for standing wave flumes.

(b) Venturi Flumes

(i) Discharge equation

The discharge equation is given by

$$Q = 0.5445 C_v C_e \sqrt{g} b h^{1.5} \qquad (4.8)$$

Where,

C_v is the coefficient of velocity which varies from 1.04 to 1.15.

C_e is the effective coefficient of discharge varying from 0.885 to 0.99 depending upon h/l varying from 0.05 to 0.70 where 'l' is the length of throat in the direction of flow.

(ii) Limitations

Venturi flumes should be used only when head available is between 50 and 1800 mm. Minimum width of the flume should be 90 mm.

(iii) Accuracy

The discharge values obtained by measurement with venturi volumes would vary from 95 to 105% of the true discharge.

4.2.1.4 Drops

(i) Discharge Equation

When the flow falls freely from a channel or conduit to a lower level (ground), measurement can be conveniently made at the point of drop which offers a rough estimate of the discharge. There should be a minimum straight length of 20 times the end depth in the approach channel. The ratio of the end depth to the critical depth in horizontal and mildly sloped channels has a value of 0.70. The discharge may be calculated from

$$Q = d_c^{1.5}\sqrt{g}b \tag{4.9}$$

Where,

d_c = critical depth (m)

b = width of channel (m)

(ii) Limitations

Width of channel should be a minimum of 300 mm. Critical depth dc should be a minimum of 50 mm.

(iii) Accuracy

The discharge values obtained by measurements made at drops would vary from 90 to 110% of the true discharge.

4.2.2 Velocity Area Methods

The rate of flow through a section of a pipe or open channel is often determined by multiplying the cross sectional area of water at the section at right angles to the flow by the mean velocity of water at the section. Cross sectional area is usually determined by direct measurements. Determination of the mean velocity is generally more difficult and time consuming, since the velocity differs considerably from point to point in the cross section. For determining the mean velocity, several methods such as use of current meter, float, velocity rod, pitot tube, tracer technique and trajectory method are available.

When velocity measurements are made at only one point, this point is usually around 0.6 mid-depth. The exact location of this point is decided on the basis of vertical velocity distribution experiments.

Average velocity of flow at any subsection of the cross section can be approximated by the average of velocities at 0.2 and 0.8 depths in that subsection. The cross section is accordingly divided into various small vertical sections and average velocity v_i of each section is found.

The mean velocity of flow in the cross section is found by the expression

$$\frac{\sum_{i=1}^{n}(a_i v_i)}{\sum_{i=1}^{n} a_i} \tag{4.10}$$

Where a_i is area of the individual section and v_i is the average velocity in that section.

The velocities are usually obtained by current meter. For floats, the surface velocities are found and the average velocity is computed approximately as 0.87 of surface velocity. Normally the discharge measurements are 95 to 105% of the true discharges.

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4.2.3 ELECTRO MAGNETIC PROBE METHOD

When an electro-magnetic probe is immersed in flowing water, a voltage is created around the probe. This voltage, sensed by electrodes imbedded in the probe is transmitted through the cable to the meter box. The voltage created by water flowing through the magnetic field is proportional to the velocity of flow of water. These small voltages are electronically processed and displayed on the panel meter. Accuracies of ± 2% are attainable over a velocity range from 1.5 cm/s to 3.0 m/s.

4.3 MEASUREMENT IN CLOSED CONDUITS

4.3.1 DIFFERENTIAL PRESSURE DEVICES

The venturi meters, orifice plates and nozzles are used specifically for closed conduits. They shall have minimum straight length of 5D on upstream side and 2D on the downstream side of the device (where D is the diameter of upstream pipe).

4.3.1.1 Venturi Meters

Venturi meters provide a most dependable relation of differential pressure to velocity through the ranges of flow required by engineering practice and return of at least 85% of the velocity head when constructed in accordance with standard proportions. Of extreme importance is the establishment of the accuracy of their coefficients, which give them preference as a means for producing suitable velocity heads.

Standard venturi meters usually are constructed with piezo-meter rings at the main and throat sections which are connected to the interior surface of the meters. Alternatively the pressure chambers could be omitted and pressure taps at main and throat are provided. Each of these taps is equipped with a manually operated cleaning valve.

Where fluids contain sediment or carry substance that may tend to clog the piezometer openings, clear water flushing disconnectors and cleaning valves at both main and throat sections are included.

Under special conditions, a venturi with a circular inlet and outlet and an elliptical throat section, providing a flat invert as well as a flat top for the entire length of the tube can be employed. The flat invert is self scouring and prevents accumulation of grit or other solids under low flow conditions while the flat top prevents the trapping and accumulation of air and gases, which under some conditions could adversely affect the accuracy of the instrument reading.

Discharge through a venturi meter is given by the expression

$$\text{Discharge } Q = K \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \cdot \sqrt{2gh} \qquad (4.11)$$

Where,

a_1 = area of the pipe in m^2

a_2 = area at the throat section in m^2

h = sum of the difference between pressure heads and potential heads at the inlet and throat sections, in m

The coefficient K varies from 0.95 to 0.98.

The ratio of the diameter at the throat to the diameter at normal inlet section varies from 1/4 to 3/4 and the usual ratio is 1/2. Smaller ratios give increased accuracy of gauge reading but are accompanied by higher frictional losses and low pressure at the throat which could lead to cavitation. The angles of convergence and divergence in a venturimeter are 20^0 and 5^0 respectively.

4.3.1.2 Orifice Plates And Nozzles

For computing the discharge in orifice plates and nozzles, the following expression is used :

$$Q = \alpha \frac{\pi}{4} d^2 \sqrt{\frac{2\Delta p}{\rho}} \qquad (4.12)$$

Where,

d = diameter of the orifice or nozzle, in m.

p = differential pressure in kgf/m^2 and

ρ = density of water (kg/m^3).

and α varies from 0.6 to 0.765 for orifice plates for flows with Reynold's number from $5x10^3$ to $1x10^7$. Similarly α varies from 0.99 to 1.19 for nozzles for flows with Reynold's numbers from $2x\ 10^4$ to $1x10^6$. Both give 98 to 102% of true discharge.

4.3.1.3 Pitot Tubes

Pitot tube, in which the velocity head is converted to static head, is one of the device for measuring the flow rate in a pipe or discharge from a pump under test after it has been installed. A straight length of pipe, of not less than 20 diameters upstream of the gauging point and 5 diameters on the downstream side is necessary.

It is most convenient to make the test gauging with the pitot tubes set to read the velocity of flow at the centre, the discharge being then obtained by multiplying this by the area of the pipe and by a coefficient (pipe factor) representing the mean velocity divided by the centre velocity. To obtain this coefficient, the pipe should be traversed (Pitot traverse) on two diameters at right angles to one another covering the range of the test.

The simple type of pitot tube consists of a single tube with an orifice facing upstream to take the impact pressure, the static pressure being obtained from connections in the wall of the pipe, care being taken that the end connections are flush with the inside of the pipe.

The pitot static tube has two orifices one facing upstream and taking the impact pressure, the other trailing and receiving a pressure less than the static The coefficient K in the velocity equation given below is not constant, but has to be obtained from the coefficient curve supplied by the manufacturers or to be calibrated. For a given velocity, this type produces a greater differential head than the simple type.

$$V = K\sqrt{2gH} \tag{4.13}$$

Where,

V = Velocity of flow, mps at the point

K = Instrument coefficient

g = gravitational head in m of water between the impact and static (or trailing) orifices

H = Differential head in meters of water between the impact and static (or trailing) orifices.

The coefficient has a value of about 0.99.

Pitot tubes would offer hindrance to flow and hence may be restricted to pipes larger than 300 mm dia. The values obtained with pitot tubes would vary from 98 to 102% of the true discharge for 1000 mm dia or larger pipes. For smaller diameters, the variation would be larger depending upon the obstruction caused.

4.3.1.4 Water Meters

Water meters are generally used for measuring flows in the mains and house service connections. They are of different types but inferential water meters of single jet or multiple jet with dry or wet dial are commonly in use.

(a) Domestic consumer meters

The domestic consumer meters normally suffer from the following deficiencies:

(i) They involve a high head loss and hence consumers are likely to bye pass the meter.

(ii) The minimum flow that can be registered is as high as 40 litres per hour.

(iii) Deposition of silt and foreign particles clogs the meter and hence the meter goes out of order.

(iv) Since the sealing is not fully water tight, the metallic gears get rusted and the plastic gears deteriorate due to which the meter goes out of order frequently.

(v) In the absence of hermetically sealed dials, there is an ingress of moisture on to the face of dial and hence the meter becomes unreadable.

(vi) Meters with pointers can be tampered by changing the position of pointer needles.

As per the amended IS 779 - 1994 (ISO 4064) for the domestic consumer meters, the meters are magnetically driven and hermetically sealed. It is preferable to use only magnetic meters ISO 4064

Salient features of these meters are (i) there is no contact of the meter mechanism with water, (ii) the meter starts registration at very small flows (minimum flow of 10 litres) with minimum head loss, (iii) the totaliser chamber remaining completely dry, (iv) the gears are self lubricating and readings can be directly read and are clearly visible in any weather.

Some of the advantages of these meters are:

(i) The inferential meters are magnetically driven. Since there is no contact of the meter mechanism with water, there is no friction. Hence the meter starts registration at very small quantities of flow (at 10 litres per hour) and involves a head loss of about 1.5 m.

(ii) The hermetically sealed meters cannot be tampered and the readings can be read directly. Further, in the absence of ingress of moisture, the dial is clearly visible.

(iii) Since the dial is hermetically sealed, the gear train is fully dry, is above the water and is self lubricating.

(iv) Since there is no change in direction of flow, the head loss through the meter is small.

(b) Bulk Meters

For use on distribution mains the bulk meters of Vane Wheel type with sizes of 50 to 300 mm or Helical type with sizes of 50 to 300 mm conforming to IS 2373 - 1981 are in use. These meters also suffer from the same deficiencies stated in previous section (a) for domestic meters.

The IS 2373 is being revised (fourth revision) to incorporate the following modifications which are likely to address some of the deficiencies;

(i) Indicating devices to include pointers, digital and combination of the two

(ii) Class A and Class B meters are to be introduced and performance requirements are to be more stringent

(iii) Pressure loss requirement is to be more stringent

(iv) Removable type helical meters to be introduced in addition to fixed type

(v) Sizes of 65 mm, 600 mm and 800 mm are to be added.

4.4 SPECIAL METHODS

4.4.1 GENERAL

There are several special methods. The dilution techniques and the pulse-velocity methods are applicable to both open channels as well as closed conduits. The trajectory measurements and bend or centrifugal head meters are applicable to closed conduits only. The more common method of dilution techniques is described below.

4.4.2 DILUTION METHOD

This is based on the fact that a chemical or radioactive tracer, injected into a river or pipe will be completely and uniformly mixed with the natural flow and that the diluted concentration down stream will decrease with increasing discharge. Chemical concentrations are measured by titration or colorimetric methods and radioactivity by Geiger counter. This method permits the direct computation of discharge without measurement of cross-sectional area. The usual tracers used are sodium chloride, sodium dichromate, manganese sulphate, sodium nitrite, lithium and potassium salts, dyes like sodium fluoroscein and radioactive isotopes.

These techniques are of particular value in hilly streams where the other methods of measurement gaugings are not feasible and also in waste water gaugings.

Analysis of concentrations of the injected solution and the diluted samples at a position far enough downstream for ensuring complete mixing, allows determination of the discharge of the stream. If C_1, and C_2 are the concentrations of injected solution and the diluted downstream samples respectively and Q and q are the discharge rates of the main flow to be measured and added chemical flow respectively, then

$$\frac{Q+q}{q} = \frac{C_1}{C_2} \tag{4.14}$$

if q is very small relative to Q,

then $$Q = q\frac{C_1}{C_2} \tag{4.15}$$

(a) Constant Rate Injection Method

The concentrated solution of chemical is usually prepared at the gauging site in a tank and thoroughly mixed. This is then injected into the stream at a controlled constant rate of flow, the device used being a constant volume displacement pump or constant head tank. The rate is measured with a flow meter in the line with a high degree of accuracy. A steady injection of chemical into the stream should continue for a period equal to the time for reaching steady conditions plus time of sampling at the sampling cross-section. A highly turbulent flow and a narrow reach are desirable. The reach should be long enough for complete mixing to occur. Empirical formulae are available in arriving at the mixing length. The samples are taken upstream from the injection point, from the concentrated solution at the point of injection and from two or three points at the sampling section.

(b) Integration (Sudden Injection) Method

This is preferred for very large flow measurements in natural stream. A known volume q of chemical tracer solution of concentration C_1 is introduced into the stream as quickly as possible. Sampling of water is carried out at a point sufficiently far downstream ensuring complete lateral mixing, for a period during which the tracer passes (which includes the complete injection cycle).

The stream discharge Q is given by the expression

$$Q = \frac{q}{T}\frac{C_1}{C_2} \tag{4.16}$$

Where T is the total sampling time and C_2 is the average concentration of tracer in water removed at the sampling point during the sampling period.

This expression holds when the tracer is not naturally present in the stream. In this method constant rate injection equipment is not required and the procedure is simple. No calibration is needed and it is not necessary to measure the dimensions of the test section. This method can be used with radioactive tracers.

CHAPTER 5
SOURCES OF SUPPLY

5.1 KINDS OF WATER SOURCES AND THEIR CHARACTERISTICS

The origin of all sources of water is rainfall. Water can be collected as it falls as rain before it reaches the ground; or as surface water when it flows over the grounds; or is pooled in lakes or ponds; or as groundwater when it percolates into the ground and flows or collects as groundwater; or from the sea into which it finally flows. The quality of the water varies according to the source as well as the media through which it flows.

5.1.1 WATER FROM PRECIPITATION

Rain-water collected from roofs or prepared catchments for storage in small or big reservoirs, is soft, saturated with oxygen and corrosive. Microorganisms and other suspended matters in the air are entrapped but ordinarily the impurities are not significant. But the collecting cisterns or reservoirs are liable to contamination.

5.1.2 SURFACE WATERS

(a) Natural Quiescent Waters As In Lakes And Ponds

These waters would be more uniform in quality than water from flowing streams. Long storage permits sedimentation of suspended matter, bleaching of color and the removal of bacteria. Self-purification which is an inherent property of water to purify itself is usually less complete in smaller lakes than in larger ones. Deep lakes are also subject to periodic overturns which bring about a temporary stirring up of bottom sediment. The microscopic organisms may be heavy in such waters on occasions. If the catchment is protected and unerodible, the stored water may not require any treatment other than disinfection.

(b) Artificial Quiescent Waters As In Impounding Reservoirs

Impounding reservoirs formed by hydraulic structures thrown across river valleys, are subject, more or less, to the same conditions as natural lakes and ponds. While top layers of water are prone to develop algae, bottom layers of water may be high in turbidity, carbon dioxide, iron, manganese and, on occasions, hydrogen sulphide. Soil stripping before impounding the water would reduce the organic load in the water.

(c) Flowing Waters As In Rivers, Other Natural Courses And Irrigation Canals

Waters from rivers, streams and canals are generally more variable in quality and less satisfactory than those from lakes and impounded reservoirs. The quality of the water depends upon the character and area of the watershed, its geology and topography, the extent and nature of development by man, seasonal variations and weather conditions. Streams from relatively sparsely inhabited watersheds would carry suspended impurities from eroded catchments, organic debris and mineral salts. Substantial variations in the quality of the water may also occur between the maximum and minimum flows. In populated regions, pollution by sewage and industrial wastes will be direct. The natural and man-made pollution results in producing color, turbidity, tastes and odors, hardness, bacterial and other micro-organisms in the water supplies.

(d) Sea Water

Though this source is plentiful, it is difficult to extract economically water of potable quality because it contains 3.5% of salts in solution, which involves costly treatment. Offshore waters of the oceans and seas have a salt concentration of 30,000 to 36,000 mg/l of dissolved solids including 19,000 mg/l of chloride, 10,600 mg/l of sodium, 1,270 mg/l of magnesium, 880 mg/l of sulphur, 400 mg/l of calcium, 380 mg/l of potassium, 65 mg/l of bromine, 28 mg/l of carbon, 13 mg/l of strontium, 4.6 mg/l of boron. Desalting or de-mineralizing processes involve separation of salt or water from saline waters. This is yet a costly process and has to be adopted in places where sea water is the only source available and potable water has to be obtained from it, such as in ships on the high seas or a place where an industry has to be set up and there is no other source of supply.

(e) Waste Water Reclamation

Sewage or other waste waters of the community may be utilized for non-domestic purposes, such as water for cooling, flushing, lawns, parks, etc., fire fighting and for certain industrial purposes, after giving the necessary treatment to suit the nature of use. The supply from this source to residences is prohibited because of the possible cross connection with the potable water supply system.

5.1.3 GROUNDWATER

(a) General

Rain water percolating into the ground and reaching permeable layers (aquifers) in the zone of saturation constitutes groundwater source. Groundwater is normally beyond the reach of vegetation except certain species of plants called phreatophytes, and is usually free from evaporation losses. Groundwater resources are less severely affected by vagaries of rainfall than surface water resources.

The water as it seeps down, comes in contact with organic and inorganic substances during its passage through the ground and acquires chemical characteristics representative of the strata it passes through.

Generally, groundwaters are clear and colorless but are harder than the surface waters of the region in which they occur. In limestone formations, groundwaters are very hard, tend to form deposits in pipes and are relatively non-corrosive. In granite formations they are soft, low in dissolved minerals, relatively high in free carbon dioxide and are actively corrosive. Bacterially, groundwaters are much better than surface waters except where subsurface pollution exists. Groundwaters are generally of uniform quality although changes may occur in the quality because of water logging, over-draft from areas adjoining saline water sources and recycling of water applied for irrigation and pollution.

While some of the chemical substances like fluorides and those causing brackishness are readily soluble in water, others such as those causing alkalinity and hardness, are soluble in water containing carbon dioxide absorbed from the air or from decomposing organic matter in the soil. Such decomposing matter also removes the dissolved oxygen from the water percolating through. Water deficient in oxygen and high in carbon dioxide dissolves iron and manganese compounds in the soil. Hydrogen Sulphide also occurs sometimes in groundwater and is associated with the absence of oxygen, the decomposition of organic matter or the reduction of sulphates. Percolation into the sub-soil also results in the filtering out of bacteria and other living organisms. in fissured and creviced rock formations such as limestone, however, surface pollution can be carried long distances without material change.

(b) Spring

Springs are due to the emergence of groundwater to the surface. Till it issues out on the surface as a spring, the groundwater carries minerals acquired from the subsoil layers, which may supply the nutrients to microorganisms collected *by* spring if it flows as a surface stream. Spring waters from shallow strata are more likely to be affected by surface pollutions than deep-seated waters. Springs may be either perennial or intermittent. The discharge of a spring depends on the nature and size of catchment, recharge and leakage through the sub-surface. Their usefulness as sources of water supply depends on the discharge and its variability during the year.

5.1.4 SALINE INTRUSION

Saline intrusion or salt water creep may occur in tidal estuaries or in groundwater. Longitudinal mixing in tidal estuaries is kept in check by the prevention of fresh water and salt water flow components to mix vertically. Engineering studies are needed to examine this salt water creep viz, the upstream progress of a tongue of salt water moving inland while overriding fresh water may still flow towards the sea or ocean. The salt content of such river waters may also vary with the tides and it is essential to determine the periods when the supply should be tapped to have the minimum salt content.

Groundwater in coastal aquifers overlies the denser saline water. Every metre rise of the water table above the sea level corresponds to a depth of 41 metres of fresh water lens floating over the saline water. In such cases the pumping from wells has to be carefully controlled or a fresh water barrier created to avoid the salt water tongue entering the well and contaminating the same.

5.1.5 SANITARY SURVEY

Though the specific characteristics of the several sources have been delineated above, the importance of sanitary survey cannot be over-emphasized. This survey is a study of the environmental conditions that may affect its fitness as a source. The scope of the sanitary survey should include a discerning study of the geological, geophysical, hydrological, climatic, industrial, commercial, agricultural, recreational and land development factors influencing the water drainage into the source and the surface and the subsurface pollutions likely to affect it. The subsurface pollution may be derived from privy pits, leaching cess pools, leaking sewers and land fills. Pollution introduced at or below the groundwater table is especially serious. Sources of pollution located on the ground surfaces are more easily countered than those on subsurface sources.

5.2 ASSESSMENT OF THE YIELD AND DEVELOPMENT OF THE SOURCE

5.2.1 GENERAL

A correct assessment of the capacity of the source investigated is necessary to decide on its dependability for the water supply project in view.

The capacity of flowing streams and natural lakes is decided by the area and nature of the catchment, the amount of rainfall and allied factors.

The safe yield of subsurface sources is decided by the hydrological and hydrogeological features relevant to each case.

5.2.2 FACTORS IN ESTIMATION OF YIELD

The incidence and the intensity of rainfall, the run-off from a given catchment and the actual gauged flows in streams are the main factors in estimating the safe yield from any source. Reliable statistics on the rainfall over representative regions of the catchment area, recorded through a number of years, should be collected wherever available. In order to cover deficiencies in such data, it is desirable that rainfall recording stations are set up over all water sheds as part of a total water conservation programme by the state public health engineering authority.

River gauging records should be collected and studied in regard to such sources under investigation. The setting up of river gauging stations should also form part of a total water conservation programme of the state government. In respect of groundwater, aquifer geometries, boundaries, and properties, groundwater levels, and surface-water ground water relationships should be studied for the estimation of the resource.

5.2.3 METHODS FOR ASSESSMENT OF SURFACE FLOWS

5.2.3.1 Computation Of Minimum And Maximum Discharges

(a) Use Of River Gauging Data

When river gauging data for at least 8 years is available, the minimum and maximum discharges likely once in a 30 year period may be statistically arrived at and adopted. A 100 year period may be used, if the data available is for a minimum of 25 years.

(b) Other Methods

When such data is lacking, the following methods may be adopted in the order of preference:

(i) Unit hydrograph method based on rainfall runoff studies;

(ii) Frequency analysis based on rainfall;

(iii) Envelope curves based on observed floods in similar catchments; and

(iv) Empirical formulae based on catchment characteristics.

(i) Unit hydrograph method

It is a hydrograph (discharge along Y axis and time along X axis) of rainfall-runoff at a given point that will result from an isolated event of rainfall excess (the portion of rainfall that enters a stream channel as storm runoff) occurring within a unit of time and spread in an average pattern over the contributing drainage area. This is identified by the unit time and unit volume of the excess raintall, e.g., 1 hour 1 cm unit graph.

The assumptions are:

(a) The effects of all physical characteristics of a given drainage basin are reflected in the shape of the hydrograph for that basin;

(b) At a given point on a stream, discharge ordinates of different unit graphs of the same unit time of rainfall excess are mutually proportional to respective volumes; and

(c) A hydrograph of storm discharge that would result from a series of bursts of excess rain or from continuous excess rain of variable intensity may be constructed from a series of overlapping unit graphs each resulting from a single increment of excess rain of unit duration.

The limitations are:

(a) The drainage basin should be more than 25 km^2 but less than 5000 km^2;

(b) Large number of rain gauges should be located to reflect the time weighted rainfall of the catchment; and

(c) The proportion of the snow in the precipitation should be very small.

(ii) Frequency analysis based on rainfall

This method involves the statistical analysis of observed data of a fairly long (at least 25 years) period by a suitable method such as Gumbels (see IS: 5477 Pt. II 1971).

A purely statistical approach when applied to derive design floods for long recurrence intervals several times larger than the data has many limitations and hence this method has to be used with caution.

(iii) Envelope Curves

In this method, maximum flood is obtained from the envelope curve of all the observed maximum floods for a number of catchments in a homogeneous meteorological region plotted against the drainage area. This method, although useful for generalizing the limits of floods actually experienced in the region under consideration, can not be relied upon for estimating maximum probable floods, except as an aid to judgement.

(iv) Empirical Formulae

The empirical formulae commonly used in the country are Dicken's formula, Ryve's formula and Inglis formula in which the peak flow is given as a function of the catchment area and a coefficient. The values of the coefficient vary within rather wide limits and have to be selected on the basis of judgement. They have limited regional application, and should be used with caution and only when a more accurate method can not be applied for lack of data.

From the observations of the unit hydrograph or the frequency analysis methods, floods occurring once in 100 years as also the least flow or minimum quantity likely to obtain once in 30 years may be determined.

5.2.3.2 Use Of Maximum And Minimum Discharge Figures-Mass Diagram

The maximum discharge figures are used for the design of the spill-ways of the dam of any impounding reservoir across the stream. The figures will also be useful in determining the maximum scour effects and the maximum water level likely to be attained, so that components of the project located in the river bed could be designed suitably.

The probable minimum flow as computed from the methods described above could be used for assessing the dependable yield from the source and for determining the maximum period of storage called for with the aid of a mass diagram drawn up for the purpose as detailed in Appendix-5.1.

While the computed figures for surface run-off from catchments contributing to the stream flow represents the total inflow into the river as surface flow from its catchment area, the stream discharge may be supplemented by the subsurface flow from the catchment basin, emerging into the stream through the subsoil, depending on the geological formations and hydrological conditions in the river valley. The subsurface seepage contributing to the river flow can not usually be computed by the use of any formula as such because of the several indefinite factors involved. Continuous river gaugings at any point however, would be the total discharge at the point, contributed both by the surface and subsurface flows which join the stream.

In assessing the dependable yield from natural lakes and ponds, the computations must be reckoned largely on the capacity of the basin, with reference to the total catchment area and the computable run-off available therefrom. Here again, supplemental quantities received by the basin through any subsurface flow from the catchment area is usually not a computable factor and usually not taken into account in assessing the total quantity available for the project. In all computations on the reservoir storage and capacity, probable losses due to seepage and evaporation should be given due consideration.

5.2.4 Assessment of Groundwater Resource Potential

Prior to the year 1979 for the assessment of replenishable groundwater resource potential, various methodologies were being adopted by the States and the Central Groundwater Board (CGWB). However, with a view to project a unified view and assessing the resource on scientific lines, a committee known as "Over Exploitation Committee" was constituted with the then Chairman, Central Groundwater Board as the Chairman to suggest methodology for estimation of the groundwater potential and also to lay down the norms for development of various types of structures and areas. The methodology suggested by the committee has been adopted by the Agricultural Refinance and Development Corporation (ARDC). The Committee had further recommended that the methodology may be further revised to make it more scientific as and when data from the work carried out by the Central Groundwater Board was available.

The National Bank for Agriculture & Rural Development (NABARD) approached the Government of India to contribute material for inclusion in its approach for availing World Bank Assistance under NABARD -1 project which was of two years duration-1984 and 1985. The Central Groundwater Board, to whom the matter was referred, examined the methodology suggested by the Over Exploitation Committee in great detail and felt that there was enough room for improvement of the methodologies for estimation of the resource potential under various conditions. It circulated a paper suggesting a revised methodology.

The Groundwater Estimation Committee (G.E.C.) which submitted its report to Government of India in 1984, considering the norms to be followed in evaluation of groundwater resources recommended that the groundwater recharge should be estimated based on groundwater level fluctuation method. The water table fluctuation in an aquifer corresponds to the rainfall of the year of observation. The rainfall recharge estimated should be corrected to the long term normal rainfall for the area as given by Indian Meteorological Department (IMD). To estimate the effects of drought or surplus rainfall, the recharge during monsoon may be estimated for a period of 3 to 5 years and an average taken. Recharge from winter rainfall may also be estimated on the same lines.

Total groundwater resources for water table aquifers is the sum of annual recharge and potential recharge in shallow water table and water logged areas. It also recommended that 15 percent of total groundwater resources be kept for drinking and industrial purposes, for committed base flow and to account for the unrecoverable losses. In case the committed base flow and the domestic and industrial loss is more than 15% of the total groundwater resource, the utilizable resource for irrigation in these areas may be decreased accordingly.

The quantum of groundwater available for development is usually restricted to long term average recharge of the aquifer and is 100% dependable source of supply.

Groundwater being a dynamic and replenishable resource has to be estimated primarily based on normal annual recharge which could be developed by means of suitable groundwater abstraction structures and judiciously harnessed for various purposes. The mean annual groundwater recharge largely depends on the climatic and hydrogeological conditions. The physiogeographical setting of India paradoxically presents the heaviest rainfall in the north eastern part of the country and almost along the same latitude, the existence of a desert on western part with very low rainfall (less than 180 mm). Similarly a long stretch of semi-arid belt which gets a rainfall of 600-800 mm, runs almost north-south of the west central part of peninsular India. The average rainfall of the country being of the order of 1,190 mm and distributed unevenly over the country during the north-east and south-west monsoon and melting of snow from the Himalayas and contribution to the river systems in north, more or less define the availability of water resources in the country. The hydrogeological conditions and groundwater availability and quality is well described in the second edition-(1988) of Hydrogeological Map of India published by Central Groundwater Board, Government of India which may be referred to for more details.

A scientific assessment of the groundwater potential of the country has been made tentatively on the basis of recommendations of Groundwater Estimation Committee (1984) and data being generated by Central Groundwater Board. Total annual replenishable

Groundwater Resources is 45.23 million hectare metre. A part of it (6.93 M.ha m) is kept reserved for the drinking, industrial and other uses. The utilizable groundwater resources for irrigation is 38.30 M.ha. m. The present net yearly draft for irrigation is 10.68 M.ha. meters leaving a balance of Groundwater Resource Potential of 27.32 M.ha.m/yr for further development in the irrigation sector. The State-wise breakup is given in Appendix 5.2.

5.2.4.1 Rock Types

Groundwater is obtained through aquifers which may be composed of consolidated (held firmly together by compaction, cementation and other processes forming granite, sandstone and limestone) or unconsolidated (loose material such as clay, sand and gravel) rocks. Sometimes, they are also called hard and soft rocks respectively. The rock materials must be sufficiently porous (contains a reasonably high proportion of pores or other openings to solid material) and be sufficiently permeable (the openings must be interconnected to permit the travel of water through them). Appendix 5.3 classifies the soils.

Rocks may be classified with respect to their origin into the three main categories of sedimentary rocks, igneous rocks and metamorphic rocks.

Sedimentary rocks are the deposits of material derived from the weathering and erosion of other rocks. Though constituting only about 5 per cent of the earth's crust, they contain an estimated 95 per cent of the available groundwater. Sedimentary rocks may be consolidated or unconsolidated depending upon a number of factors such as the type of parent rock, mode of weathering, means of transport, mode of deposition and the extent to which packing, compaction, and cementation have been taking place. Sand, gravel and mixtures of sand and gravel are among the unconsolidated sedimentary rocks that form aquifers. Granular and unconsolidated, they vary in particle size and in the degree of sorting and rounding of the particles. Consequently, their water-yielding capabilities vary considerably. However, they constitute the best water bearing formations. They are widely distributed through out the world and produce very significant proportions of the water used in many countries. Unconsolidated sedimentary aquifers include marine deposits, alluvial or stream deposits (including deltaic deposits and alluvial fans), glacial drifts and wind-blown deposits such as dune sand and loess (very fine silty deposits). With greater degree of compaction and lithification, the unconsolidated deposits grade into consolidated ones when original porosity is lost and secondary porosity by fractures as in sand stones and solution channels as in lime stones, is introduced. Great variations in the water yielding capabilities of these formations can be expected.

Igneous rocks are those resulting from the cooling and solidification of hot, molten materials called magma which originate at great depths within the earth. When solidification takes place at considerable depth, the rocks are referred to as intrusive or plutonic. While those solidifying at or near the ground surface are called extrusive or volcanic. Plutonic rocks such as granite are usually coarse-textured and non porous and are not considered to be aquifers. However, water has usually been found in crevices and fractures and in the upper, weathered portions of these rocks. Volcanic rocks, because of the relatively rapid cooling taking place at the surface, are usually fine textured and glassy in appearance. Basalt or trap rock, one of the chief rocks of this type, can be highly porous and permeable as a result of interconnected openings called vesicles formed by the development of gas bubbles as the lava (magma flowing at or near the surface) cools. However, the vesicles may be filled by secondary minerals resulting in reduction in porosity and permeability. Basaltic aquifers may also contain water in crevices and broken up or brecciated tops and bottoms of successive layers.

Metamorphic rock is the name given to rocks of all types, igneous or sedimentary, which have been altered by heat and pressure. Examples of these are quartzite or metamorphosed sandstone, slate and mica schist from shale and gneiss from granite. Generally, these form poor aquifers with water obtained only from cracks and fractures. Marble, a metamorphosed limestone, can be a good aquifer when fractured and containing solution channels.

5.2.4.2 Occurence Of Groundwater In Rocks

In the crystalline areas, granite and gneisses are usually the most predominant rocks. Normally, neither the granite nor the gneisses contain inter-connected pore spaces, through which water can move down. But near the surface, the rocky massions are more commonly fractured by intersecting joints and crevices of running dimensions and water passing down through the crevices and joints brings about disintegration and decomposition. This zone of weathered rock usually very porous, is found in many places in crystalline rocks and such zones form valuable receptacles for water which can be tapped by wells. Much of the groundwater reserves lies within a depth of a few hundred metres under structurally favourable conditions. The lateral extent and the thickness of the decomposed zones usually vary from place to place. The thicker the zone of decomposed rock and the larger the area extent of decomposed zone, the larger is the quantity of underground water likely to be met. The best location for high yielding wells in crystalline rock is normally in areas with a thick weathered and water saturated zone and where open cracks or fractures exist below this zone. The deeper the fracture the better the yield, as normally the area of influence increases with increasing depth or draw down in the well.

In some sedimentary basins (like the Ganga sedimentary basin) unconsolidated sediments are saturated to depths of thousand metres or more and contain permeable horizons at intervals throughout the depth. Some of the coarser types like sand and gravel have effective porosity of 10 to 20 percent of the volume of the material and hence can store and yield large quantities of water. The consolidated sedimentary rocks, on the other hand, store and yield less water. Cavernous limestones yield copious supplies but striking them in wells in a matter of chance.

5.2.4.3 Methods For Groundwater Prospecting

(a) Remote Sensing

The search for groundwater i.e. the water beneath the land surface enclosed in pores of the soil, regolith or bedrocks in greatly aided by remote sensing techniques. The clue to groundwater search is the fact that sub-surface geologic elements forming aquifers have almost invariably surface expressions which can be discerned by remote sensing techniques. It should be understood at the beginning that remote sensing techniques complement and supplement the existing techniques of hydrogeological and geophysical techniques and are not a replacement for these techniques.

For convenience, we can divide the aquifers into two groups: (i)Aquifers in alluvial areas, and (ii) Aquifers in hard rock areas.

(i) Aquifers In Alluvial Areas

Most well-sorted sands and gravels are fluvial deposits, either in the form of stream channel deposits and valley fills or as alluvial fans. The remainder are cheniers, beach ridges, beaches, and some well-deposited dunes. Table 5.1 lists the keys to detection of such aquifers on the satellite imagery. Although hydrogeologically significant landforms etc. can be delineated easily on landsat images, more details are visible on aerial photographs. In favorable cases landsat images can be used to select locations for test wells. In other areas locales can be marked for more detailed ground surveys or examination of aerial photographs.

TABLE 5.1

KEYS TO DETECTION OF AQUIFERS IN ALLUVIAL AREAS ON SATELLITE IMAGES

SHAPE OR FORM

Sl. No.	Description
1.	Stream valleys; particularly wide, meandering (low gradient) streams with a large meander wavelength and with broad and only slightly incised valleys
2.	Underfit valleys represented by topographically low, elongate areas with impounded drainage or with a stream meander wavelength smaller than that of the floodplain or terraces
3.	Natural levees (levees themselves may be fine-grained materials)
4.	Meander loops showing location and relative thickness of point bars
5.	Meander Scars in lowland; oxbowlakes arcuate dissection of upland areas
6.	Braided drainage-channel scars
7.	Drainage line offsets; change in drainage pattern; or change in size or frequency of meanders (may be caused by faults and cuestas as well as by changes in lithology)
8.	Arc deltas (coarsest materials) and other deltas
9.	Cheniers; beach ridges; parabolic dunes.
10.	Alluvial fans, coalescing fans; bajadas.
11.	Aligned oblong areas of different natural vegetation representing landlocked bars, spits, dissected beaches, or other coarse and well-drained materials.

PATTERNS

1. Drainage patterns imply lithology and degree of structural control; drainage density (Humid regions) and drainage texture (and regions) imply grain size, compaction and permeability.
2. Snowmelt; if every thing else is equal, anomalous early melting snow and greening of vegetation show areas of ground-water discharge; ice free areas on rivers and lakes.
3. Distinctive types of native vegetation commonly show upstream extensions of drainage patterns, areas of high soil moisture, and landform outlines (Humid regions); abrupt changes in land cover type or land use imply landforms that may be hydrologically significant but do not have a characteristic shape.
4. Elongate lakes, sinuous lakes, and aligned lakes and ponds representing remanents of a former stream valley.
5. Parallel and star dunes.
6. Spliya of parallel linear patterns representing old alluvial fans or landlocked chenier complexes.

TONE

1. Soil type; fine-grained soils commonly are darker than coarse-grained soils.
2. Soil moisture; wet soils are darker than dry soils.
3. Type and species of native vegetation; vegetation is well adapted to type and thickness of soil, drainage characteristics, and seasonal period of saturation of root zone.
4. Land use and land cover: for example, percent bare soil may correlate with drainage density; also for example, native vegetation in lowlands and drainage density; and agriculture on uplands may indicate periodic flooding.
5. Anomalous early or late seasonal growth of vegetation in areas of high soil moisture, as where water table is close to land surface.

TEXTURE

1. Uniform or mixed types of native vegetation; some species and vegetation associations are indicators of wet versus dry sites, thick versus thin soils, or particular mineral compositions of soils.
2. Contrast between sparse vegetation on topographic highs and denser vegetation in low (wetter) areas.
3. Texture contrasts at boundaries of grass, bush and forest cover types; possible boundaries of soil types or moisture conditions.

(ii) Aquifers In Hard Rock Areas

The groundwater abundance depends on rock type, amount and intensity of fracturing. The keys to location of aquifers in hard rock areas is given in Table 5.2. The only space for storage and movement of groundwater in such areas is in fractures enlarged by brecciation, weathering, solution or corrosion. These have surface expressions. in fact weathering, solution, and corrosion operate on land surface as well, in addition to geomorphic processes such as mass wasting and frost wedging. A fracture that is a plane of weakness for

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enlargement by groundwater may be represented on the land surface by topographic depression, a different soil tone, or a vegetation anomaly at land surface.

Many fractures are vertical, in this case, lineaments may represent favourable locations for water wells. Other fractures may be oblique.

TABLE 5.2

KEYS TO DETECTION OF AQUIFERS IN HARD-ROCK AREAS ON SATELLITE IMAGES

OUTCROPPING: ROCK TYPE	
Sl.No.	**Description**
1.	Landforms; topographic relief
2.	Outcrop patterns; banded patterns for sedimentary rocks (outlined by vegetation in some regions); lobate outline for basalt flows; curving patterns for folded beds.
3.	Shape of drainage basins
4.	Drainage patterns, density and texture
5.	Fracture type and symmetry (as implied by lineaments); triangular facets above fault or fault-line scarps and alluvial fans below; discontinuities in bedding patterns, topography or topographic texture; and vegetation types
6.	Relative abundance, shape and distribution of lakes
7.	Tones and textures (difficult to describe; best determined by study of known examples)
8.	Types of native land cover

FOLDS

1. Cuestas and hogbacks; asymmetric ridges and valleys; flatirons on dip slope and irregular topography on back slope; uniform distribution of vegetation on dip slope and vegetation banding parallel to ridge crest on back slope; bajada on dip slope and separate alluvial fans on back slope.

2. Banded outcrop patterns not related to topography; closed to arcuate patterns; U-shaped to V-shaped map patterns of ridges; sedimentary rock patterns with an igneous core

3. Trellis, radial, annular, and centripetal drainage patterns, partly developed patterns of these types superimposed on drainage patterns' of other types.

4. Major deflections in stream channels; changes in meander wavelength or changes from meandering to straight or braided patterns.

5. Asymmetric drainage; channels not centered between drainage divides,

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LINEAMENTS

1. Continous and linear stream channels, valleys, and ridges, discontinous but straight and aligned valleys, draws, swags and gaps.
2. Elongate or aligned lakes, large sinkholes and volcanoes
3. Identical or opposite deflections (such as doglegs) in adjacent stream channels, valleys, or ridges; alignment of nearby tributaries and tributary junctions.
4. Elongate or aligned patterns of native vegetation; thin strips of relatively open (may be rights of way) or dense vegetation.
5. Alignment of dark or light soil tones.

(iii) Limitation

Though remote sensing is a versatile tool, the presence of important indicators of groundwater occurrence can-not always be recognised as such on satellite images especially where morphological expressions of geologic structures are relatively small. The tone differences between rock types are indistinct and variation in the inclination of rock formations minimal.

The limitations of remote sensing in groundwater exploration are:

1. No quantitative estimates of expected yield of wells can be given from remotely sensed data.
2. No depth estimation of aquifers can be made. It may, however, be noted that empirical observations show that length of a lineament (fracture zone) is related to the depth of the lineament.
3. Assessment of quality of water is also not possible. Although the type and vigour of vegetation present on the land surface does provide a clue to the quality of water underneath.
4. In high-relief areas, satellite imagery may not be adequate to locate groundwater controls. Aerial photography may also have to be used.
5. Lateral extent of only those aquifers which are directly exposed or manifest through land covered e.g. shallow aquifers (vegetation), valley fills etc. can be delineated.

(b) Geophysical

Geophysical methods play an important role in any groundwater exploration work. Geophysical methods detect differences or anamolies of physical properties within the earth's crust. Density, magnetism, elasticity and electrical resistivity are the properties most commonly measured. Experience and research have enabled difference in these properties to be interpreted in terms of geologic-structures, rock type and porosity, water content and water quality.

All the four major geophysical methods viz; electric, magnetic, seismic and gravimetic find their use in groundwater exploration in addition to the method of electrical logging which is used extensively to study the physical character, especially porosity and permeability of aquifers penetrated by bore holes. Of the four major methods, electrical and seismic refraction generally find the maximum use in that order.

In unconsolidated and consolidated sediments, the problem from the geophysical point of view may more often be not specifically of locating groundwater as such, but determination of water table and delineation of saline aquifers from potable water zones. On the other hand, in igneous and metamorphic rocks where groundwater generally occurs in fissures and shattered zones or in basins of decomposition, the problem is mainly to locate such structural features which constitute the possible location of the aquifers yielding sufficient quantities of water.

(i) The Electrical Resistivity Method

The electrical resistivity of a rock formation limits the amount of current passing through the formation when an electrical potential is applied. It may be defined as the resistance in ohms between opposite faces of a unit cube of the material. If a material of resistance R has a cross-sectional area A and a length L, then its resistivity $\int$ can be expressed as

$$\int = \frac{RA}{L} \tag{5.1}$$

In the metric system, units of resistivity are ohms-m^2/m or simply ohm-m.

Resistivities of rock formations vary over a wide range, depending upon the material, density, porosity, pore size and shape, water content, quality and temperature.

(ii) Seismic Refraction Method

This method involves the creation of a small shock at the earth's surface either by the impact of a heavy instrument or by exploding a small dynamite charge and measuring the time required for the resulting sound, or shock wave to travel known distances.

Electric logging and other related geophysical tools, such as gamma ray, neutron logging, help to determine where the aquifers are located to reduce the number of failures. Besides, surface operated equipment, such as the seismograph (non-explosive type) are necessary adjuncts for maximum groundwater exploitation.

5.2.5 HYDRAULICS OF GROUNDWATER FLOW

(a) General Hydrologic Equation

Hydrological equilibrium is expressed by the following equation:

$$\sum R = \sum D + \Delta S \tag{5.2}$$

where,

$\sum R$ = summation of flows due to hydrological factors of recharge

$\sum D$ = Summation of flows due to hydrological factors of discharge

ΔS = associated change in storage volume

More specifically the recharge ($\sum R$) is composed of the following:

1. Natural infiltration derived from rainfall and snow melt;
2. Infiltration from surface bodies of water;
3. Underflow;
4. Leakage through confining layers, or water displaced from them by compression; and
5. Water derived from diffusion, charging and water spreading operations.

Conversely, the discharge includes:

1. Evaporation and transpiration;
2. Seepage into surface bodies of water;
3. Underflow;
4. Leakage through confining layers or absorbed by them by reduction of compression; and
5. Water withdrawal through wells and infiltration galleries.

The associated change in storage volumes, ΔS, depends on the properties of soil or rock particularly, the porosity or void ratio, size, shape and compaction of the formation which are all reflected in the specific yield of the formation. ΔS increases with the specific yield.

(b) Rate Of Groundwater Flow

The flow of groundwater through aquifers under the hydraulic conditions of non-turbulent or straight line flow is governed by Darcy's Law which states that head loss due to friction varies directly as velocity of flow and is expressed as:

$$V = KI \qquad (5.3)$$

where

V = velocity of flow in metres per day

I = slope of hydraulic grade line, i.e. slope of the groundwater table or piezometric surface

K = Coeff. of permeability or proportionality constant for water of a given temperature flowing through a given material in metres/day

and

$$Q = ApV \qquad (5.4)$$

where,

Q = Groundwater flow in m^3 per day

A = cross-section of aquifer in m^2

p = porosity of water-bearing medium, it being assumed that the product 'Ap' represents the areas of the channels through which flow is taking place.

This should not be used for flows having Reynolds number greater than 10. This limit is generally reached as water approaches face of wells in coarse-grained sandy soils. In practice no lower limit has been observed even at small hydraulic gradients.

Since 'I' is dimensionless ratio, `K' has the dimension of velocity and in fact is the velocity of flow under a hydraulic gradient of unity.

(c) Conditions Of Groundwater Flow

The groundwater is obtained from aquifers through a "gravity well" or "pressure well" or an "infiltration gallery".

In the "gravity well" the surface of the water outside of and surrounding the well is at atmospheric pressure.

In a "pressure well" the aquifer holds water under pressure greater than atmospheric.

An "infiltration gallery" is a horizontal tunnel or open ditch constructed through the aquifer in a direction nearly normal to the direction of groundwater flow. The tunnel type of gallery is sometimes called a horizontal well.

If a gravity or pressure well is pumped at a constant rate, the drawdown in the well around the area of influence will continue to increase until the rate of replenishment is equal to the rate of pumping i.e. until the equilibrium has been established. The flow into the well until this equilibrium is established is under "Non-equilibrium" conditions. The flow into the well after the equilibrium has been established will be under "Equilibrium" conditions and the flow will be called steady. The steady flow may be "unconfined" or "confined". The flow in a gravity well is "unconfined" and in a pressure well is "confined".

(d) Formulae For Flow Under Equilibrium Conditions

Assumptions

- Direction of the flow of groundwater is horizontal;
- The flow is at a constant rate and in a radial direction towards the centre of the well; and
- The well penetrates to the bottom of the aquifer and is in equilibrium condition unless it is specified to the contrary.

(i) Flow Into A Gravity Well Under Equilibrium Conditions (Refer Fig 5.1)

The flow into a gravity well under equilibrium conditions is given by the formula:

$$Q = \frac{1.36K\left(H^2 - h^2\right)}{Log\frac{(R)}{(r)}} \tag{5.5}$$

Where,

Q = Rate of flow into well in m^3/d

K = Permeability constant in m/d

H = Depth of the water in the well before pumping in m

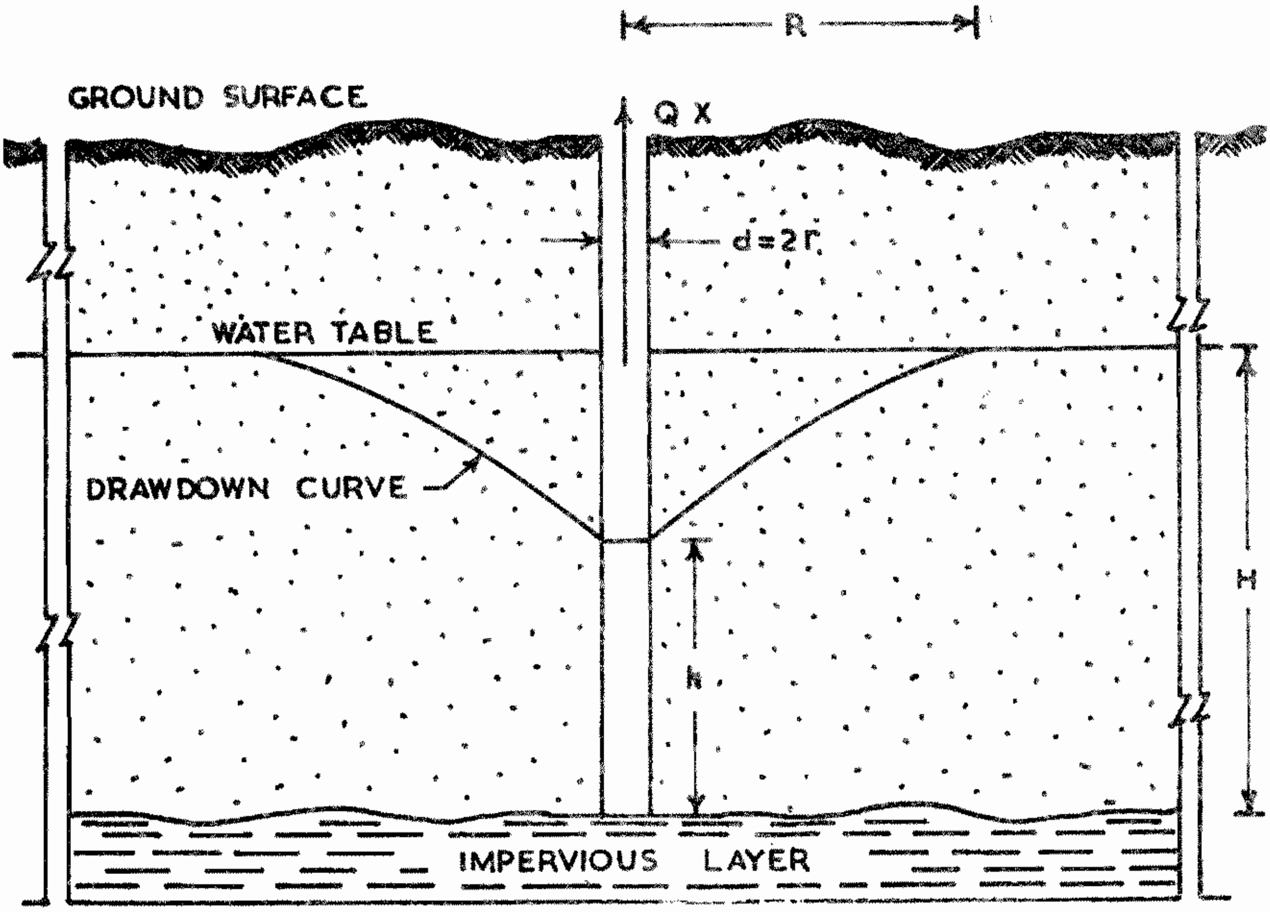

FIG5.1: GRAVITY WELL UNDER EQUILIBRIUM CONDITIONS

h = Depth of water in the well after pumping = (H – drawdown) in m

R = Radius of influence in m

r = Radius of well in m

(ii) Flow into a pressure well under Equilibrium Conditions. (Refer fig 5.2)

Flow into a pressure well under equilibrium conditions is given by the formula:

$$Q = \frac{2.72 K\, m(H-h)}{Log \frac{(R)}{r}} \qquad (5.6)$$

Where,

Q = rate of flow into well in m^3/d

K = permeability constant in m/d

m = thickness of the confined aquifer in m

H = depth of water in the well before pumping in m

h = depth of water in the well after pumping in m

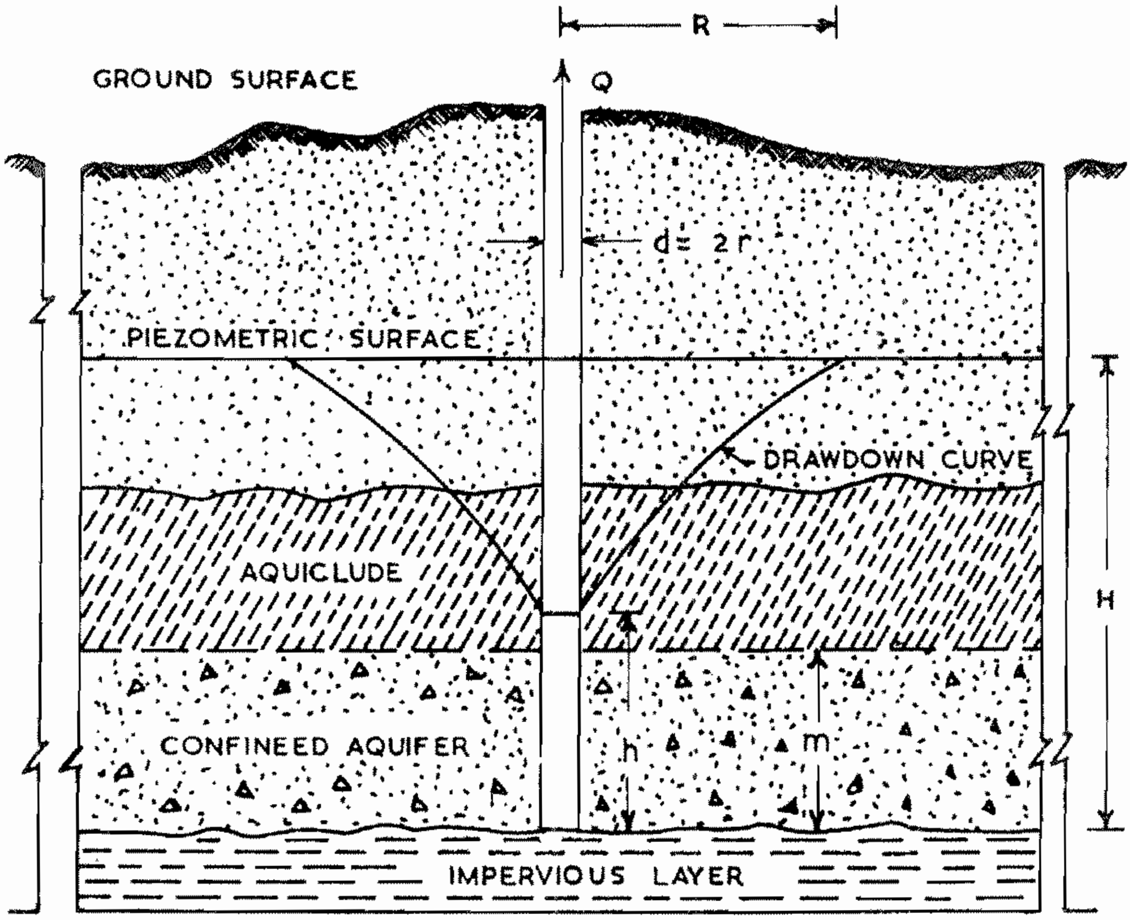

FIG 5.2 : PRESSURE WELL UNDER EQUILIBRIUM CONDITIONS

R = radius of influence in m

r = radius of well in m.

(iii) Flow Into An Infiltration Gallery Under Equilibrium Conditions (Refer Fig 5.3)

The expression for the rate of flow into an infiltration gallery is given by the formula:

$$Q = KL\frac{H^2 - h^2}{2R} \qquad (5.7)$$

where,

Q = rate of flow in m^3/d

K = permeability constant in m/d

L = length of the gallery in m

H = initial depth of water level in m

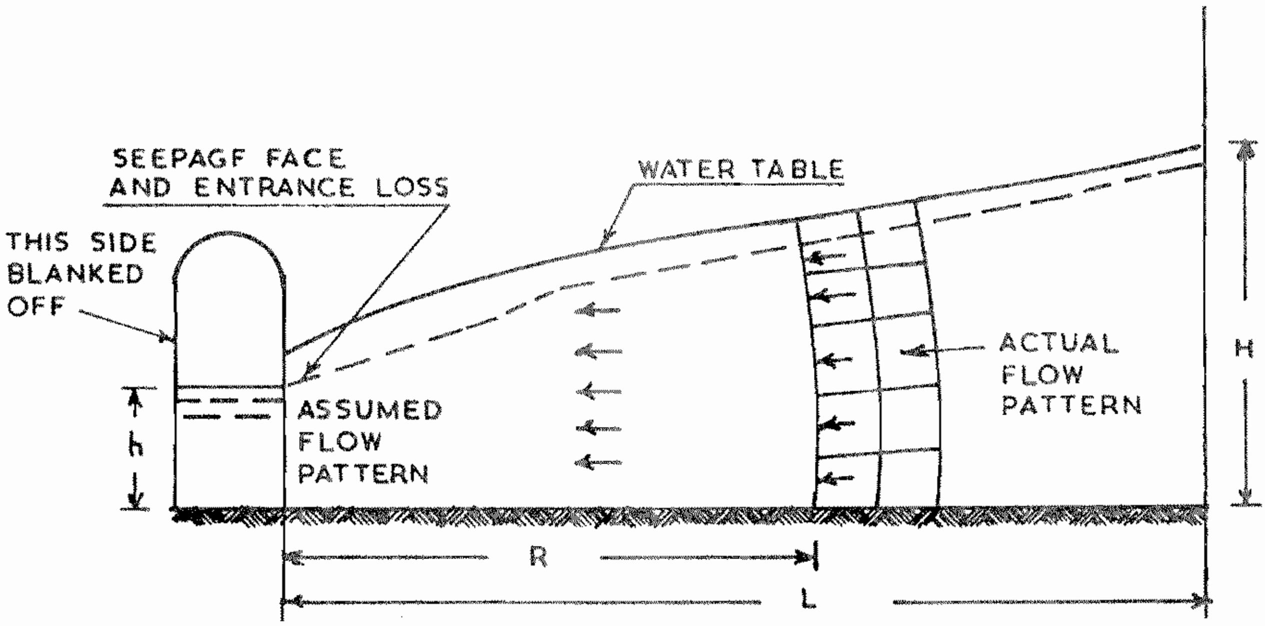

FIG5.3: INFILTRATION GALLERY UNDER EQUILIBRIUM CONDITIONS

h = final depth of water level in m

R = radius of influence in m

(iv) Partial Penetration Of An Aquifer By A Well

If the gravity well does not penetrate to the bottom of the aquifer, the expression (5.5) is not applicable. The flow into a partially penetrating gravity well is given by the expression:

$$Q = \frac{1.36K\left(H^2 - h^2\right)}{Log\frac{R}{r}}\left[p\left(1 + 7\sqrt{\frac{r}{H_1}}\cos\frac{\pi p}{2}\right)\right] \qquad (5.8)$$

where,

R = radius of influence in m

r = radius of well in m

H = thickness of aquifer in m

H_1 = thickness of aquifer penetrated in m

p = H_1/H

$\pi p/2$ = angular perimeter in radians

(e) Flow Into Wells Under Non-Equilibrium Conditions Or Unsteady Flow Conditions

The rate of flow under non-equilibrium conditions is given by the expression:

$$p = \frac{114.60Q}{T} F(u) \qquad (5.9)$$

$$u = \frac{250S}{T} \frac{x^2}{t} \qquad (5.10)$$

where,

F(u) = well function of u whose values could be found out from the Table at Appx 5.4 or the type curve at Appendix-5.5 for different values of u

Q = uniform rate of pumping in lpm

S = storage coefficient

t = time during which the well has been pumped (expressed in days)

T = coefficient of transmissibility in lpd per metre width

x = distance from the well in m

p = draw-down in m.

Q, S and T are considered to be constant.

Then, $\frac{Q \times 114.6}{T} = C_1$ $\frac{T}{250S} = C_2$ are also constants

The equations (5.9) and (5.10) above be written as:

$$\log C_1 = \log p - \log F(u) \qquad (5.11)$$

and

$$\log C_2 = \log \frac{x^2}{t} - \log(u) \qquad (5.12)$$

The values of C_1 and C_2 can be found out from the field observations. Drawdowns in the observation wells (x metres away from the central well) are observed at different intervals, when the central well is pumped out at uniform rate.

The measured values of 'p' are plotted as ordinates against measured values of x^2/t as abscissae on a log-log paper and a curve drawn as at Appendix-5.5

Because of the similarity of expressions (5.11) and (5.12) and the methods of plotting this curve and the type curve (plotted with values of F(u) as ordinates against values of u as abscissae on a log-log paper) there is a corresponding point on the type curve which is displaced vertically by a fixed distance representing log C_1 and horizontally by a fixed amount representing log C_2. Therefore, a fixed amount of vertical and horizontal shift will bring the two curves into coincidence.

If transparent paper is used for the plot of the observed data and it is placed over the type curve, to be shifted horizontally and vertically until a best fit of the plotted points to the type curve is obtained, then any matching point will identify the values of F (u) and u that correspond to the values of p and x^2/t by which equations (5.9) and (5.10) can be solved for T and S.

Though these equations apply rigidly only when (i) the aquifer is homogenous; (ii) the aquifer is infinite in areal extent; (iii) the well penetrates the entire thickness of the aquifer (iv) the coefficients of transmissibility and storage are constant at all times and places; and (v) water is released from storage as soon as the cone of depression develops, they could be used in the field conditions generally encountered.

This method is very useful for long term prediction of groundwater yield and regional planning of groundwater extraction (Appendix-5.6)

5.2.6 DEVELOPMENT OF SUBSURFACE SOURCES

The subsurface sources include springs, wells and galleries. The wells may be shallow or deep. Shallow wells may be of the dug well type, sunk or built, of the bored type or of the driven type. They are of utility in abstracting limited quantity of water from shallow pervious layers, overlying the first impermeable layer.

Deep wells are wells taken into pervious layers below the first impermeable stratum. They can be of the sunk well type or the bored or drilled type. They are of utility in abstracting comparatively larger supplies from different pervious layers below the first impervious layer. Because of the longer travel of groundwater to reach pervious layers below the top impermeable layers, deep wells yield a safer supply than shallow wells.

5.2.6.1 Classification Of Wells

The wells are classified according to construction as follows:

(a) dug wells;

(b) sunk wells;

(c) driven wells; and

(d) bored wells.

(a) Dug Wells

Dug well of the built type has restricted application in semi-permeable hard formations. The depth and diameter are decided with reference to the area of seepage to be exposed for intercepting the required yield from the sub-soil layers. Unsafe quality of water may result if care is not taken in the well construction. It is necessary to provide a water-tight steining upto a few metres below the vertical zone of pollution which usually extends 3 to 5 m or more below natural ground surface.

The steining should extend well above the ground surface and a water-tight cover provided with water-tight manholes.

The bottom of the well should be at a level sufficiently below the lowest probable summer water table allowing also for an optimum drawdown when water is drawn from the well. Adequate provision should also be made to take care of interference by other pumping wells. To facilitate infiltration into the well, either the steining is constructed in dry masonary or, weepholes are left in the steining at suitable intervals. It is usual to insert cut lengths of pipes in the steining with the outer end covered with a wire gauze and shrouded with gravel to arrest ingress of fine material..

(b) Sunk Wells

Sunk wells depend for their success on the water bearing formations which should be of adequate extent and porosity. The sunk well is only the inter-position of a masonary barrel into such a deposit so as to intercept, as large a quantity of water, as is possible.

(i) Size vs Yield

The yield of any form of a well is dependent on the rate of flow of the groundwater and the area made tributary by the depression of the water level in the well rather than on the size or form of construction. As is well known, the effect of size alone is very small and an increase in the yield of large wells will not commensurate with the increase in size.

The large well has an advantage over the small well in its storage capacity and facility for placement of pump sets economically. Trouble may often be experienced in the small wells through clogging and the entrance of fine sand. This is largely avoided in the large well as the entrance velocity of the water is correspondingly small. Opportunity is also given for the settling of fine material.

Wells for water supply are constructed of diameters normally ranging from 3 m and above. As the cost of a well increases with increase in diameter, more rapidly than does the yield, any large diameter should be adopted after a careful consideration.

(ii) Construction Methods

The minimum depth of a well is determined by the depth necessary to reach and penetrate, for an optimum distance, the water bearing stratum allowing a margin for dry seasons for storage and for such draw-down as may be necessary to secure the required yield. The method of construction employed depends on the size and depth of the well, characteristics of material to be excavated and quantity of water to be encountered. The procedure generally adopted is to have open excavation upto the sub-soil water table and thereafter to commence sinking the steining built in convenient heights, over a wooden or R.C.C. curb with a cutting edge at the bottom, the curb projecting about 4 cms beyond the outside face of the steining to facilitate easy sinking. Mild steel holding down rods are run from the bottom of the curb through the steining spaced about 2 metres circumferentially, with horizontal ties in steel or of concrete rings, spaced about 2 metres vertically. The material from inside the well is dredged and removed either mechanically or by manual labour using divers with diving equipment.

(iii) Measures to Increase Yield

Dewatering the well to an optimum extent is resorted to, during the sinking operations. The constructions supervision should ensure uniform vertical sinking of the steining. Entry for the infiltration water into the well is usually at the bottom below the curb. In order to reduce the velocity of entry and to abstract a larger yield for the same draw-down, weepholes in the steining at suitable intervals, horizontally and vertically, would be useful. These could be of cut length of pipes 75 or 100 mm dia, built into the steining, with wire gauze at the outer end, which will be kept flush with the outside face of the steining. Such weepholes would draw water from the pervious layers extending over the depth of the steining, apart from the influx at the bottom. In the initial stages of pumping and during the training of the yield, the fines from water bearing strata round each weephole would be drawn out facilitating a larger influx through each weephole under normal pumping.

(iv) Porous Plugs

In the case of infiltration wells sunk in sandy soils, a porous plug in the form of a reverse filter is placed at the bottom of the well after the initial training of the yield from such well, to facilitate the abstraction of a greater yield, as the plug would permit increased velocities of entry without sand blows. The graded plug is usually an inverted filter comprising of coarse sand and broken metal of appropriate sizes to suit the texture of the sub-soil layers in the aquifer immediately below the well-curb. The depth and the composition of the porous plug will be designed to maintain the natural sandy layer immediately below the curb level undisturbed during pumping.

Radial strainer pipes are driven horizontally from the interior of sunk wells into the water bearing pervious strata as a measure of increasing the yield for the same draw-down. The arrangement in effect enlarges the zone of influence of the well. Further details are given under Radial Collector Wells in 5.2.6.3.

(v) Protection Measures

All wells should be covered so as to prevent direct pollution of water. Where infiltration wells are sunk in the bed of streams liable to carry floods, the top of the well should be kept 0.5 to 1 m above the maximum flood level if it is not very high. If the well top is kept below flood level, provision should be made for ventilating the well with a porous concrete ring placed below the cover slab or with holes in the cover slab filled with a graded filter material.

(c) *Driven Wells*

(i) Construction

The shallow tube well, also called a driven well, is sunk in various ways depending upon its size, depth of well and nature of material encountered. The closed end of a driven well comprises a tube of 40 to 100 mm in diameter, closed and pointed at one end and perforated for some distance therefrom. The tube thus prepared is driven into the ground by a wooden block until it penetrates the water bearing stratum. The upper end is then connected to a pump and the well is complete. Where the material penetrated is sand, the perforated portion is covered with wire gauze of suitable size depending upon the fineness of the sand. To prevent injury to the gauze and closing of the perforations, the head of the shoe is usually made larger than the tube or the gauze may be covered by a perforated jacket.

Such a driven well is adopted for use in soft ground or sand upto a depth of about 25 m and in places where the water is thinly distributed. On account of the ease with which it can be driven, pulled up and redriven, it is especially useful in prospecting at shallow depths and for temporary supplies. It is useful as a community water standpost in rural area.

(ii) Protection Measures

Special care is necessary during construction to avoid surface pollution reaching the sub-soil water level directly, through any passage between the pipe and the soil. The usual precaution is to have the perforations confined to the lower depths of the aquifer with the plain tubing extending over the top few metres of the soil. In addition, a water-tight concrete platform with a drain should be provided above ground level, in order to deflect any surface pollution away from the pipe.

(d) Bored Wells

(i) General

Bored wells are tubular wells drilled into permeable layers to facilitate abstraction of groundwater through suitable strainers inserted into the well extending over the required range or ranges of the-water bearing strata. There are a variety of methods for drilling such wells through different soils and for providing suitable strainers with a gravel shrouding where necessary.

Bored wells useful for obtaining water from shallow as well as deep aquifers are constructed employing open end tubes, which are sunk by removing the material from the interior, by different methods. The deeper strata are usually more uniform and extensive than strata near the surface, so that in regions already explored, deep wells can be sunk with far more certainty of success than is usually the case with shallow wells. Methods of sinking deep wells are in many respects different from those already described and matters of spacing, pipe friction, arrangement of connections, etc., are much more important than in the shallow wells.

For bored wells, the hydraulic rotary method and the percussion method of drilling such wells through hard soils are popular. For soft soils, the hydraulic jet method, the reverse rotary recirculation method and the sludger method are commonly used.

(ii) Direct Rotary Method

With the hydraulic direct rotary method, drilling is accomplished by rotating suitable tools that cut, chip and abrade the rock formations into small particles. The equipment used consists of a derrick, suitable cables and reels for handling the tools and lowering the casing into the hole, a rotary table for rotating the drill pipe and bit, pumps for handling mud laden fluid and a suitable source of power. As the drill bit attached to the lower end of the drill pipe is rotated, circulating mud is pumped down the drill pipe, out through opening in the bit and up the surface through the space between the drill pipe and the walls of the hole. The mudladen fluid removes the drill cuttings from the hole and also prevents caving by plastering and supporting the formations that have been penetrated. For soft and moderately hard materials a drilling tool shaped like the tail of a fish, the 'fishtail bit' is used. In hard rock a 'rock bit' or 'roller bit' is substituted. This bit has a series of toothed cutting wheels that revolve as the drill pipe is rotated.

Water wells drilled by the hydraulic rotary method generally are cased after reaching the required depth, the complete string of casing being set in one continuous operation. If the water-bearing formation lies so deep that it probably cannot be reached by a hole of uniform diameter, the hole is started one or more sizes larger than the size desired through the water-bearing formation. Separate strings of casing are used as required through the separate sections of the hole. If the formation is so well consolidated that the hole will remain open without casing, a well may be finished with one string of casing and a well screen.

This method is most suitable for drilling deep holes in unconsolidated formations. It is unsuitable for drilling in boulders and hard rocks due to slow progress and high cost of bits. It is also unsuitable for drilling in slanted and fissured formations and serious lost circulation zones. Mud drilling is harmful in low pressure formations due to mud invasion. The hydraulic rotary drilling generally requires large quantity of water which may have to be brought from long distances, if not locally available. Because of adding large quantities of water and sand or clay to the drill cuttings, the hydraulic rotary method is less suitable for obtaining accurate logs of the strata encountered.

A recent advance is the use of organic drilling fluids instead of inorganic and permanently gelatinous clays such as bentonite. The organics are almost completely self-destructive within a period of few days which means no drilling muds are left in the pores of the aquifer and, therefore, almost always higher yields are obtained with accompanying lesser development expenditures. In addition to higher specific capacities, cleaner holes (more cuttings settle on the surface equipment) and faster drilling rates also result.

(iii) Percussion Method

In the percussion method of drilling, the hole is bored by the percussion and cutting action of a drilling bit that is alternately raised and dropped. The drill bit, a clublike, chisel-edge tool, breaks the formation into small fragment; and the reciprocating motion of the drilling tools mixes the loosened material into a sludge that is removed from the hole at intervals by a bailer or a sand pump. The drilling tools are operated by suitable machinery; which is usually of the portable type mounted on a truck or a trailer so that it can be moved readily from job to job. This method is best suited for drilling on boulders, slanted and fissured formations and lost circulation zones. Rate of drilling in alluvial formations, particularly those having clay or sticky shale strata, is much lower as compared to direct or reverse rotary methods. Percussion drilling in hard rock is a slow process and is being gradually replaced by pneumatic rotary drilling because of economy and speed of completion regardless of the higher initial cost.

'Pneumatic Drilling'

Pneumatic drilling with top-hammer and eccentric bit and pneumatic drilling with down the-hole hammer are the two principal methods available for drilling in consolidated (hard rock) formations:

(a) Top Hammer and Eccentric Bit

This rapidly expanding drilling method is most valuable when drilling in hard rocks covered with difficult over-burden. The overburden, even if it is of the collapsible type, presents no problem as the method is based on the simultaneous drilling and inserting of casing tubes down to and even into the bed rock. The principle of the drilling method is as follows:

A compressed air powered rock drill with a separate rotation coupled to it, works at the top of a drill string. At the bottom of the string is a tungsten carbide set drill bit, the pilot bit, to which the impact and rotation is transmitted. Immediately above this bit is a reamer with a tungsten carbide set cutting edge. With normal rotation to the left, the reamer will swing out eccentrically and cut a hole which is of larger diameter than the pilot bit, allowing the. casing tubes which enclose the drill string to enter into the hole at the same pace as the drilling proceeds. Since no external obstructions can be tolerated on the string of casing tubes, they will have to be flush-jointed with male and female threads or, preferably, by welding. The cuttings are flushed up between the drill string and the casing tubes. To make this effective and also prevent the formation of large amounts of dust, foam-producing chemicals are introduced into the flushing air.

(b) Down-the-Hole Hammer

This drilling method, called DTH for short; permits rapid and effective drilling in rock and through over-burden which is not susceptible to collapse. In this method the impact mechanism blows directly on the drill bit and accompanies it down into the hole. Compressed air for the impact mechanism is supplied through drill tubes which are jointed as required as the drilling advances. The same air is, after it has passed the hammer, made use of for flushing. The necessary rotation is supplied from a rotation unit connected to the upper drill tube.

As the drill tubes are not required to transmit the violent impact energy of the hammer, they can be manufactured with large diameter and still be relatively thin walled. This gives the method better flushing characteristics than conventional top hammer drilling. Theoretically, the rate of penetration is independent of the hole depth with the DTH method no water is required during drilling. The equipment is also cheaper and lighter as a much smaller compression is required than for top hammer drilling.

(iv) Hydraulic Jet Method

This is the best and most efficient method for small diameter bores in soft soils. Water is pumped into the boring pipe fitted with a cutter at the bottom and escapes out through the annular space between the pipe and the bored hole. The pipe is rotated manually with the aid of pipe wrenches with a steady downward pressure. The soil under the cutter gets softened and loose by the action of the jet of water and is washed with it as the cutter proceeds, down with the weight of the pipe. Additional lengths of pipe are added till the required depth is reached. The wash water emanating from the annular space indicates the type of soil that is being encountered by the cutter. When the desired depth is reached, the pipes are withdrawn and the well tube with the strainer is lowered by the same process using a plug cutter with the plug removed instead of the ordinary steel cutter. When the pipe is in position, the plug is dropped down to seal the bottom. Then the tube well is cleaned by forcing water through a 20 mm pipe lowered right to the bottom of the tube well. Then it is withdrawn and the pump fitted on top.

For bigger diameter tube wells, casing pipes are used and mechanically driven pump set is used for jetting. The tube well pipe with the strainer is lowered into the casing pipe and the outer casing withdrawn. Generally compressed air is used for developing the well. To economise the use of water during the operation, the wash water carrying from the bore is led to a sump wherefrom the water is again drawn for being forced into the bore.

(v) Reverse Rotary Method

In this method the water is pumped out of the bore through the pipe and fed back into the annular space between the bore and the central pipe. No casing is required in this method which is used only in clayey soils with little or no sand. This method is suitable for large diameter bores upto a depth of 150 m. The cutting pipe is clamped to a turn-table which rotates slowly operating the cutter. The water pumped out of the tube contains the washings and is led to a series of sumps for effective sedimentation of the solid particles before the water is put back to flow into the bore. Bentonite or some clayey material which can adhere to the sides of the bore firmly, is used from time to time.

After the required depth is reached, the pipe with the cutter is taken out of the bore and the well pipe with the strainer is then lowered into the hole. The annular space between the bore and the well screen is then shrouded with pea gravel.

(vi) Sludger Method

In this method the boring pipe with the cutter attached is raised and lowered by lever action and the bore filled with water from a sump nearby. When the boring has proceeded a few metres down, the pumping out of the water from the inside of the bore pipe is carried out in an improvised manner by the operator closing the top end of the pipe during the upward stroke and releasing it during the downward stroke. This method when done with quick up and down strokes enables the washings from the bore pipe to come out of the pipe. The bore is always kept full with the water from the sump. Bentonite or some clayey material is added sometimes. This method is suitable for depths upto about 50 metres. When the proper depth is reached, the bore pipe is taken out and the well tube with the strainer is lowered as in other methods. This method is suitable for small diameter wells in soft soils and medium hard soils. This is particularly applicable for use in areas not easily accessible where labour is available for the unskilled complement.

(vii) Casing of Wells

Wells in soft soils must be cased throughout. When bored in rock, it is necessary to case the well atleast through the soft upper strata to prevent caving. Casing is also desirable for the purpose of excluding surface water and it should extend well into the solid stratum below. Where artesian conditions exist and the water will eventually stand higher in the well than the adjacent groundwater, the casing must extend into and make a tight joint with the impervious stratum; otherwise water will escape into the ground above.

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If two or more water bearing strata are encountered, the water pressures in different strata are likely to be different, that from the lower usually being the greater. Where different pressures thus exist, it is only possible to determine their amount by separately testing each stratum as reached, the others being cased off. This operation is an essential part of the boring and should be carefully performed. Important differences in quality and yields are discovered in this way.

When quality stratification exists, which may be ascertained from geophysical logs or drill-stem tests, blank casings should be provided against zones containing undesirable quality of water and the annular space between the casing and hole wall should be sealed with cement grout or packers. This will ensure that the fresh water aquifers are not contaminated by leakage.

Large casing is generally made of welded or riveted steel pipe. For smaller sizes of pipes which are to be driven, the standard wrought iron pipe is ordinarily used, but for heavy driving extra strong pipe is necessary. The life of good heavy pipes is ordinarily long, but they are liable to rapid corrosion due to the presence of excess amount of carbonic acid. The use of rust resisting alloys would be economical in such special cases. Non-reinforced plastic, usually PVC, casing upto 100 mm dia and reinforced plastic casing and fibre glass for longer dia upto 400 mm are coming into vogue.

(viii) Well Strainer and Gravel Pack

In providing the strainer arrangement whereby water is admitted and sand or gravel excluded, it is desirable to make the openings of the strainer as large as practicable in order to reduce friction, while at the same time preventing entrance of any considerable amount of sand.

The openings in well strainers are constructed in such a fashion as to keep unwanted sand out of the well while admitting water with the least possible friction. In fine uniform strata, the openings must be small enough to prevent the entrance of the constituent grains. Where the aquifer consists of particles that vary widely in size, however, the capacity of the well is improved by using strainer openings through which the finer particles are pulled into the well, while the coarser ones are left behind with increased void space. A graded filter is thereby created around, with the aid of back-flushing operations or by high rates of pumping.

The selection of the well screen is important; on it depends the capacity and the life of the well. The size of the openings may be selected, after a study of the mechanical analysis of the aquifer, to permit the passage of all fine particles representing a certain percentage, by weight, of the water-bearing material. It is common practice to use openings that will pass about 70 per cent or more of the sand grains in the natural aquifer whose uniformity coefficient should range between 2 to 2.5. For soils with a uniformity coefficient less than 1.5, gravel shroud should be used. The shape of the openings should be such as to prevent clogging and bridging, which can be diminished by V-shaped openings with the larger end towards the inside of the well. Long, narrow, horizontal or vertical slotted pipes are preferred for large diameters. The openings should be placed as close together as the strength of the screen will permit.

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The total area of the openings in a screen should be such as to maintain an entrance velocity less than necessary to carry the finest particle of sand that is to be excluded by the screen. In general, it should be less than about 4 to 6 cm/s. with gravel shrouding. It is generally desirable that the length of the screen is made slightly less than the thickness of the aquifer penetrated and placed centrally in respect of the aquifer. The length, diameter and total area are inter-dependent dimensions that must be adjusted to give the desired entrance velocity. Some margin of safety in screen size is desirable to allow for incrustation and clogging and to prolong the life of the screen.

Where the water-bearing sand stratum contains little or no gravel, it is very advantageous to insert a layer of fine gravel between the strainer and the sand strata, thus permitting the use of larger orifices in the strainer and greatly decreasing ground friction. The gravel wall so provided may vary in the thickness to suit the size and depth of the boring. It may vary from 10 cm to 25 cm, but it is usually 10 cm. The size of the gravel to be provided would be decided by the particle size distribution in the layer penetrated and the slot size in the well screens proposed to be adopted. Since screen sizes can now be custom-tailored to fit any grading of desired gravel, there is a shift from the former multiple (concentrically placed) gravel packs to single ones.

Well effectiveness and performance may be adversely affected if the gravel pack ratio, that is, the mean size of gravel divided by mean size of formation material, exceeds 5. Beyond this limit, wells may require longer time for development or, if the ratio is excessive, they may turn out to be sand pumpers ultimately resulting in failure. The gravel size should be related to the size of the formation materials in the finest section of the aquifer materials against which screen is provided.

In gravel packed wells the screen size should be related to the gravel in about the same manner as it is related to the aquifer materials in a non-gravel pack (for natural pack) well, i.e. it should correspond to the size that separates 90 per cent of coarser fractions of gravel.

(a) Bimetal Strainers

For small driven tube wells generally of 3 cm to 5 cm dia, the strainers are generally of bimetal-sometimes called jacketed strainers. It consists of a galvanised iron pipe with about 300 rectangular slots of 1 cm x 1 1/2 cm in a standard length of pipe of 1.8 m, having an area of opening of about 17% covered with a brass wiremesh of 24 or 32 mesh which again is enveloped by a perforated brass metal sheet of 26 gauge, having about 2 to 3 holes of 3 mm dia per cm^2 or an area of opening of about 18.5%. The effective area of opening resulting is roughly 5 to 7%. These are available upto 150 mm dia. Gravel shrouding is not essential for this type of strainer.

(b) Monometal strainers

The monometal type is of a single metal with diameters in the range 30 mm to 300 mm usually fabricated from brass sheets 2 or 3 mm thick. These have V-shaped slots of varying sizes to permit a proper selection of strainer to suit the sand size in the aquifer. Slots 2.5 mm wide and 30 mm long are usual.

A thin gravel shroud (50 to 75 mm) is also provided in some cases.

Sometimes the brass monometal strainer is strengthened with an inner G.I. slotted pipe for greater rigidity and longer service.

(c) Slotted Pipe Strainers

Galvanised iron or brass pipes having bigger slots about 3 mm in width and 750 mm in length are provided in-conjunction with pea gravel shroud, 100 mm to 250 mm thick. The slots are V-shaped with the smaller opening on the outside. The gravel shroud makes it possible to use strainers with large sized slots and abstract a larger yield than is otherwise possible. The slots are preferably to be kept horizontal with unslotted strips left between successive rows or columns of slots.

The advantage with this type of strainer over the others is that there is less damage by galvanic action or chockage due to incrustation.

(d) New Type of Strainers

Strainers of different makes are marketed claiming specific advantage for each. One such is a slotted mild steel pipe core, coated with special anti-corrosive plastic paint and provided with an enveloping graded sand shroud bonded with heat resistant, water repellant plastic.

Strainers made of special alloys such as stainless steel (types 304 and 316), monel metal, red brass etc., are also used where indicated and if available..

High density polythene or P.V.C. and metal combined strainers are gaining popularity in view of their non-choking, non-corroding and non-incrusting properties which give long and uninterrupted service.

5.2.6.2 Infiltration Galleries

(a) Wells Vs. Galleries

Infiltration galleries offer an improvement over a system of wells, in that a gallery laid at an optimum depth in a shallow aquifer serves to abstract the sub-soil flow along its entire length, with a comparatively lower head of depression. Moreover, in the case of a multiple system of infiltration wells, the frictional losses contributed by the several connecting pipes diminish the draw-down in the farther wells to that extent and the utility of a well becomes less and less in the total grid. All the same, wells have to be located with a minimum distance in between each pair, so as to avoid mutual interference under normal pumping. It also becomes uneconomical to lay long lengths of connecting pipes in river beds at depths where constructional difficulties add to the cost of their laying and jointing against high sub-soil water level conditions. These pipes are themselves vulnerable to damages from undue scour during high floods if adequate safeguards are not provided. The pipes are liable to break at their junction with the well steining, should there be a subsidence of the well structure under floods.

(b) General Layout

Essentially, a gallery is a porous barrel inserted within the permeable layer, either axially along or across the groundwater flow. A collecting well at the shore end of the gallery serves as the sump from where the infiltrated supply is pumped out. The collecting well is the point at which the maximum head of depression is imposed under pumping operation, the depression head being diffused throughout the length of the gallery to induce flow from the farthest reach.

The exact alignment of a gallery must be decided with reference to the actual texture of the sub-soil layers, after necessary prior investigations to map out the entire sub-soil. A gallery could be laid axially along a river or across a river. In both the cases, the head of depression induced is the factor influencing the abstraction of the sub-surface flow into the gallery liner; and the zone of influence exerted along the entire length of the gallery line will have the same variations irrespective of the direction of the gallery. A cross gallery would have the advantage of the same potential head in the sub-soil water level along its entire length, whereas the axial gallery will have a varying potential in the sub-soil water level, from a maximum at the farthest end upstream, to a minimum at its other end down stream. But a cross gallery has a distinct advantage when it is used as an instrument for abstracting the maximum available sub-surface flow, in the river-bed if this was possible, in which case the cross gallery becomes virtually a sub-surface barrage.

(c) Structure of a Gallery

The normal cross section of a gallery comprises loosely jointed or porous pipe or rows of pipes, enveloped by filter media of graded sizes, making up a total depth of about 21/2 m and a width of 2 1/2 m or above, depending on the number of pipes used for collection of the infiltrated water. The enveloping media round the collecting pipe functions more as a graded plug whereby water from the sub-surface sandy layers of the river bed is abstracted without drawing in fine particles at the same time. Total reliance need not, therefore, be placed on the filter media of the gallery as such, for effecting the full scale purification of the inflow.

The gallery has necessarily to be located sufficiently below the lowest groundwater level in the aquifer, under optimum conditions of pumping during adverse seasons. The gallery should, of course, be located lower than the scouring zone of the river bed under high floods, so that the top-most sand layer of the gallery media remains undisturbed at all times. The natural permeable layers of the aquifer over the gallery media serve as the initial filtering layers for the sub-soil flow and also safeguards the gallery from scouring effects.

The disposition of the filter media around the porous collecting pipe and the particle size distribution for each layer of the media are of importance. If the invert of the gallery is taken up to an impervious layer, there is no need to provide any filter media underneath the collecting pipe except perhaps a nominal layer of coarse aggregate to separate the pipe from the soil immediately below and to ensure a uniform bedding for the pipe. The galleries consist of either a single or double row of stoneware or concrete pipes loose jointed with cement lock filters. Perforated PVC pipes can also be used. The pipes are laid usually horizontally or to a gradient if aligned in the direction of flow. The coarse aggregate envelope in the pipe material is in three layers, followed by coarse and medium sand layers, as detailed below:

Filtering medium near pipe line - 38 mm broken stone.

2nd layer - 38 to 19 mm broken stone.

3rd layer - 12 to 6 mm broken stone.

4th layer	-	Coarse sand passing through a sieve of 3.35 mm size and retained on a sieve 1.70 mm size.
5th layer	-	Fine sand retained on 70 micron sieve and passing through 1.70 mm sieve.

In the older practice, the pipe was surrounded on three sides by two or three layers of the coarse media, while the finer layers of the coarse media and sand formed further horizontal layers on the top alone. This is not quite rational, as the entry into the gallery is also through the sides and a repetition of all the layers of the enveloping media on both sides of the collecting pipes is also necessary.

The particle size distribution between each successive layer should preferably be based on a multiple of four. Precast perforated concrete barrels are also used as collecting pipes with the enveloping media on the three sides.

Filter media round the gallery pipe-line may be said to function like graded plugs to infiltration well bottoms. In the latter case, the plug has to be designed to suit the actual particle sizes of the sub-soil layers on which the well is founded, in order to arrest the entry of fine particles into the well under continuous pumping operation and to induce a greater head of depression than is otherwise possible without the plug. It eventually serves to train the yield into the well and increase it to an optimum quantity under actual pumping operations. Likewise, the enveloping media round the gallery pipe line is best designed to suit the actual layers of the sub-soil which will immediately surround the gallery media. Preliminary boring operations and sieve analysis of samples could help to decide on the different variations in such sub-soils, so that if a gallery system was on an extensive scale, the gallery media could be designed suitably for the different reaches, in order to obtain maximum yield under optimum heads of depression.

(d) Constructional Features

The constructional features during the execution of such galleries are of importance. Trenches are dug with adequate shoring or piling facilities right down to the required level decided upon for the invert of the gallery, which would normally be placed several metres below the sub-soil water level, a greater depth indicating a greater potential for the yield from the gallery. The gallery can be laid under water, if dewatering the trench completely for the purpose is not feasible or economical. Manholes should be provided at intervals of about 75 m for inspection. These are sunk into the bed before the gallery is laid and the floor of these wells are taken a little below the invert level of the gallery pipe. The pipes are covered with R.C.C. slab with water-tight manhole frame and cover.

A practical limit on the yield potential of the gallery is set by the diminishing effect of the depression head, if the gallery is unduly extended from a single point of pumping. For maximum effects to be realised, the pumping operations are best located centrally with reference to the gallery grid, with manhole wells located at the junction of all gallery arms as also at the blind end of each gallery arm. The arrangement could be duplicated with a second pumping point, if the grid system necessary to abstract the required quantity of supply, should be too extensive and unwieldy for a single pumping point. The limiting factor for the total length of gallery under any single arrangement is the ratio between the total quantity abstracted and the total sub-surface flow in the river past the gallery section. So long as the flow abstracted is less than the total flow past the area, additional gallery systems could be inserted in the same area, with one or more pumping points, in order to draw out the maximum quantity. When the maximum quantity possible has been abstracted through a gallery system at a single location, the potentiality of the source at that point will have been fully exploited. In such a case, any augmentation of the supply from the same river as the source will have to be attempted at a new point either upstream or downstream, with a distance left in between, such as would bring into the stream course adequate supplies from the catchment, which could be tapped, without affecting the yield from the gallery already in service.

When infiltration gallery systems are inserted in aquifers with confined groundwater, the rate of abstraction from the gallery must bear a practical relation to the replenishable capacity of the sub surface area which comes within the influence of the gallery under pumping.

The provision of a gallery within a tank or a lake-bed suffers certain inherent disadvantages in that the static water on top, in a state of continuous sedimentation, builds up a silt blanket on the top of the gallery, which may retard the free passage of water through the lake-bed under-layers and into the gallery media. Periodical removal of the surface silty layer so collected would overcome such a handicap.

(e) Check-dams

Under certain conditions, the provision of a sub-soil barrage or check dam across a river just downstream of a gallery system, helps in inundating the river-bed area over the gallery and providing permanent saturation of the sub-soil layers contributing to the yield through the gallery. The barrage is usually keyed into the river-bed on an impermeable layer and into the banks for it to function successfully. Incidentally, it would also save the gallery system against damages by scour during floods.

5.2.6.3 Radial Collector Wells

A collector well consists of a cylindrical well of reinforced concrete say 4 to 5 m in diameter, going into the aquifer to as great a depth of the sub-strata as possible, i.e. upto an impermeable stratum. Normally the saturated aquifer should not be less than 7 m above the top of the radial pipes. From the bottom of the well, slotted steel pipes, normally of 200 mm to 300 mm diameter on the inside and going upto 30-35 metres in length are driven horizontally. The length is determined by the composition and yield from the aquifer. The drain tubes are made up of short length of pipes each 2.4 metres in length which are welded to each other electrically one after the other.

These steel pipes are driven horizontally into the aquifer by means of suitable twin jacks placed in the well and crossing the steining of the well, through the special openings or port holes. At the same time, desanding operation is carried out through the head of the drain pipes. This operation is very important and results in the removal of all the fine particles in the alluvium thus increasing the draw-off.

A sketch of a collector well is given in Appendix-5.7.

(a) Desanding Operation while Driving Radials

An important operation in the driving of the drains is the operation of desanding of drain tubes of 200 mm to 300 mm dia which will remain inside the sand bed being driven to a certain distance. An inner tube is then introduced into the drain which is used for sending a blast of compressed air for loosening and separating the fine particles of the alluvium at the head of the drain. When the compressed air is turned off, the pressure of the water, due to the head of the water table, enables the fine particles into the interior of the well to be carried until clear water without any fine particles is obtained. This indicates that the pressure of the water is insufficient to move the fine particles. Then the drains are driven further.

This process ensures formation of big sheath around the steel drain, composed of the coarser particles in the alluvium; this sheath itself forms a drain of large section of a reverse filter. During the course of desanding, the quantities of sand removed are measured carefully which enables one to estimate the diameter of the sheath thus formed around the drain.

(b) Advantages

(i) The surface of draw-off of collector well is many times greater than that in the case of an ordinary or traditional well. It also ensures a very low velocity of flow with a high total yield.

(ii) The danger of clogging is eliminated by the process of desanding which removes all fine particles around the drains and creates a high sheath through which a large yield with low velocity is obtained.

(iii) The collector well uses 90% of the head available from the water table whereas ordinary well under water table conditions can use only 66%.

(iv) The collector well is able to obtain high yields varying from 500-2500 m^3/hr depending upon the strata and depth of submergence.

(v) The draw-off from a collector well is regulated by valves controlling each radial pipe. The valves have shafts extending to the top of the well, which make control and regulation of the supply easy. This also enables the well to be easily cleaned by closing the valves, if cleaning is ever necessary. The facility of cleaning by repeating the desanding process, if at all there is any clogging and resulting falling off in the yield, ensures a much longer life for the installation, while cleaning of infiltration gallery is difficult and expensive.

(vi) In coarse and saturated river beds, installation of radial collector well system is cheaper both in capital and operation costs than any conventional method.

(c) Limitations

(i) A saturated aquifer of minimum depth of 7-8 m is necessary,

(ii) The aquifer should be coarser than 2mm,

(iii) The aquifer should be homogeneous and loose.

5.2.6.4 Filter Basins

When there is a perennial flow in a river and the sub-soil met with is hard rock below an average depth of 1.5 to 3 m filter basins are constructed to take advantage of the perennial flow, assuming a filter rate similar to that of a slow sand filter. Sand in this area is removed and under-drains, usually loose jointed stoneware pipes or perforated PVC pipes, are laid and covered with sand. The water from the under-drains will be led to a collecting well by C.I. or R.C.C. pipes. The collecting well which is also used as pump house is located on the bank of the river.

5.2.6.5 Syphon Wells

When the depth of saturated aquifer is 20 - 30 m and the conventional wells and galleries cannot be laid to take full advantage of such depths, certain alternate devices have to be tried. A syphon well will be most suitable in this case. A syphon well consists of a masonry well, 4-5 m diameter, sunk to a shallow depth and sealed at the bottom. Tube wells are to be sunk all round the well to the full depth of the aquifer and syphoned into the central well from where the water is pumped.

5.2.6.6 Determination Of The Specific Capacity Of A Well

The specific capacity of a well is the discharge per metre of drawdown at the well. In the case of artesian wells it is usually assumed that the specific capacity is constant within the working limits of the drawdown. The specific capacity decreases with duration of pumping, increase in drawdown and the life of well. High specific capacity can be ensured by proper selection of screens and gravel and thorough development.

(a) Measurement of Drawdown

The actual drawdown in wells under pumping is ascertained in several ways. In the case of shallow tubewells, dug or sunk wells, the more common method is to drop a weighted string upto the water level, before and during pumping and computing the difference. In the case of deep tubewells, a satisfactory procedure is to adopt the air pressure method. An air tube is inserted into the well to reach below the anticipated maximum depressed water level. Air is pumped into the tube and based on the air pressure initially required to depress the water level in the air tube down to its bottom and the reduction in such pressure with increasing drawdown in the well under pumping, the drawdown during the pumping operations is measured by a calibrated gauge at the top.

The specific capacity may be determined either by the discharge method or by the recuperation method.

(b) Discharge Method

Using a pump discharging at a constant rate, the water level is lowered in a well and at intervals of time Δt, the water levels are noted.

The discharge equation for this method will be:

$$Q\Delta T = A\Delta h + Kh\Delta t. \qquad (5.13)$$

where,

Q = steady rate of pumping;

A = area of section of well;

K = specific capacity of the well;

h = average drawdown during the interval Δt;

Δt = interval of time; and

Δh = depression during the interval Δt.

In the above equation, Q, A and Δt are known, Δh is observed, h is measured and K can be calculated for each set of observation.

The selection of the pump capacity should be such that a desirable depression is obtained finally. The time interval Δt should be such that the depressions during the time interval are neither too great nor too small.

When the water level is maintained constantly after a particular drawdown, the equation becomes:

$$Q\Delta t = Kh\Delta t \tag{5.14}$$

or

$Q = Kh$, i.e., the rate of pumping equals the yield for that particular drawdown and sp. cap. $= Q/h$

A practical way to confidently predict yields and drawdowns for larger dia gravel packed permanent production wells is to construct two 65 mm dia test-wells, 0.6 m apart, pumping one well with a centrifugal pump (about 30 KL/min capacity) and measuring the drawdown in the other. The resulting discharge divided by the drawdown in the well 0.6 m away is the expected specific capacity of 1.2 m gravel packed well to be drilled at the site.

5.2.6.7 Maximum Safe Yield And Critical Yield

If the well is not developed to the full capacity of the aquifers, the maximum yield is limited by the maximum permissible drawdown at the well and by the size and the method of construction of the well. In the case of shallow tubular wells, the maximum permissible draw-down may be limited by the suction lift of the pumps or by the depth of the wells. In the case of masonry sunk wells as well as tubewells, the drawdown can be further restricted with a view to preventing sand blows which may disturb the aquifer unduly. Sand blows which help to remove the fines and help in the training of the yield are, however, desirable. The maximum quantity that can be drawn may be fixed with reference to the diameter of the well and the hydraulic subsidence value of the largest size of the particles proposed to be removed during the training of the yield to get the best results. This may be termed the critical yield.

5.2.6.8 Maximum Safe Head Of Depression Or Critical Head Of Depression

From the maximum safe yield and the calculated specific capacity, the safe maximum head of depression can be calculated. The maximum safe head of depression, usually termed the critical head of depression is the limit which, when exceeded, may cause serious sand blows which will disturb the aquifer and cause damage to the well.

5.2.6.9 Other Influencing Factors

(a) Head Losses

The resistances to flow not usually considered are the friction of entrance into the well-tube or well, friction in the tube itself and the velocity head.

Inadequate area of openings into the well and the effects of clogging and corrosion may cause the loss of head of entrances to be a good proportion of the total head. The velocity head is usually too small to be worth considering. The friction head in wells upto 30 metres in depth is usually small, but in deeper wells of small diameter, it is often a very large item and needs to be carefully considered. If the well is cased for large portion of its length, the friction in the casing pipe may be taken into account. Where not cased, the friction would probably be greater, the amount depending on the roughness of the well surface. It may be assumed as 25 percent greater than that for smooth pipes.

Where friction head is of considerable amount, the yield will not be proportional to drawdown but to drawdown minus friction head. For deep wells of small diameter and with high pressures the yield is largely dependent on the pipe friction but with large diameters the yield depends rather upon the ground friction and is little affected by the diameter. Thus, while predicting performance of wells at a site, based on the equations earlier given, well losses must be computed and added.

If the well does not penetrate to the impervious stratum, but reaches short of this, there will be increased resistance near the well for higher quantities of water or, for the same head, the flow will be decreased. This added resistance due to decreased cross-section occurs only in the immediate vicinity of the well and if the total loss of head or total depression is great and if the well extends half or twothird through the porous stratum, the added resistance will be but a small proportion. Where the water bearing formation is made up of layers of different degrees of porosity and the resistance to flow from the stratum to another is great, the yield will be largely influenced by the depth of the well.

(b) Rate of Draw and replenishment

In the case of shallow groundwater supplies, conditions of equilibrium between flow of groundwater and draft from wells are established and the yield of a collecting system will continue from year to year with little variation except that due to rainfall. In the case of deep and artesian supplies of large capacity, however, this is generally not true. The effect of the immense reservoir of stored water commonly present in such cases is such that equilibrium of slope of pressure is established very slowly and the pressure head or groundwater level is likely to continue to decrease for many years. It would be necessary in this case to widen the area of the well system constantly to increase the depth of pumping.

(c) *Yield from Fissures*

Where groundwater flow takes place through fissures and not through the interstices of a porous material, the effect is greatly to increase the capacity of the material and at the same time to modify the law of flow. The resistance to flow through large fissures will vary approximately as the square of velocity instead of the first power. As a result, the yield of a well supplied through fissured sources will not increase at the same rate as the lowering of the water in the well, but much more slowly.

(d) *Draft and Total Flow*

When developing a collecting system, the problem to be decided is the extent to which the groundwater flow can be tapped or utilised. In the case of shallow seated supplies, almost the entire flow over a given width can be captured by suitable design and the ultimate capacity may be a question of total percolation in the tributary area. With a system of wells, the total flow can be utilised only when the water is lowered such that there is no head to cause flow away from the wells on the lower side.

(e) *Mutual Interference*

If two or more wells penetrating to the same stratum are placed near together and are simultaneously operated, the total yield will be relatively much less than the sum of their individual yields when pumped independently to the same level. This mutual interference in wells depends upon the size and spacing of the wells, the radius of the circle of influence of the wells when operated singly and upon the drawdown. The amount of the interference is expressed as the percentage of reduction in yield per well below that of a single well uninfluenced by others.

(f) *Arrangement of Wells*

The most favourable arrangement for a system of small wells is in a line at right angles to the direction of flow of the groundwater, as in this way the largest possible area will be drawn upon. By placing the wells across the line of flow or along a groundwater contour, the advantage of equal heads in the several wells is also secured. Where, an area of small width needs to be drawn upon, the arrangement is not so material, as the water will flow towards the wells from all directions. But with a long line of wells and a large draw off, it is of much importance.

(g) *Spacing of Wells*

The amount of water which can be obtained from a system of wells depends upon the extent by which the water level can be lowered along the line of wells. The maximum amount of water obtainable from a given system of wells would be when they are spaced for enough apart so that their circles of influence will not over-lap. But on account of cost of piping and loss of head by friction, this would not be the most economical spacing. If wells are deep and therefore, expensive, they should be spaced to interfere comparatively to a lesser extent than the shallow wells which could be spaced closer. The extent of mutual interference can be judged by pumping tests on trial wells, or on those first sunk, the wells being operated at different rates and in various combinations. With the information so obtained together with a knowledge of comparative costs of wells, the best spacing of subsequent wells could be determined.

The economical spacing for deep wells will be much greater than for shallow wells and likewise the economical draw-down and yield per well will be much greater. Questions of the size and spacing also depend upon the economy of different types of pumps and a correct solution requires a careful study of all relevant factors governing local conditions.

(h) Coastal Aquifer and Salinity Ingress

In coastal areas, the principal aquifers are the unconsolidated quarternary sedimentary formations deposited under various sedimentary environments. Occasionally, the underlying tertiary formations also contain potential aquifers. Generally, the aquifers in coastal areas occur under confined conditions under high hydraulic head. Often the potential fresh water aquifers are overlying the saline water aquifer or more commonly wedged between the overlying and underlying saline water bodies. Development of such potential fresh water aquifers brings in problems of unusual lowering of piezometric surface coupled with decrease in yields controlled by the reservoir capacity of the aquifers.

Construction of suitable groundwater structures in coastal aquifers is also beset with hazards like vertical downward percolation and/ or upcoming of saline water and corrosion of casing of tubewells while taping the multilayered fresh water aquifers wedged between the saline water aquifers. The peculiar problem of sand rushing in tubewells tapping finegrained aquifers is also observed very frequently.

The variability of geologic conditions and that of groundwater occurrence in the coastal tracts demands special attention for hydrogeological investigations both in exploratory and development stages. Continuous research for improvements in well screens and well design to cater to the special needs of the groundwater development in the coastal tract is essential. Monitoring of groundwater regime consequent to extensive groundwater development would help in suggesting suitable methods to prevent salt intrusion and land subsidence hazards.

State Groundwater Departments and Central Groundwater Board have a good network of observation stations to monitor the water levels and water quality. Some reports on specific studies are also available which may be consulted.

5.2.6.10 Well Development

The object of well development is the removal of silt, fine sand and other such materials from a zone immediately around the well screen, thereby creating larger passages in the formation through which water can flow more freely towards the wells and the development process continued until the stabilisiation of sand and gravel-pack is fully assured. Well development incidentally corrects any clogging or compacting of the water bearing formation which has occurred during drilling and also grades the material in the water bearing formation immediately around the screen in such a way that the well yields sand free water at the maximum capacity. Well development includes the operations of flushing, testing and equipping the wells before they are put into service.

(a) Flushing

Flushing can be done either by (i) surging including washing and agitating or by (ii) pumping and back washing with an air lift.

(i) Surging

If the development operation is to be effective, it must cause reversal of flow through the screen opening of the formation immediately around the well. This is necessary to avoid the bridging of openings by groups of particles as can occur when flow is continuously in one direction. Reversals of flow are caused by forcing the water out of the well through the screen and into the water bearing formation and then removing the force to allow flow to take place from the formation through the screen and back into the well. This process is known as surging. The outflow (with respect to the well) portion of the surge cycle breaks down any bridging of openings that may occur while the inflow portion moves the fine material towards and through the screen into the well from which it is later removed. Surging is done by raising and lowering a plunger which on the downstroke forces water outwards through the screen, the plunger being of either solid plunger or valve type.

(a) Solid type plunger

A simple solid type surge plunger consists of two leather or rubber belt discs sandwiched between wooden discs, all assembled over a pipe nipple with steel plates serving as washers under the end couplings, the leather or the rubber discs forming a reasonably close fit in the well casing.

Before surging, the well should be washed with a jet of water and bailed or pumped to remove some of the mud cake on the face of the bore hole and any sand that may have settled in the screen. This ensures that a sufficiently free flow of water will take place from the aquifer into the well to permit the plunger to run smoothly and freely. The surge plunger is then lowered into a well to a depth of 3 to 5 m under the water but above the top of the screen. A spudding motion is then applied, repeatedly raising and dropping the plunger through a distance of 0.5 to 1 m. If a cable tool drilling rig is used, it should be operated on the long stroke spudding motion. It is important that enough weight be attached to the surge plunger to make it drop readily on the down-stroke. A drill stem or heavy string of pipes is usually found adequate for this purpose.

Surging should be started slowly, gradually increasing the speed but keeping within the limit at which the plunger will rise and fall smoothly. Surging is done for several minutes, the speed, stroke and time for this initial operation being noted. Then the plunger is withdrawn and the bailer or sand pump lowered into the well and the sand accumulation in the screen bailed out and measured. The surging and bailing operations are repeated until little or no sand is pulled into the well. The time should be increased for each successive period of surging as the rate of entry of sand into the well decreases. The sand-pump type of bailer is generally favoured for removing sand during development work.

(b) Valve type plunger

The valve type surge plunger differs from the solid type surge plunger in that the former carries a number of small port holes through the plunger which are covered by soft valve leather.

Valve type surge plungers are operated in a similar manner to solid plungers. They pull water from the aquifer into the well on the upstroke and by allowing some of the water in the well to press upward through the valves on the down-stroke to produce a smaller reverse flow in the aquifer. This creation of a greater in- rush of water to the well than the out-rush during the surging operation is the principal and most important feature of this type of plunger. The valve type surge plunger, because of this feature, is particularly suited to use in developing wells in formations with low permeabilities, since it ensures a net flow of water into the well rather than out of it. A net outward flow can result in the water moving upwards to wash around the outside of the casing since the low permeability of the aquifer will not permit flow readily into it. Washing around the outside of the casing could cause caving of the upper formations and thus create very difficult problems. An incidental benefit gained from the use of this type of plunger is the accumulation of water above the plunger with the eventual discharge of some water, silt and sand over the top of the well. The valves in effect produce a sort of pumping action in addition to the surging of the well and thus reduce the number of times it is necessary to remove the plunger to bail sand out of the well.

Surge plungers can also be operated within the screen. This may be desirable in developing wells with long screens. By operating a plunger within the screen, the surging action can be concentrated at chosen levels until the well is fully developed throughout the entire length of the screen. The surge plungers should, for much use, be sized to pass freely through the screen and its fittings and not form a close fit in them, as is the case when operating within the well casing. Special care must be exercised when surging within the screen to prevent the plunger from becoming sandlocked by settling of sand above it. For this reason the use of plunger within the screens should only be attempted by experienced drillers. Care must also be exercised when using surge plungers to develop wells in aquifers containing many clay streaks or clay balls. The action of the plunger can, under such conditions, cause the clay to plaster over the screen surface with a consequent reduction rather than increase in yield. In addition, surging of the partly or wholly plugged screen can produce high differential pressures, with a possiblity of collapse of the screen.

(ii) Pumping and Backwashing

(a) High velocity jetting

High velocity jetting or back washing of an aquifer with high velocity jets of water directed horizontally through the screen opening is generally the most effective method of well development. The principal items of equipment required are a simple jetting tool, a high pressure pump, the necessary hose, piping, swivel and water tank or other source of safe water supply.

The procedure is to lower the tool on the jetting pipe to a point near the bottom of the screen. The upper end of the pipe is connected through a swivel and hose to the discharge and of a high pressure pump such as the mud pump used for hydraulic rotary drilling. The pump should be capable of operating at a pressure of atleast 7 kg/cm^2 and preferably at about 10.5 kg/cm^2 while delivering 40 to 45 liters per minute for each 5 mm nozzle. While pumping water through the nozzles and screen into the formation, the jetting tool is slowly rotated, thus washing and developing the formation near the bottom of the well screen. The jetting tool is then raised at intervals of a few centimetres and the process repeated until the entire length of the screen has been back washed and fully developed. It has been found that about 10 to 15% more water should be removed from the well than jetted into it, creating a cone of depression and ensuring that the undesirable fines loosened by jetting are purged from the well. Often air lift or centrifugal pumps are used, the major portion of the water being recirculated through a settling tank to be used to supply the high pressure pump for the jetting. Simultaneous pumping and jetting provides the means for measuring the progress of the work. If both are terminated for a few minutes until water levels return to static, the pumping alone can be commenced with periodic water level measurements to determine the specific capacity with time of pumping. Such measurements provide the theoretical expected specific capacity, the actual specific capacity (and therefore instantaneous efficiency measurement) and whether or not improvements are being made with increased development activity can be ascertained.

The high velocity jetting method is more effective in wells constructed with continuous slot type well screens. The greater percentage of open area of this type of screen permits a more effective use of the energy of the jet in disturbing and loosening formation material rather than in being dissipated by merely impinging upon the solid areas of the slotted pipe. Jetting is the most effective of development methods because the energy of the jets is concentrated over small areas at any particular time and every part of the screen can be selectively treated. Thus uniform and complete development is achieved throughout the length of the screen. This method is also relatively simple to apply and not likely to cause trouble as a result of over application.

(b) Pumping

Another back washing method of development, suitable for use in small wells is one which uses a centrifugal pump with a suction force connected directly on the top of the well casing and carrying a sluice valve on the discharge end. This procedure simply involves the periodic opening and closing of the discharge valve when the pump is in operation. This creates a surging effect on the well. This process is continued until the discharge is clear and sand-free. The method is only applicable where static water levels are such as to permit pumping by suction lift. Some damage can be caused to the pump through the wearing of its parts by the sand pumped through it, particularly if in large quantities. The use of the pump to be permanently installed at the well is, therefore, not recommended for use in development of a well by this method.

Development of gravel packed wells is aimed at removing the thin skin of relatively impervious material which is plastered on the wall of the hole and sandwiched between the natural water-bearing formation and the artificially placed gravel. The presence of gravel envelope creates some difficulty in accomplishing the job. Success depends upon the grading of the gravel, the method of development and the avoidance of an excess thickness of gravel pack. The jetting method, because of its concentration of energy over small areas, is usually more effective than the other methods in developing gravel packed wells. The thinner the gravel pack, the more likely is the removal of all the undesirable material, including any fine sand and silt.

The use of dispersing agents such as polyphosphates at about 6 kg per kiloliter of washwater effectively assist in loosening and removing silt and clay from the aquifer as well as the face of the drilled hole. Flushing is stopped when the presence of fine sand in the discharging water is insignificant. During development, the discharge should correspond to the depression of 50 per cent higher than the normal depression at which the tubewell is later pumped on continuous duty. Where a depression of 50 per cent higher than the normal depression can not be arranged, the tubewell may be over developed so as to yield a discharge 20 per cent in excess of the rated discharge.

(c) Testing

A tubewell out of alignment and containing kinks or bends should be rejected because such deviations cause severe wear on the pump-shaft, bearings and discharge casing and, in a severe case, might make it impossible to get a pump in or out. If a deep well turbine pump is to be installed in a tubewell, the housing should be true to line within permissible limits of deviation from its top to a point just below the maximum depth at which it is proposed to set the pump. If an air lift or suction pump is used for pumping, the alignment is not so important and the same claim has been advanced for the submersible type of pump. It is suggested, however, that even if it is intended to install a type of pumping equipment that will function satisfactorily in an out of line well, the requirements of these specifications should be enforced.

Tubewells are to be tested for plumbness and alignment normally after completion of drilling but immediately after the housing pipes are installed but prior to commencing the gravel filling in the case of gravel-shrouded tubewells.

In the case of gravel-shrouded tubewells, if the pipe assembly is found inclined in a slant position before filling the gravels, the assembly should be pulled in a desired direction by applying force through jacks or by other means with a view to rectifying the slantness and bringing the pipe assembly within the permissible limits of verticality. The gravel operation should be undertaken immediately after the verticality has been tested and rectified. If necessary, remedial measures should be adopted in between by means of jacks or any other means to bring the pipe assembly within the permissible limits of verticality.

For wells encased with pipes less than 350 mm diameter, the verticality of the tubewell shall have a deviation not exceeding 10 cm per 30 m of depth of the tubewell and the deviation shall be in one direction and in one plane only. The deviation of the tubewell shall be determined according to the method as recommended in IS: 2800-1964.

After the tubewell is completed, step drawdown tests and recuperation tests are done to determine the well characteristics such as specific capacity and coefficients of transmissibility and permeability of the aquifer to select suitable size and type of pumps to be installed in the tubewells as also well spacing.

The water is also collected during aquifer performance test and analysed chemically for the different constituents depending upon the use to which the tubewell water is to be put.

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(d) Equipping

(i) Selection of Pumps

Depending upon the discharge and drawdown noted during the tests, a suitable pump, such as a centrifugal pump, vertical turbine pump, submersible pump or reciprocating pump shall be fitted to the tubewells.

A recent innovation is to use airtight packers or seals between pump columns and well casings to (i) produce less than atmospheric pressures beneath them which enables more draw-down (a maximum additional of about 6m) and concomitant additional yield to be produced from a well and (ii) prevent oxygen from entering the lower portion of the well and thus inhibiting the growth of aerobic iron bacteria.

(ii) Sanitary Sealing

For all drinking water tubewells it is necessary that the annular space between the bore and the housing pipe be cement grouted upto atleast 5 m below ground level or upto first impermeable layer like clay bed. In gravel packed tubewells, two gravel feeding pipes on either side of the housing pipe should be provided to the full depth of foundation.

5.2.6.11 Failure Of Wells And The Remedial Measures

The clogging of wells by filling with sand or by corrosion or incrustation of the screen may reduce the yield very greatly. Wells may be readily cleaned of sand by means of a sand pump or bucket but if the strainers are corroded, they must be pulled out, cleaned and renewed or replaced. If the clogging is due to fine sand collecting outside the tube, it may be removed to some extent by forcing water into the wells under high pressure, or by use of a hose or by other suitable means. Sometimes instead of the yield of a well becoming less through continued operation, it is actually increased owing probably to the gradual removal of the fine material immediately surrounding the well.

(a) Surging

The principal purpose of surging a well is as described in 5.2.6.10(a) (i), to dislodge clogging and incrusting material from the screen. Immediately after a well has been surged, it should be strongly and continuously pumped until all dislodged material has been removed. Otherwise, the improvement resulting from surging will be only temporary. Surging is not always successful and occasionally it may cause permanent damage.

In surging with a plunger, the drop pipe is removed from the well and a solid plunger fitting the wells of the casing is lowered beneath the water in the well. The plunger is attached to a well rig and is moved violently up and down as in spudding, causing water to rush into and out of the well through the screen. By placing a check valve in the plunger, water can be forced through the screen in one direction only. If the top of the well casing is sealed, compressed air can be discharged into it to force water violently back through the screen. If air is permitted to flow through the aquifer, it may cause "air-logging" or clogging of the aquifer with pockets of air.

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(b) *Use of Dry Ice*

Dry ice or solidified carbon dioxide when dropped into a well quickly turns to a gas generating a strong pressure if the gas is confined. The charge is suddently released to fall into the well and vaporise, generating pressure. The method has its own dangers such as freezing the hands and suffocation of the operators due to fumes of carbon dioxide and rupture or lifting of the casing or collapse of the screen. The method is also not to be advocated because of its practical limitations in operation and utility.

(c) *Chemical Treatment*

Chemicals such as acids, chlorine and sodium hexametaphosphate may be added to a well for the purpose of dissolving or dislodging clogging meterial or incrustation on the screen or in the sand surrounding the screen.

(i) Acids

Acid treatment may be resorted to only where the metal of the screen will not be seriously attacked by them. They should be introduced in sufficiently high concentrations, that the acid concentration will reach at least 25 per cent near the screen, by means of a wide mouthed funnel and 25 mm or smaller dia black iron or plastic pipe. When used in long screens, acid should be added in quantities to fill 1.5 m of the screen and the conductor pipe raised 1.5 m after pouring each instalment. The acid solution in the well should be agitated by means of a surge plunger or other suitable means for 1 to 2 hours following which the well should be bailed until the water is relatively clear and the operation repeated twice or thrice as necessary. If acid is added in granular form, the quantity added should be based on the total volume of water standing in the well, and not on that in the screen only. A number of precautions must be exercised like, all persons handling the acid wearing goggles and water proof gloves, pouring the acid slowly into the water to prepare the solution, provision of adequate ventilation in pump houses or other confined spaces around treated wells and disallowance of personnel to stand in a pit or a depression around the well during treatment (as the heavier toxic gases lend to settle in the lowest areas). After a well has been treated, it should be pumped to waste to ensure the complete removal of all acid before it is turned to normal supply.

(ii) Chlorine

Chlorine treatment (100 to 200 mg/l of available chlorine is needed) with usually calcium or sodium hypochlorite being used as source of chlorine, with proper agitation by the use of high velocity jetting or surging with a surge plunger is found to be more effective than acid treatment, particularly in loosening bacterial growths and slime deposits which often accompany the deposition of iron oxide. The recirculation provided with the use of the jetting technique greatly improves the effectiveness of the treatment. The treatment should be repeated 3 or 4 times to reach every part of the formation that may be affected and it may also be alternated with acid treatment, the latter being performed first.

(iii) Polyphosphates

Polyphosphates effectively disperse silts, clays and the oxides and hydroxides of iron and manganese and the dispersed materials can be easily removed by pumping. In addition they are safe to handle and, therefore, find considerable application in the chemical treatment of wells.

For effective treatment, 12.5 to 25 kg of polyphosphates are needed for every kilolitre of water in the well. A solution is usually made by suspending a wire basket or gunny bag containing the polyphosphate in a tank of water. About a kg. of calcium hypochlorite should be added for every kilolitre of water in the well in order to facilitate the removal of iron bacteria and their slimes and also for disinfection purposes. After pouring this polyphosphate and hypochlorite solution into the well, a surge plunger or the more effective high velocity jetting technique is used to agitate the water in the well. Two or more successive treatments may be used for better results.

No single treatment is suitable for all tubewells. But with proper diagnosing of the well sickness and taking appropriate steps as discussed above the best and cost effective method can be selected. Table 5.3 gives well clogging problem and suggested treatment and Table 5.4 gives application of various well rehabilitation methods on different types of formations.

(iv) Disinfection

The procedure to be adopted for disinfection of new or renovated wells etc. is presented in Appendix 5.8.

TABLE 5.3

WELL CLOGGING PROBLEMS AND SUGGESTED TREATMENTS

Sl.No.	Problem	Treatement Suggested
1.	Clogging due to fine sand, clay and silts.	Sodium hexametaphosphate 50 g/l depending on the capacity of well bore be left therein for 24 hrs. The same should be followed by surging, jetting with chemical mix or normal development till well is freed from clogging.
2.	Chemical clogging	Hydrochloric acid or sulphuric acid with inhibitor are added to the well. The dosage can be kept as in the case of sodium hexametaphosphate.

3.	Bacterial clogging	Chlorine has been found to be effective in loosening this type of clogging. It not only kills the bacteria but it oxidises the material, so that it is dissolved. Calcium hypochlorite should be used to form solution of 200mg/litre which is introduced in well through small polythene pipe. We need 280 gm of hypochlorite at 70% concentration for 1,000 litres in water to give a solution of 200 mg/litres for the killing of bacteria. The well is agitated through surging method, then left for 10 hrs for removal of slimes by bailing/surging or air jetting or air lifting

TABLE 5.4

WELL REHABILITATION FOR VARIOUS ROCK FORMATIONS AND METHODS EMPLOYED

Sl. no.	Method Employed	Unconsolidated (a)	Consolidated Sand Stone (b)	Consolidated Lime Stone (c)
1.	Use of compressed air	Removes the settled deposits of fine silt and clay.	Not very applicable	Not very applicable
2.	Use of Polyphosphates	Removes fine sands, silt, shale and soft iron deposits	Not very effective	Not very effective
3.	Use of hydrochloric acid, followed by chlorine	Removes sulphates, carbonates and iron deposits	Not very effective	Sometimes beneficial, acid treaunent is recommended
4.	Dynamiting	Not used	Effective for all types of well screen deposits	Effective if large charges are introduced
5.	Surging	Same as compressed air	Rarely used	Rarely used

Sl. no.	Method Employed	Unconsolidated (a)	Consolidated Sand Stone (b)	Consolidated Lime Stone (c)
6.	Dry ice (compressed) carbon dioxide gas)	Same as compressed air	Rarely used	Not effective
7.	Chlorine	Removes iron and other bacteria	Same as under (a)	Same as under (a)
8.	Caustic soda	Removes oil scum left by oil lubricated pump	Same as under (a)	Same as under (a)

5.2.6.12 Design Criteria

(a) Tubewells

Design of the tubewell is based on the following considerations:

(i) The effective area of opening of the strainer (the length and diameter of a strainer) is based on the critical velocity of entry of water through the strainer openings (normally 1 to 6 cm/s).

(ii) Velocity of rise in the pipe is usually restricted to 0.6 to 1.2 mps.

(iii) The allowable drawdown arrived at by the formula is usually restricted to 3 to 6 m in soft rocks.

(iv) In a well under water table conditions at least one-third to half the bottom of the aquifer should be screened.

(b) Dugwells

In a shallow dug well, the allowable pumping rate depends on the critical draw-down where the velocity of entry of water may carry the sand, thus resulting in the wells, in tilting.

5.2.7 Development of Surface Sources

5.2.7.1 Intakes

A water works intake is a device or structure placed in a surface water source to permit the withdrawal of water from the source. They are used to draw water from lakes, reservoirs or rivers in which there is either a wide fluctuation in water level or when it is proposed to draw water at the most desirable depth.

(a) Types of Intakes

(i) Wet intakes;

(ii) Dry intakes;

(iii) Submerged intakes; and

(iv) Moveable and floating intakes.

(b) Location

The following factors should be considered for locating the intake:

(i) The location where the best quality of water is available

(ii) Absence of currents that will threaten the safety of the intake

(iii) Absence of ice float etc

(iv) Formation of shoal and bars should be avoided

(v) Navigation channels should be avoided as far as possible

(vi) Fetch of wind and other conditions affecting the waves

(vii) Ice storms

(viii) Floods

(ix) Availability of power and its reliability

(x) Accessibility

(xi) Distance from pumping station

(xii) Possibilities of damage by moving objects and other hazards.

Conditions affecting the quality of water will include currents due to wind, temperature and seasonal turnover and other causes that will bring water of unsuitable quality at the intake. Channels with high velocity currents carrying floating debris and ice are hazardous to the safety of the structure. Navigation channels add to the danger of pollution from toilets and other refuse discharged from ships. Ice flocs are hazardous because of its impact on the structure and closing of the ports even 10 metres below the water surface. Waves are hazardous to the superstructure of an intake; also they stir up mud and silt from the bottom in such quantity as to affect the quality of the water.

A study of the currents in a lake or river should be made before the location of an intake is selected in order to ensure water of the best quality and the avoidance of polluted water.

An intake in an impounding reservoir should be placed in the deepest part of the reservoir, which is ordinarily near the dam, to take full advantage of the reservoir capacity available. Provision for ports at different depths to take advantage of better water quality should be made.

(c) Design Considerations

The intake structures design should provide for withdrawal of water from more than one level to cope up with seasonal variations of depth of water. Undersluices should be provided for release of less desirable water held in storage.

In the design of intake a generous factor of safety must be allowed as forces to be resisted by intakes are known only approximately. The intake in or near navigable channels should be protected by clusters of piles or other devices, against blows from moving objects.

Undermining of foundations due to water currents or overturning pressures, due to deposits of silt against one side of an intake structure, are to be avoided.

The entrance of large objects into the intake pipe is prevented by coarse screen or by obstructions offered by small openings in the crib work placed around the intake pipe. Fine screens for the exclusion of small fish and other small objects should be placed at an accessible point. The area of the openings in the intake crib should be sufficient to prevent an entrance velocity greater than about 8 metres per minute to avoid carrying settlable matter into the intake pipe. Submerged ports should be designed and controlled to prevent air from entering the suction pipe, by keeping a depth of water over the port of at least three diameters of the port opening.

The conduit for conveying water from the intake should lead to a suction well in or near the pumping station. For conduits laid under water, standard cast iron pipe may be used. Larger conduits may be of steel or concrete. A tunnel, although more expensive, makes the safest conduit.

The capacity of the conduit and the depth of the suction well should be such that the intake ports to the suction pipes of pumps will not draw air. A velocity of 60 to 90 cm/s in the intake conduit with a lower velocity through the ports will give satisfactory performance. The horizontal cross-sectional area of the suction well should be three to five times the vertical cross- sectional area of the intake conduit.

The intake conduit should be laid on a continuously rising or falling grade to avoid accumulation of air or gas pockets of which would otherwise restrict the capacity of the conduit.

5.2.7.2 Impounding Reservoirs

Impounding reservoir is a basin constructed in the valley of a stream to store water during excess stream flow and to supply water when the flow of the stream is insufficient to meet the demand for water. For water supply purposes the reservoir should be full when the rate of stream flow begins to become less than the rate of demand for water.

(a) Choice of Reservoir Site

The suitability of a site must be judged from the following stand points:

(i) Quantity of water available.

(ii) Quality of source.

(iii) Possibility of the construction of a reasonably water tight reservoir.

(iv) Distance of the source from the consumer.

(v) Elevation of the supply.

(vi) Possibility of biological troubles in the case of a shallow reservoir.

(b) Physical Considerations

The estimation of the quantity of water which any impounding works will yield is the first consideration in any scheme. This consists essentially of relating the capacity of the reservoir (and therefore the height of the dam) to the distribution of run-off from the catchment area (i.e. the variations in a steam flow) during a dry period. Each catchment area should receive consideration on its merits as it may prove economical to construct sufficient storage to provide the increased yield from the driest five or more consecutive years.

In the preliminary choice of the site for a dam to impound the required quantity of water, the first study will be the topography of the valley under consideration. From an examination of contour maps, alternative sites will probably present themselves as worthy of investigation. A natural construction in the valley will suggest a possible site for a dam, prelimnary calculations of the potential capacity of reservoirs which might be so formed, will indicate whether such a site could be developed to yield the quantity required.

Any scheme designed to develop a particular source only partially should, if possible, be conceived so as to be capable of further expansion. However, if an earthwork dam is to be built, it should be for the full development of the source. Even with the present knowledge of the behaviour and design of earth structures, there is an element of finality in the construction of an earth dam which makes it difficult to raise its height afterwards to any great extent.

From topographical considerations the choice between alternative sites can be resolved in such terms as, the length of dam required, its height, the value of the land and property to be submerged, the effect of its loss on agricultural production, the length of roads to be diverted around or over the new reservoir, the proximity and the elevation of the supply. A careful comparison of all these factors will determine which site is to be preferred.

(c) Geological Considerations

The decision as to the practicability of dam construction on a particularly favoured site is one which rests largely on geological considerations; these fall into three categories, viz, the geology of the catchment area, of the reservoir area and of the dam site itself.

The geological maps should be used to study the nature of the catchment area, the reservoir area and the dam site. In addition, the latter demands a particularly detailed and exhaustive geological exploration. The geological features and sequence of the strata in a catchment may have a profound effect on the distribution of the run off. The presence of permeable strata may account for high percolation losses, particularly if the general direction of the dip is away from the valley. Permeable strata dipping into a catchment area from a neighbouring valley may result in increased run-off and a higher dry-weather flow, since the tendency will be for the water stored underground to issue from the strata as springs.

(d) Site Exploration

The geological investigation should not stop at the determination of a suitable stratum into which a water-tight cut-off can be made, but it should extend to the exploration of the foundations to determine their ability to carry the structure. This will involve the sinking of numerous trial holes or borings in addition to those sunk along the centre line of the dam.

Preliminary exploration work should also include an examination of the valley slopes above the dam in order to discover whether the ground in the vicinity of the dam has been subject to disturbances in the past and whether further movement may occur to endanger the safety of the dam and its ancillary works.

In general, therefore, the preliminary geological investigations should be as complete and exhaustive as possible and the expense of such work (often considerable) will be well justified. Inadequate examination may prove calamitous.

(e) Computation of Storage

To determine the required capacity of a storage reservoir, the first step is to prepare a table showing the amount of rain fall by months for as long a period in the past as possible. Rain records of the Meteorological Department for the locality under consideration are best. If they are not available, the best possible data should be obtained from places at which conditions correspond as closely as possible to those at the place under consideration. The required storage is then based on the drought year expected once in 30 years. In exceptional cases, the figures for the drought year expected once in 20 years may be adopted and during the drought years worse than anticipated once in 20 years, rationing of supplies and more rational use of water will have to be enforced.

The second step is to obtain and study run off records, if there are any, to determine for each month of the year the percentage of rainfall available as run off. Usually such data are very limited and it may be necessary to use known data from an area having similar characteristics.

The third step is to establish and tabulate monthly evaporation losses. These are based on reservoir area, which is not known before hand but generally ranges from 3 to 10 per cent of the water shed area. A table is then prepared to show the expected draft or consumption for each month of the year. The amount of stream flow for each month is determined from runoff records or multiplying the rainfall by the percentage of runoff. The required quantity of water is found by adding the consumption and the estimated losses from evaporation, percolation and leakage.

These data will show the difference between supply and demand for each month. The required storage capacity will be the greatest total deficiency during any succession of months when the stream flow is less than the draft on the reservoir.

A mass diagram can be drawn to determine the required storage. The deficit value occurring one in 30 years may be statistically worked out and used.

Other information furnished by the mass diagram includes; (1) date that the reservoir is full and stops overflowing; (2) dates that the reservoir is full and starts overflowing; (3) the dates that the reservoir is empty; (4) the dates that the level of the surface of the water in the reservoir stops falling and starts to rise, the reservoir not being completely empty; and (5) whether the flow of stream is sufficient or insufficient to fill the reservoir.

The volume of water that can be held in the impounding reservoir can be determined approximately by multiplying average surface areas between contours by contour interval or the prismoidal formula may be used.

$$V = \frac{h}{6}\left(A_1 + A_2 + A_3\right) \tag{5.15}$$

Where,

V = Volume between contours, corresponding to surface areas A_1 and A_3.

A_1, A_2 & A_3 = Respective areas enclosed within lower, medium and upper contour, where contour interval is h.

(f) Biological Considerations

The catchment area should be prepared so that water from the collecting grounds can flow quickly into the reservoir instead of collecting in pools and swamps where it can pick up organic matter. The area to be submerged should be prepared by cutting all the trees and bushes close to the ground and burning out the vegetation. Roots that may be exposed to wave action should also be removed. Buildings, barnyards, manure heaps and such sources of pollution should also be carefully removed from the area to be flooded. The margins of the reservoir should be dressed to avoid irregularities where the water may collect in stagnant pools and favour the growth of weeds. Plants can be removed physically from margins or in reservoirs which may be tedious and may not be very efficient. Caution has to be exercised in the use of herbicides, as it can impart oily or musty odour to water and can be toxic to fish. Channels should be cut to pockets within the reservoir bottom to promote self-draining when the level is lowered. The reservoirs should be made as deep as practicable. They should consist of at least two compartments for reasons explained in 9.2.3.2 c(l) (i) (dd). The intake should permit water to be drawn off at different depths so that the operator can change the depth of draft to exclude the algae-laden water.

(g) Reservoir Management

(i) Silting

Loss of capacity due to the deposition of silt in a reservoir may impair, if not destroy, the usefullness of the reservoir in a few years. It may be minimised by proper site selection, erosion control, reservoir operation and desilting works. The reservoir site may preferably be chosen on a non-silt bearing stream, or the reservoir may be located in a basin off the main channel so that heavily silt-laden waters may be by-passed around the basin. Reservoirs should be located on the smallest drainage area possible. The rate of silting (hectare metres per year per sq. kilometre) under Indian conditions varies from 0.1 to 0.2

After silt has been deposited in a reservoir, there is no practicable method, widely applicable, for removing it other than to operate gates in the dam to flush out the silt to some extent at times of high stream flow. Dredging is expensive and the disposal of the dredged material presents a serious problem

Soil erosion and control are closely related to the silting of reservoirs since without erosion there would be no silting. Erosion prevention methods recommended for soil conservation include proper crop rotation, ploughing on contours, terracing, strip cropping, protected drainage channels, check dams, reforestation, fire control and grazing control.

Hence it is necessary to provide for silting capacity for all impounding reservoirs, based on studies or data pertaining to similar catchments.

(ii) Evaporation

By evaporation, a process by which water passes from the liquid state to the vapour state, water is lost from water surface and moist earth surfaces. Hence it is of importance in determining the storage requirements and estimating losses from impounding reservoirs, and other open reservoirs. Evaporation from water surface is influenced by temperature, barometric pressure, mean wind velocity, vapour pressure of saturated vapour and vapour pressure of saturated air and dissolved salt content of water. The evaporation loss in storage tanks in India amounts to 2-2.5 m/year. It is essential that the available surface storage is adequately protected from evaporation as losses upto 30% can be reduced economically.

A number of liquid and solid organic compounds have the property of spreading on the water surface and forming a thin film. It is possible to select organic compounds which give monomolecular films and are capable of expansion and contracton by wave action thus being undamaged under field conditions. Such a monomolecular film offers resistance to the evaporating water particles as a result of which the rate of evaporation is reduced.

Hexadecanol or Cetyl alcohol and Octadecanol or Stearyl alcohol or a mixture of these two chemicals is commonly used for suppressing evaporation from lakes and reservoirs. NOIGEN-101, which is mixture of Cetyl and Stearyl alcohols and indigenously available may be used for suppressing evaporation from lakes and reservoirs by spraying on water surface so as to cover the entire surface with this film. The chemical can be used in solution, in powder form or as an emulsion. Spraying in powder form is the simplest and most widely used process. A dose of 1.2 kg/hectare/ day is adequate for wind velocities below 8 kmph.

(iii) Seepage

Seepage occurs wherever the sides and bottom of the reservoir are sufficiently permeable to permit entrance of water and its discharge through the ground beneath the surrounding hills. Apart from making them impermeable to the extent possible economically, erosion control measures such as proper crop rotation, contour ploughing, terracing, strip cropping, reforestation or afforestation, cultivation of permanent pastures and the prevention of gully formation though the construction of checkdams could also be useful on a long term basis.

(iv) Algal Problems

Reservoir management is also of value in reducing the algal problems. Small inflows of water rich in organic matter should be bypassed wherever possible instead of allowing them to infect the main body of the water. The water weeds in the reservoir should be controlled by suitable methods such as dragging and under-water cutting. Algicidal measures as described in chapter 9 may be adopted to control algae in reservoirs.

CHAPTER 6
TRANSMISSION OF WATER

Water supply system broadly involves transmission of water from the sources to the area of consumption, through free flow channels or conduits or pressure mains. Depending on topography and local conditions, conveyance may be in free flow and/or pressure conduits. Transmission of water accounts for an appreciable part of the capital outlay and hence careful consideration of the economics is called for, before deciding on the best mode of conveyance. While water is being conveyed, it is necessary to ensure that there is no possibility of pollution from surrounding areas.

6.1 FREE FLOW AND PRESSURE CONDUITS

6.1.1 Open Channels

Economical sections for open channels are generally trapezoidal while rectangular sections prove economical when rock cutting is involved. Uniform flow occurs in channels where the dimensions of the cross-section, the slope and the nature of the surface are the same throughout the length of the channel and when the slope is just equal to that required to overcome the friction and other losses at the velocity at which the water is flowing.

Open channels have restricted use in water works practice in view of the losses due to percolation and evaporation as also the possibility of pollution and misuse of water. Also they need to be taken along the gradient and therefore the initial cost and maintenance cost may be high. While open channels and canals are not recommended to be adopted for conveyance of treated water, they may be adopted for conveying raw water. Sometimes diversion channels meant for carrying floodwaters from other catchments are also used to augment the yield from the reservoirs.

6.1.2 Gravity Aqueducts And Tunnels

Aqueducts and tunnels are designed such that they flow three quarter full at required capacity of supply in most circumstances. For structural and not hydraulic reasons, gravity tunnels are generally horseshoe shaped.

Gravity flow tunnels are built to shorten the route, conserve the head and to reduce the cost of aqueducts, traversing uneven terrain. They are usually lined to conserve head and reduce seepage but they may be left unlined when they are constructed by blasting through stable rock.

Mean velocities, which will not erode channels after ageing, range from 0.30 to 0.60 mps for unlined canals and 1 to 2 metres per second for lined canals.

6.1.3 Pressure Aqueducts And Tunnels

They are ordinarily circular in section. In the case of pressure tunnels, the weight of overburden is relied upon to resist internal pressure. When there is not enough counter-balance to the internal pressure, steel cylinders or other reinforcing structure, for example, provide necessary tightness and strength.

6.1.4 Pipelines

Pipelines normally follow the profile of the ground surface quite closely. Gravity pipelines have to be laid below the hydraulic gradient. Pipes are of cast iron, ductile iron, mild steel, prestressed concrete, reinforced cement concrete, GRP, asbestos cement, plastic etc..

6.2 HYDRAULICS OF CONDUITS

The design of supply conduits is dependent on resistance to flow, available pressure or head, allowable velocities of flow, scour, sediment transport, quality of water and relative cost.

6.2.1 Formulae

There are a number of formulae available for use in calculating the velocity of flow. However, Hazen-Williams formula for pressure conduits and Manning's formula for free flow conduits have been popularly used.

(a) Hazen-Williams Formula

The Hazen-Williams formula is expressed as:

$$V = 0.849\ C\ r^{0.63}\ S^{0.54} \tag{6.1}$$

For circular conduits, the expression becomes

$$V = 4.567 \times 10^{-3}\ C\ d^{0.63}\ S^{0.54} \tag{6.2}$$

and

$$Q = 1.292 \times 10^{-5}\ C\ d^{2.63} S^{0.54} \tag{6.3}$$

Where,

Q = discharge in cubic metre per hour

d = diameter of pipe in mm

V = velocity in mps

r = hydraulic radius in m

S = slope of hydraulic gradeline and

C = Hazen-Williams coefficient.

A chart for the Hazen-Williams formula is given in Appendix 6.1.

(b) Manning's Formula

The Manning's formula is:

$$V = \frac{1}{n} r^{2/3} S^{1/2} \tag{6.4}$$

For circular conduits:

$$V = \frac{3.968 \times 10^{-3} \times d^{2/3} \times S^{1/2}}{n} \quad \text{and} \tag{6.5}$$

$$Q = 8.661 \times 10^{-7} \times \frac{1}{n} d^{8/3} \times S^{1/2} \tag{6.6}$$

Where,

- Q = discharge in cubic metre per hour
- S = slope of hydraulic gradient
- d = diameter of pipe in mm,
- r = hydraulic radius in metres,
- V = velocity in mps, and
- n = Manning's coefficient of roughness

A chart for Manning's formula is given in Appendix 6.2.

(c) Darcy-Weisbach's Formula

Darcy and Weisbach suggested the first dimensionless equation for pipe flow problems as,

$$S = \frac{H}{L} = \frac{f V^2}{2 g D} \tag{6.7}$$

Where,

- H = head loss due to friction over length L in metres
- f = dimensionless friction factor, and
- g = acceleration due to gravity in m/s^2
- V = velocity in mps
- L = length in metres
- D = dia in metres

(d) Colebrook-White formula

The Colebrook - White formula for calculation of frictional coefficient is

$$\frac{1}{\sqrt{f}} = -2\log_{10}\left[\left(\frac{k}{3.7d}\right) + \frac{2.51}{R_e\sqrt{f}}\right]$$

Where

f = Darcy's Friction Coefficient

R_e = Reynold's Number = Velocity x Diameter/Viscosity

d = Diameter of pipe ; k = Roughness projection

For more details of the Colebrook-White formula, reference may be made to any standard reference book on Fluid Mechanics. Recommended Design Values of roughness (k) for pipe materials are shown below.

S.No.	Pipe Material	Value of 'k' mm	
		New	Design
1.	Metallic Pipes - Cast iron and Ductile Iron	0.15	*
2.	Metallic Pipes - Mild Steel	0.06	*
3.	Asbestos Cement, Cement Concrete, Cement Mortar or epoxy lined Steel, C I and D I pipes	0.035	0.035
4.	PVC,GRP and other plastic Pipes	0.003	0.003

* *Reference may be made to IS : 2951 for roughness values of aged metallic pipes.*

6.2.2 COEFFICIENT OF ROUGHNESS

In today's economic climate, it is essential that all water utilities ensure that their resources are invested judiciously and hence there is an urgent need to avoid over designing of the pipelines. Despite technological advancements, improved methods of manufacturing of all types of pipes and advent of new pipe materials, the current practice of adopting conservative Coefficient of Roughness (C values) is resulting in under utilization of the pipe materials.

The coefficient of roughness depends on Reynolds number (hence on velocity and diameter) and relative roughness (d/k). For Reynolds number greater than 10^7, the friction factor 'f' (and hence the C value) is relatively independent of diameter and velocity. However, for normal ranges of Reynolds number of 4000 to 10^6 the friction factor 'f' (and hence the C value) do depend on Diameter, Velocity and relative roughness.

PVC, Glass Reinforced Plastic (GRP) and other plastic pipes are inherently more smooth compared to Asbestos Cement (AC), Concrete and cement mortar/epoxylined metallic pipes. Depending on quality of workmanship during manufacture and the manufacturing process, the AC, Concrete and cement mortar/epoxylined metallic pipes tend to be as smooth as PVC, GRP and other plastic pipes.

The metallic pipes lined with cement mortar or epoxy and Concrete pipes behave as smooth pipes and have shown C values ranging from 140 to 145 depending on diameter and velocity. Reference may be made to " Manual of Water Supply Practices", AWWA/M9 published by American Water Works Association (AWWA), second edition 1995.

With a view to reduce corrosion, increase smoothness, and prolong the life of pipe materials, the metallic pipes are being provided with durable smooth internal linings. AC, Concrete and cement mortar/epoxylined metallic pipes, PVC, GRP and other plastic pipes may not show any significant reduction in their carrying capacity with age and therefore the design roughness coefficient values (C values) should not be substantially different from those adopted for new pipes.

However, pipes carrying raw water are susceptible to deposition of silt and development of organic growth resulting in reduction of carrying capacity of such pipes. In case of buildup of substantial growth /buildup of deposits in such pipes, they can be removed by scraping and pigging the pipelines. Reference may be made to Chapter 10 section 10.3 of this manual ('Preventive maintenance- cleaning of pipes').

Unlined metallic pipes under several field conditions such as carrying waters having tendency for incrustation and corrosion, low flow velocity and stagnant water and alternate wet and dry conditions (resulting from intermittent operations), undergo substantial reduction in their carrying capacity with age. Therefore lower 'C" values have been recommended for design of unlined metallic pipes. As such, use of unlined metallic pipes should be discouraged.

The values of the Hazen-Williams coefficient 'C' for new conduit materials and the values to be adopted for design purposes are shown in Table 6.1.

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Table 6.1: HAZEN - WILLIAMS COEFFICIENTS

Pipe Material	Recommended C Values	
	New Pipes[@]	Design Purpose
Unlined Metallic Pipes		
Cast Iron, Ductile Iron	130	100
Mild Steel	140	100
Galvanized Iron above 50 mm dia. #	120	100
Galvanized Iron 50 mm dia and below used for house service connections. #	120	55
Centrifugally Lined Metallic Pipes		
Cast Iron, Ductile Iron and Mild Steel Pipes lined with cement mortar or Epoxy		
Up to 1200 mm dia	140	140
Above 1200 mm dia	145	145
Projection Method Cement Mortar Lined Metallic Pipes		
Cast Iron, Ductile Iron and Mild Steel Pipes	130*	110**
Non Metallic Pipes		
RCC Spun Concrete,		
Prestressed Concrete		
Up to 1200 mm dia	140	140
Above 1200 mm dia	145	145
Asbestos Cement	150	140
PVC, GRP and other Plastic pipes.	150	145

@ *The C values for new pipes included in the Table 6.1 are for determining the acceptability* *of surface finish of new pipelines. The user agency may specify that flow test may be conducted for determining the C values of laid pipelines.*

\# *The quality of galvanizing should be in accordance with the relevant standards to ensure resistance to corrosion through out its design life.*

* *For pipes of diameter 500 mm and above; the range of C values may be from 90 to 125 for pipes less than 500mm..*

** *In the absence of specific data, this value is recommended. However, in case authentic field data is available, higher values upto 130 may be adopted.*

The coefficient of roughness for use in Manning's formula for different materials as presented in Table 6.2 may be adopted generally for design purposes unless local experimental results or other considerations warrant the adoption of any other lower value for the coefficient. For general design purposes, however, the value for all sizes may be taken as 0.013 for plastic pipes and 0.015 for other pipes.

Table 6.2 : MANNING'S COEFFICIENT OF ROUGHNESS

Type of lining	Condition	n
Glazed coating of enamel Timber	In perfect order	0.010
	(a) Plane boards carefully laid	0.014
	(b) Plane Boards inferior workmanship or aged,	0.016
	(c) Non-plane boards carefully laid	0.016
	(d) Non-plane boards inferior workmanship or aged	0.018
Masonry	(a) Neat cement plaster	0.013
	(b) Sand and cement plaster	0.015
	(c) Concrete, Steel troweled	0.014
	(d) Concrete, wood troweled	0.015
	(e) Brick in good condition	0.015
	(f) Brick in rough condition	0.017
	(g) Masonry in bad condition	0.020
Stone work	(a) Smooth, dressed ashlar	0.015
	(b) Rubble set in cement	0.017
	(c) Fine, well packed gravel	0.020
Earth	(a) Regular surface in good condition	0.020
	(b) In ordinary condition	0.025
	(c) With stones and weeds	0.030

Type of lining	Condition	n
	(d) In poor condition	0.035
	(e) Partially obstructed with debris or weeds	0.050
Steel	(a) Welded	0.013
	(b) Riveted	0.017
	(c) Slightly tuberculated	0.020
	(d) Cement Mortar lined	0.011
Cast Iron & Ductile Iron	(a) Unlined	0.013
	(b) Cement mortar lined	0.011
Asbestos Cement		0.012
Plastic (smooth)		0.011

Note : *Values of n may be taken as 0.015 for unlined metallic pipes and 0.011 for plastic and other smooth pipes.*

The friction factor values in practice for commonly used pipe materials are given in Table 6.3.

TABLE6.3: RECOMMENDED FRICTION FACTORS IN DARCY-WEISBACH FORMULA

Sl. No	Pipe Material	Diameter(mm)		Friction Factor	
		From	To	New	For Design Period of 30 Years
1.	R.C.C.	100	2000	0.01 to 0.02	0.01 to 0.02
2.	A.C	100	600	0.01 to 0.02	0.01 to 0.02
3.	HDPE/PVC	20	100	0.01 to 0.02	0.01 to 0.02
4.	SGSW	100	600	0.01 to 0.02	0.01 to 0.02
5.	C.I. (for corrosive waters)	100	1000	0.01 to 0.02	0.053 to 0.03
6.	C.I (for non-corrosive waters)	100	1000	0.01 to 0.02	0.034 to 0.07
7.	Cement Mortar or Epoxy Lined metallic pipes (Cast Iron, Ductile Iron, Steel)	100	2000	0.01 to 0.02	0.01 to 0.02
8.	G.I.	15	100	0. 014 to 0.03	0.0315 to 0.06

(Reference may be made to I.S. 2951 for calculation of Head Loss due to friction according to Darcy-Weisbach formula).

6.2.3 HAZEN-WILLIAMS FORMULA

The commonly used Hazen-Williams formula has following inherent limitations:

(i) The numerical constant of Hazen-Williams formula (1.318 in FPS units or 0.85 in MKS units) has been calculated for an assumed hydraulic radius of 1 foot and friction slope of 1/1000. However, the formula is used for all ranges of diameter and friction slopes. This practice may result in an error of upto ± 30% in the evaluation of velocity and ± 55% in estimation of frictional resistance head loss.

(ii) The Darcy-Weisbach formula is dimensionally consistent. The Hazen-Williams coefficient C is usually considered independent of pipe diameter, velocity of flow and viscosity. However to be dimensionally consistent and to be representative of friction conditions, it must depend on relative roughness of pipe and Reynold's number. A comparison between estimates of Darcy-Weisbach friction factor f, and its equivalent value computed from Hazen-Williams C for different pipe materials brings out the error in estimation of 'f' upto ±45% in using Hazen Williams formula. It has been observed that for higher 'C' values (new and smooth pipes) and larger diameters, the error is less, whereas it is appreciable for lower 'C' values (old and rough pipes) and lower diameters at higher velocities.

(iii) The Hazen-Williams formula is dimensionally inconsistent, since the Hazen-Williams C has the dimension of $L^{-0.37}$ T^{-1} and therefore is dependent on units employed.

6.2.3.1 Discussion On Various Formulae For Estimation Of Frictional Resistance

(i) With a view to avoid the limitations of the Hazen Williams formula, the present trend is to use the Colebrook-White equation for estimation of friction factors and then use the Darcy-Weisbach formula for estimation of head-loss due to friction in the pipelines. This practice will yield correct results compared to the Hazen Williams formula.

The estimation of Darcy's 'f' for variations in velocity and diameter involves repetitive and tedious calculations. Further, there is a need for assuming a correct k value in the Colebrook-White equation for calculation of friction coefficient 'f' in the Darcy-Weisbach formula. Conservative assumption of 'k' values will also result in under-utilization of carrying capacity of the pipes. However it is recommended that frictional losses should be estimated with Darcy-Weisbach formula by changing 'f' values for varying velocity and diameter combinations and assuming a correct k value in the Colebrook-White equation.

Recommended 'k' values for use in Colebrook-White formula are shown in 6.2.1 (d).

(ii) If there is a choice for use of pipe friction formulae, Darcy-Wiesbach yields accurate results but involves extra computational effort and therefore Hazen-Williams (HW) formula is commonly used. The Modified Hazen Williams (MHW) formula being an improvement was suggested for use in lieu of HW formula. The MHW formula shown in Para 6.2.4 is derived from Darcy-Weisbach (DW) and Colebrook- White equations. Since the friction coefficient depends on relative roughness of pipe and Reynolds number; C_R values also have to be varied for various diameter and velocity combinations to give correct estimation of the frictional resistance which also results in extra computational efforts. Average C_R values are given in Table 6.4 for use in the Modified Hazen Williams formula which will estimate frictional resistance within ±5% accuracy as per Table 6.4. Darcy-Weisbach formula coupled with Colebrook-White equation gives most accurate results followed by modified Hazen-Williams formula and Hazen-Williams formula.

(iii) It is significant to note that irrespective of the formula used for estimation of frictional resistance, it is necessary to adopt different roughness coefficient values for the various velocity-diameter combinations if the frictional resistance is to be accurately estimated involving changing the C values, k or 'f' or C_R values for the same pipe material. In design, various velocity-diameter combinations are required.

6.2.4 MODIFIED HAZEN-WILLIAMS FORMULA

The Modified Hazen Williams formula has been derived from Darcy-Weisbach and Colebrook-White equations and obviates the limitations of Hazen-Williams formula.

$$V = 3.83\, C_R d^{0.6575} (gs)^{0.5525} / \nu^{0.105} \qquad (6.8)$$

Where,

C_R = coefficient of roughness

d = pipe diameter

g = acceleration due to gravity

s = friction slope

ν = viscosity of liquid

For circular conduits, ν_{20^0C} for water = 10^{-6} m^2/s and g = 9.81 m/s^2

The Modified Hazen Williams formula derived as

$$V = 143.534\, C_R\, r^{0.6575}\, S^{0.5525} \qquad (6.9)$$

$$h = [L(Q/C_R)^{1.81}]/994.62D^{4.81} \qquad (6.10)$$

in which,

V	=	velocity of flow in m/s;
C_R	=	pipe roughness coefficient; (1 for smooth pipes; <1 for rough pipes);
r	=	hydraulic radius in m;
s	=	friction slope;
D	=	internal diameter of pipe in m;
h	=	friction head loss in m;
L	=	length of pipe in m; and
Q	=	flow in pipe in m^3/s.

A nomograph for estimation of head loss by Modified Hazen-Williams formula is presented in the Appendix. 6.3.

6.2.5 Effect of Temperature on Coefficient of Roughness

Analysis carried out to evaluate effect of temperature (viscosity) on value of C_R reveals that the maximum variation of C_R for a temperature range of 10^0C to 30^0C is 4.5% for a diameter of 2000 mm at a velocity of 3.0 m/s. In the light of this revelation, C_R values are presented for average temperature of 20^0C.

6.2.6 Experimental Estimation of C_R Values

The coefficients of roughness in various pipe formulae are based on experiments conducted over a century ago. The values of Hazen Williams C, Mannings n and roughness k values in Moody's Diagram have also been used on experimental data collected in early nineteenth century. There have since been major advances in pipeline technology. Both the manufacturing processes and jointing methods have improved substantially over the years and newer pipe materials have come into use. Continued usage of roughness coefficients estimated without recognition of these advances is bound to result in conservative design of water supply systems. Accordingly C_R values of commonly used commercial pipe materials have been experimentally determined in a study conducted within the country. This study covered pipe diameters 100 to 1500 mm over a wide range of Reynold's Numbers (3×10^4 to 1.62×10^6) encountered in practice. The results indicate that centrifugally spun CI, RCC, AC and HDPE pipes behave as hydraulically smooth when new and hence, $C_R = 1$ for these pipes.

The use of Hazen Williams 'C' as per Table 6.1 results in under utilization of above pipe material when new. The extent of under utilization varies from 13 to 40 percent for CI pipes; 23 percent for RCC and AC pipes; and 8.4 percent for HDPE and PVC pipes.

6.2.7 Reduction in Carrying Capacity of Pipes with Age

The values of Hazen-Williams 'C' are at present arbitrarily reduced by about 20 to 23 percent in carrying capacity of pipes with age. Studies have revealed that chemical and bacteriological quality of water and velocity of flow affect the carrying capacity of pipes with age. The data on existing systems in some cities have been analyzed along with the experimental information gathered during the study to bring out a rational approach to the reduction in carrying capacity of pipes with age.

The C_R values obtained in such studies have shown that, except in the case of CI and steel pipes while carrying corrosive water, the current practice of arbitrary reduction in 'C' values as per Sec. 6.2.2 results in under utilization of pipe material to the extent of 38 to 71 percent for CI pipes for non-corrosive water; 46 to 93 percent for RCC pipes and 25 to 64 percent for AC and HDPE pipes.

6.2.8 Design Recommendations for use of Modified Hazen-Williams Formula

The following design recommendations are made to ensure effective utilization of pipe carrying capacity.

(i) New CI, DI, Steel, RCC, AC and HDPE pipes behave as hydraulically smooth and hence CR of 1 is recommended.

(ii) For design period of 30 years, no reduction in CR needs to be effected for RCC, AC, PVC and HDPE pipes irrespective of the quality of water. However, care must be taken to ensure self-cleansing velocity to prevent formation of slimes and consequent reduction in carrying capacity of these pipes with age.

(iii) For design period of 30 years, 15 percent reduction is required for unlined CI and DI pipes if non-corrosive water is to be transported. The design must also ensure self-cleansing velocity.

(iv) While carrying corrosive waters, unlined CI, DI, and steel pipes will loose 47 and 27 percent of their capacity respectively over a design period of 30 years. Hence, a cost trade-off analysis must be carried out between chemical and bio-chemical correction of water quality, provision of a protective lining to the pipe interiors and design at reduced CR value for ascertaining the utility of CI, DI and steel pipe material in the transmission of corrosive waters.

Recommended CR values are presented Table 6.4. The use of the recommended values in conjunction with Modified Hazen-Williams formula or the nomograph will permit fuller utilization of pipe materials.

TABLE 6.4

RECOMMENDED C_R VALUES IN MODIFIED HAZEN-WILLIAMS FORMULA (AT 20°C)

Sl. No.	Pipe Material	Diameter(mm)		Velocity(m/s)		C_R Value When New	C_R Value For Design Period of 30 Years
		From	To	From	To		
1.	RCC	100	2000	0.3	1.8	1.00	1.00
2.	AC	100	600	0.3	2.0	1.00	1.00

		Diameter(mm)		Velocity(m/s)			
3.	HDPE and PVC	20	100	0.3	1.8	1.00	1.00
4.	CI/DI (for water with positive Langelier's index)	100	1000	0.3	1.8	1.00	0.85*
5.	CI/DI (For waters with negative Langelier's index)	100	1000	0.3	1.8	1.00	0.53*
6.	Metallic pipes lined with cement mortar or epoxy (for water with negative Langelier's index)	100	2000	0.3	2.1	1.00	1.00
7.	SGSW	100	600	0.3	2.1	1.00	1.00
8.	GI (for waters with positive Langeliers Index)	15	100	0.3	1.5	0.87*	0.74

**These are average C_R values which result in a maximum error of ± 5% in estimation of surface resistance.*

6.2.9 RESISTANCE DUE TO SPECIALS AND APPURTENANCES

Pipeline transitions and appurtenances add to the head loss, which is expressed as velocity head as $K V^2/2g$ where V and g are in m/s and m/sec^2 respectively or equivalent length of straight pipe. The values of K to be adopted for different fittings are given in Table 6.5 and equivalent length of pipe for different sizes of various fittings with K=1 are given in Table 6.6.

TABLE 6.5 : K-VALUES FOR DIFFERENT FITTINGS

Type of Fitting	Value of K
Sudden contractions	0.3* - 0.5
Entrance shape well rounded	0.5
Elbow 90^0	0.5 - 1.0
45^0	0.4 - 0.75
22^0	0.25 - 0.50
Tee 90^0 take-off	1.5
Straight run	0.3

Type of Fitting	Value of K
Coupling	0.3
Gate valve (open)	0.3** - 0.4
With reducer and increaser	0.50
Globe	10.0
Angle	5.0
Swing check	2.5
Venturi Meter	0.3
Orifice	1.0

*Varying with area ratios.

**Varying with radius ratios.

TABLE 6.6 : EQUIVALENT LENGTH OF PIPE FOR DIFFERENT SIZES OF FITTINGS WITH K = 1

Size in mm	Equivalent length of pipe in metres	Size in mm	Equivalent length of pipe in metres
10	0.3	65	2.4
15	0.6	80	3.0
20	0.75	90	3.6
25	0.9	100	4.2
32	1.2	125	5.1
40	1.5	150	6.0
50	2.1		

6.2.10 GUIDELINES FOR COST EFFECTIVE DESIGN OF PIPELINES

The cost of transmission and distribution system constitutes a major portion of the project cost. It is desirable to adopt the following guidelines:

(i) The design velocity should not be less than 0.6 m/s in order to avoid depositions and consequent loss of carrying capacity.

(ii) In design of distribution systems, the design velocity should not be less than 0.6 m/s to avoid low velocity conditions which may encourage deposition and/or corrosion resulting in deterioration in quality. However, where inevitable due to minimum pipe diameter criteria or other hydraulic constraints, lower velocities may be adopted with adequate provision for scouring.

(iii) In all hydraulic calculations, the actual internal diameter of the pipe shall be adopted after accounting for the thickness of lining, if any, instead of the nominal diameter or outside diameters (OD).

(iv) In providing for head loss due to fittings, specials and other appurtenances, actual head loss calculations based on consideration included in subsection 6.2.9 should be done instead of making an arbitrary provision.

6.3 PIPE MATERIALS

Pipelines are major investments in water supply projects and as such constitute a major part of the assets of water authorities. Pipes represent a large proportion of the capital invested in water supply undertakings and therefore are of particular importance. Therefore pipe materials shall have to be judiciously selected not only from the point of view of durability, life and over all cost which includes, besides the pipe cost, the installation and maintenance costs necessary to ensure the required function and performance of the pipeline throughout its designed life time.

6.3.1 CHOICE OF PIPE MATERIALS

The various types of pipes used are:

I. Metallic pipes : C.I., D.I., M.S., G.I.

   (i) Unlined Metallic pipes

   (ii) Metallic pipes lined with cement mortar or epoxy lining;

II. Non Metallic pipes

   (i) Reinforced Concrete, Prestressed Concrete, Bar Wrapped Steel Cylinder Concrete, Asbestos Cement

   (ii) Plastic Pipes : PVC, Polyethylence, Glass Reinforced Plastic, etc.

The determination of the suitability in all respects of the pipes and specials, for any work is a matter of decision by the Engineer concerned on the basis of requirements for the scheme.

Several technical factors affect the final choice of pipe material such as internal pressures, coefficient of roughness, hydraulic and operating conditions, maximum permissible diameter, internal and external corrosion problems, laying and jointing, type of soil, special conditions, etc.

Selection of pipe materials must be based on the following considerations:

(a) The initial carrying capacity of the pipe and its reduction with use, defined, for example, by the Hazen-Williams coefficient C.

Values of C vary for different conduit materials and their relative deterioration in service. They vary with size and shape to some extent.

(b) The strength of the pipe as measured by its ability to resist internal pressures and external loads.

(c) The life and durability of pipe as determined by the resistance of cast iron and steel pipe to corrosion; of concrete and A.C. pipe to erosion and disintegration and plastic pipe to cracking and disintegration.

(d) The ease or difficulty of transportation, handling and laying and jointing under different conditions of topography, geology and other prevailing local conditions.

(e) The safety, economy and availability of manufactured sizes of pipes and specials.

(f) The availability of skilled personnel in construction and commissioning of pipelines.

(g) The ease or difficulty of operations and maintenance.

The life and durability of the pipe depends on several factors including inherent strength of the pipe material, the manufacturing process along with quality control, handling, transportation, laying and jointing of the pipeline, surrounding soil conditions and quality of water. Normally, the design period of pipelines is considered as 30 years. Where the pipelines have been manufactured properly as per specifications, designed and installed with adequate quality control and strict supervision, some of them have lasted more than the designed life provided the quality of water is non-corrosive. However, pipeline failures for various pipe materials even before the expiry of the designed life have been reported probably due to lack of rigid quality control during manufacture and installation, improper design, presence of corrosive waters, corrosive soil environment, improper bedding and other relevant factors.

Lined metallic pipelines are expected to last beyond the normal design life of 30 years. However, the relative age of such pipes depends on the thickness and quality of lining available for corrosion. The cost of the pipe material and its durability or design life are the two major governing factors in the selection of the pipe material. The pipeline may have very long life but may also be relatively expensive in terms of capital and recurring costs and, therefore, it is very necessary to carryout a detailed economic analysis before selecting a pipe material.

The metallic pipes are being provided with internal lining either with cement mortar or epoxy so as to reduce corrosion, increase smoothness and prolong the life.

Underground metallic pipelines may require protection against external corrosion depending on the soil environment and corrosive ground water. Protection against external corrosion is provided with cement mortar guiniting or hot applied coal-tar asphaltic enamel reinforced with fibreglass fabric yarn.

The determination of the suitability in all respects of the pipeline for any work is a matter of decision by the engineer concerned on the basis of the requirements for the scheme. A checklist in Table 6.7 for selection of pipe material has been provided to facilitate the decision makers in selecting the economical and reliable pipe material for the given conditions.

TABLE 6.7 : CHECK LIST FOR SELECTION OF PIPE MATERIAL

SNo	Attribute	Type of Pipes											Remarks if any
		PVC	AC	CI	DI	MS	PSC	GRP	HDPE	HUME	GI	OTHERS	
1	Hydraulic smoothness (C Value)												
2	Structural strength for external loads												
3	Strength to sustain internal pressure												
4	Ease in handling, transportation and storage												
5	Capacity to withstand damage in handling and maintenance												
6	Resistance to internal corrosion												
7	Resistance to external corrosion												
8	Resistance to heat/sunlight												
9	Resistance to rodent attack												
10	Sustainabilty in Black Cotton Soil												
11	Reliabilty and effective joints												
12	Capable to absorb surge pressure												
3	Ease in maintenance and repairs												
14	Use experience												
15	Durabilty (Sustainable trouble free maintenance)												
16	Consumer satisfacation												
17	Resistance to tampering by anti-social elements												
18	Economy												
19	Availability of specials												
20	Availability of skilled personnel for installation & maintenace												
21	Behaviour of pipe line – likelihood of interruptions due to leakage, bursting etc; and time for repairs												
	Recommended size range for												
	Rising Main												
	Gravity Main												
	Distribution Main												

Note : Weightage - 0 to 10 numbers in relation to the significance of the attributes (10 stands for highest quality) may be considered.

Use of this checklist is strongly recommended for large and medium projects (more than 15 Mld). The checklist can be filled up based on the merits and demerits of relevant pipe materials. It is necessary that a quantitative and qualitative assessment is made to arrive at the most economical and reliable pipe material.

The project report should include provisions for addressing the less favourable attributes along with the cost estimates for the same. Risk factors should be identified and stated clearly in the project report. Risk analysis should be carried out to arrive at the correct decision in selecting the pipe material.

6.3.2 CHECK LIST FOR SPECIFICATIONS FOR MANUFACTURE, SUPPLY, LAYING, JOINTING, TESTING AND COMMISSIONING OF PIPELINES

6.3.2.1 GENERAL

Water utilities often procure pipes from one manufacturer/supplier under one contract, procure the valves and fittings from another manufacturer/supplier under another contract and have them installed under another contract rather than entrusting the entire work of Manufacture, Supply, Laying, Jointing, Testing and Commissioning of pipelines to a single agency. This procedure is resorted to on the plea that it results in economy and saves time.

It is seen that wherever single contracts are not awarded for the entire work of Manufacture, Supply, Laying, Jointing, Testing and Commissioning of pipelines to a single agency, the responsibility for performance of the pipelines could not be assigned to any particular agency. Time delays if any, in procurement of fittings and valves will also affect the completion of the contract and also results in cost overruns. Quite often, at the time of commissioning, deficiencies are noticed which might be due to failure at the manufacturing stage or due to transportation handling, or laying/jointing defects or failure of fittings and valves.

Hence it is desirable that all pipeline contracts are awarded on a single contract responsibility so that quality assurance at various stages of manufacture, supply, delivery, laying, jointing and testing is taken care of by a single agency and the timely completion also rests with a single agency; this may result in receipt of competitive offers and hence results in economy. Further, the water utility's time and resources which otherwise are spent in monitoring the performance of several small contracts can be better utilised for quality management of the contract. This may ensure economy by timely completion and quality construction.

However it is necessary that the specifications for single contract responsibility have to be comprehensive and provide for penalty in delays so that the time and cost over runs can be avoided. There will be several site specific conditions and circumstances for the pipeline installations which vary to such an extent that it is very difficult to recommend a simple/ single all inclusive set of specifications for the pipeline contracts. A check list for drafting specifications for Manufacture, Supply, Laying, Jointing, Testing and commissioning of pipelines for procurement through a single agency is furnished. Judicious selection of items which cover cross country or city installations is required.

6.3.3 CHECK LILST FOR SPECIFICATIONS FOR MANUFACTURE, SUPPLY, LAYING, JOINTING, TESTING AND COMMISSIONING PIPELINES

PART I PROCUREMENT

Section 1 - General

1.1 Scope of work

1.2 Definitions of client, contractor, engineer etc.

1.3 Drawings and documents referred to

1.4 Reference Standards

1.5 Penal clauses for failure to meet the time schedule & performance standards and requirements.

1.6 Basis for Prices; to include all pipes, fittings, valves, jointing materials, including labour, cost of factory testing, lining, coating, marking and all other incidental expenses for manufacture, transportation, insurance and delivery at site. (any exclusions/inclusions may be clearly specified)

Section 2 - Detailed Requirements - Pipes

2.1 Material for pipes (standards for materials), manufacturing operations, testing and inspection

2.2 Diameter of pipe

2.3 Wall thickness/other dimensions of the pipe

2.4 Class of pipe

2.5 Laying length

2.6 Pipe ends-flanged-socket/spigot/plain

2.7 Special pipe lengths and special fittings

2.8 Working Pressures

2.9 Pipe lining and coating both for buried and exposed pipes

Section 3 - Transportation and delivery at site

3.1 Type of trucks used for transportation-length/weight

3.2 Handling equipment for loading and unloading

Section 4 - Field Joints for Pipes

4.1 Requirements for machined couplings/ends

4.2 Flanged/joints, pitch circle, blots type, gasket quality

4.3 Welded joints-runs-thickness

PART II INSTALLATION

Section 1 - Instruction to Bidders

1.1 Procedure for invitation of bids

1.2 Instructions to bidders

1.3 Bidders proposal to include plan/programme for construction

1.4 Agreement and performance bonds

Section 2 - General Specifications

2.1 Definitions

2.2 Scope of Work

2.3 Payment conditions

2.4 Statutory Requirements- Payment of wages-Policy-Environment control-safety

2.5 Personnel

Section 3 - Detailed Specifications

3.1 Time Schedule

3.2 Construction facilities - Right of way - storage space - interference with other services

3.3 Work and materials

3.4 Concrete

3.5 Excavation - Bracing of excavation - Safety to public - Disposal of excess material from excavation

3.6 Maintenance, removal and reconstruction of other interfering facilities

3.7 Safeguarding of excavations and protection of property

3.8 Backfill

3.9 Resurfacing of roads within city and outside

Section 4 - Pipes

4.1 Approval of drawings for laying

4.2 Distribution along trench

4.3 Preparation of bedding

4.4 Lowering and laying

4.5 Jointing

4.6 Inspection and tests

4.7 Bends, manholes, outlets

4.8 Joints- Flanged , bolting materials and gasket- machined ends - welded joints

4.9 Field touch up of site joints.

6.4 CAST IRON PIPES

6.4.1 GENERAL

Most of the old Cast Iron pipes were cast vertically but this type has been largely superceded by centrifugally spun cast iron type manufactured upto a diameter of 1050 mm(IS- 1536-1989). Though the vertically cast iron pipe is heavy in weight, low in tensile strength, and liable to defects of inner surface, it is widely used because of its good lasting qualities. There are many examples of cast iron mains in this country which continue to give satisfactory service even after a century of use.

Cast Iron pipes and fittings are being manufactured in this country for several years. Due to its strength and corrosion resistance, C.I. pipes can be used in corrosive soils and for waters of slightly aggressive character. They are well suited for pressure mains and laterals where tapings are made for house connections. It is preferable to have coating inside and outside of the pipe.

Vertically cast iron pipes shall conform to I.S. 1537-1976. The pipes are manufactured by vertical casting in sand moulds. The metal used for the manufacture of this pipe is not less than grade 15. The pipes shall be stripped with all precautions necessary to avoid wrapping or shrinking defects. The pipes shall be such that they could be cut, drilled or machined.

Cast Iron flanged pipes and fittings are usually cast in the larger diameters. Smaller sizes have loose flanges screwed on the ends of double spigot-spun pipe.

The method of Cast Iron pipe production used universally today is to form pipes by spinning or centrifugal action. Compared with vertical casting in sand moulds, the spun process results in faster production, longer pipes with vastly improved metal qualities, smoother inner surface and reduced thickness and consequent lightweight(IS. 1536 –1989).

Centrifugally cast iron pipes are available in diameters from 80mm to 1050mm and are covered with protective coatings. Pipes are supplied in 3.66m and 5.5m lengths and a variety of joints are available including socket and spigot and flanged joints.

The pipes have been classified as LA, A and B according to their thicknesses. Class LA pipes have been taken as the basis for evolving the series of pipes. Class A allows a 10% increase in thickness over class LA. Class B allows a 20% increase in thickness over class LA.

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The pipes are spigot and socket type. When the pipes are to be used for conveying potable water, the inside coating shall not contain any constituent soluble in water or any ingredient which could impart any taste or odour whatsoever to the potable water, after sterilization and suitable washing of the main.

Experiments in centrifugal casting of iron pipes were started in 1914 by a French Engineer which ultimately resulted in commercial production of spun pipes. Spun pipes are about 3/4 of the weight of vertically cast pipes of the same class. The greater tensile strength of the spun iron is due to close grain allowing use of thinner wall than for that of a vertically Cast Iron pipe of equal length. It is possible by this process to increase the length of the pipe whilst a further advantage lies in the smoothness of the inner surface.

6.4.2 LAYING AND JOINTING

Before laying the pipes, the detailed map of the area showing the alignment, sluice valves, scour valves, air valves and fire hydrants along with the existing intercepting sewers, telephone and electric cables and gas pipes will have to be studied. Care should be taken to avoid damage to the existing sewer, telephone and electric cables and gas pipes. The pipeline may be laid on the side of the street where the population is dense. Pipes are laid underground with a minimum cover of 1 metre on the top of the pipe.

Laying of cast iron pipes for water supply purposes has been generally governed by the regulations laid down by the various municipalities and corporations. These regulations are intended to ensure proper laying of pipes giving due consideration to economy and safety of workers engaged in laying.

6.4.2.1 Excavation And Preparation Of Trench

Excavation may be done by hand or by machine. The trench shall be so dug that the pipe may be laid to the required gradient and at the required depth. When the pipeline is under a roadway a minimum cover of 1.0 m is recommended. The width of the trench at bottom shall provide not less than 200mm clearance on both sides of the pipe. Additional width shall be provided at positions of sockets and flanges for jointing. Depths of pits at such places shall also be sufficient to permit finishing of joints.

6.4.2.2 Handling Of Pipes

While unloading, pipes shall not be thrown down but may be carefully unloaded on inclined timber skids. Pipes shall not be dragged over other pipes and along concrete and similar pavements to avoid damage to pipes.

6.4.2.3 Detection Of Cracks In Pipes

The pipes and fittings shall be inspected for defects and be rung with a light hammer, preferably while suspended, to detect cracks. Smearing the outside with chalk dust helps in

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the location of cracks. If doubt persists further confirmation may be obtained by pouring a little kerosene on the inside of the pipe at the suspected spot. If a crack is present the kerosene seeps through and shows on the outer surface. Any pipe found unsuitable after inspection before laying shall be rejected.

6.4.2.4 Lowering Of Pipes And Fittings

All pipes, fittings, valves and hydrants shall be carefully lowered into the trench by means of derrick, ropes or other suitable tools and equipment to prevent damage to pipe materials and protective coatings and linings. Pipes over 300mm dia shall be handled and lowered into trenches with the help of chain pulley blocks.

6.4.2.5 Cleaning Of Pipes And Fittings

All lumps, blisters and excess coating material shall be removed from socket and spigot end of each pipe and outside of the spigot and inside of the socket shall be wire-brushed and wiped clean and dry and free from oil and grease before the pipe is laid.

After placing a length of pipe in the trench, the spigot end shall be centered in the socket and the pipe forced home and aligned to gradient. The pipe shall be secured in place with approved back fill material packed on both sides except at socket.

The socket end should face the upstream while laying the pipeline on level ground; when the pipeline runs uphill, the socket ends should face the up gradient. When the pipes run beneath the heavy loads, suitable size of casing pipes or culverts may be provided to protect the casing of pipe. High pressure mains need anchorage at dead ends and bends as appreciable thrust occurs which tend to cause draw and even "blow out" joints. Where thrust is appreciable concrete blocks should be installed at all points where movement may occur. Anchorages are necessary to resist the tendency of the pipes to pull apart at bends or other points of unbalanced pressure, or when they are laid on steep gradients and the resistance of their joints to longitudinal or shear stresses is either exceeded or inadequate. They are also used to restrain or direct the expansion and contraction of rigidly joined pipes under the influence of temperature changes. Anchor or thrust blocks shall be designed in accordance with I.S. 5330-1984.

6.4.3 JOINTS

Several types of joints such as rubber gasket joint known as Tyton joint, mechanical joint known as Screw Gland joint, and conventional joint known as Lead joint are used.

6.4.3.1 Categories Of Joints

Joints are classified into the following three categories depending upon their capacity for movement.

(a) Rigid joints

Rigid joints are those which admit no movement at all and comprise of flanged, welded and turned and bored joints. Flanged joints require perfect alignment and close fittings are frequently used where a longitudinal thrust must be taken such as at the valves and meters. The gaskets used between flanges of pipes shall be of compressed fibre board or natural or synthetic rubber. Welded joints produce a continuous line of pipes with the advantage that interior and exterior coatings can be made properly and are not subsequently disrupted by the movement of joints.

(b) Semi Rigid joints

Semi rigid joint is represented by the spigot and socket with caulked lead joint. A semi rigid joint allows partial movement due to vibration etc. The socketed end of the pipe should be kept against the flow of water and the spigot end of the other pipe is inserted into this socket. A twisted spun yarn is filled into this gap and it is adjusted by the yarning tool and is then caulked well. A rope is then placed at the outer end of the socket and is made tight fit by applying wet clay, leaving two holes for the escape of the entrapped air inside. The rope is taken out and molten lead is poured into the annular space by means of a funnel. The clay is then removed and the lead is caulked with a caulking tool. Lead wool may be used in wet conditions. Lead covered yarn is of great use in repair work, since the leaded yarn caulked into place will keep back water under very low pressure while the joint is being made.

(c) Flexible joints

Flexible joints are used where rigidity is undesirable such as with filling of granular medium and when two sections cannot be welded. They comprise mainly mechanical and rubber ring joints or tyton joints which permit some degree of deflection at each joint and are therefore able to stand vibration and movement. In rubber jointing special type of rubber gaskets are used to connect cast iron pipe which are cast with a special type of spigot and socket in the groove, the spigot end being lubricated with grease and slipped into the socket by means of a jack used on the other end. The working conditions of absence of light, presence of water and relatively cool uniform temperature are all conducive to the preservation of rubber and consequently this type of joint is expected to last as long as the pipes. Hence, rubber jointing is to be preferred to lead jointing.

6.4.4 TESTING OF THE PIPELINE

6.4.4.1 General

After laying and jointing, the pipeline must be pressure tested to ensure that pipes and joints are sound enough to withstand the maximum pressure likely to be developed under working conditions.

6.4.4.2 Testing Of Pressure Pipes

The field test pressure to be imposed should be not less than the maximum of the following:

(a) 1 1/2 times the maximum sustained operating pressure.

(b) 1 1/2 times the maximum pipeline static pressure.

(c) Sum of the maximum sustained operating pressure and the maximum surge pressure.

(d) Sum of the maximum pipeline static pressure and the maximum surge pressure, subject to a maximum equal to the work test pressure for any pipe fittings incorporated.

The field test pressure should wherever possible be not less than 2/3 work test pressure appropriate to the class of pipe except in the case of spun iron pipes and should be applied and maintained for atleast four hours. If the visual inspection satisfies that there is no leakage, the test can be passed.

Where the field test pressure is less than 2/3 the work test pressure, the period of test should be increased to atleast 24 hours. The test pressure shall be gradually raised at the rate of 1 $kg/cm^2/min$. If the pressure measurements are not made at the lowest point of the section, an allowance should be made for the difference in static head between the lowest point and the point of measurement to ensure that the maximum pressure is not exceeded at the lowest point. If a drop in pressure occurs, the quantity of water added in order to re-establish the test pressure should be carefully measured. This should not exceed 0.1 litre per mm of pipe diameter per KM of pipeline per day for each 30 metre head of pressure applied.

In case of gravity pipes, maximum working pressure shall be 2/3 work test pressure.

The hydrostatic test pressure at works and at field after installation and the working pressure for different classes of pipes are given in Appendix. 6.4.

The allowable leakage during the maintenance stage of pipes carefully laid and well tested during construction, however should not exceed;

$$qL = \frac{ND\sqrt{P}}{115} \tag{6.11}$$

Where,

qL = Allowable leakage in $cm^3/hour$

N = No of joints in the length of pipe line

D = Diameter in mm

P = The average test pressure during the leakage test in kg/cm^2

where any test of pipe laid indicates leakage greater than that specified as per the above formula, the defective pipe(s) or joint(s) shall be repaired/replaced until the leakage is within the specified allowance.

The above is applicable to spigot and socket Cast Iron pipes and A.C. pressure pipes, whereas, twice this figure may be taken for steel and prestressed concrete pipes.

6.4.4.3 Testing Of Non-Pressure Conduits

In case of testing of non-pressure conduits, the pipeline shall be subject to a test for of 2.5 meters head of water at the highest point of the section under test for 10 minutes. The leakage or quantity of water to be supplied to maintain the test pressure during the period of 10 minutes shall not exceed 0.2 litres/mm dia of pipes per kilometer length per day.

6.5 STEEL PIPES

6.5.1 GENERAL

Steel pipes of smaller diameter can be made from solid bar sections by hot or cold drawing processes and these tubes are referred to as seamless. But the larger sizes are made by welding together the edges of suitably curved plates, the sockets being formed later in a press(IS:3589). The thickness of steel used is often controlled by the need to make the pipe stiff enough to keep its circular shape during storage, transportation and laying as also to prevent excessive deflection under the load of trench back filling. The thickness of a steel pipe is however always considerably less than the thickness of the corresponding vertically cast or spun iron pipe. Owing to the higher tensile strength of the steel, it is possible to make steel pipe of lower wall thickness and lower weight. Specials of all kinds can be fabricated without difficulty to suit the different site conditions. Due to their elasticity, steel pipes adopt themselves to changes in relative ground level without failure and hence are very suitable for laying in ground liable to subsidence. If the pipes are joined by a form of flexible joint, it provides an additional safeguard against failure. Steel pipes being flexible are best suited for high dynamic loading.

6.5.2 PROTECTION AGAINST CORROSION

It must be borne in mind, however, that steel mains need protection from corrosion internally and externally. Against internal corrosion, steel pipes are given epoxy lining or hot applied coal tar/asphalt lining or rich cement mortar lining at works or in the field by the centrifugal process. The outer coating for under ground pipeline may be in cement-sand guiniting or hot applied coal-tar asphaltic enamel reinforced with fibreglass fabric yarn.

6.5.3 LAYING AND JOINTING

Small size mild steel pipes have got threaded ends with one socket. They are lowered down in the trenches and laid to alignment and gradient. The jointing materials for this type of pipes are white lead and spun yarn. The white lead is applied on the threaded end with spun yarn and inserted into socket of another pipe. The pipe is then turned to tighten it. When these pipes are used in the construction of tube wells, the socketed ends after positioning without any jointing material are welded and lowered down. Lining and out-coating is done by different methods to protect steel pipes. While laying, the pipes already stocked along the trenches are lowered down into the trenches with the help of chain pulley block. The formation of bed should be uniform. The pipes are laid true to the alignment and gradient before jointing. The ends of these pipes are butted against each other, welded and a coat of rich cement mortar is applied after welding. Steel pipes may be joined with flexible joints or by welding but lead or other filler joints, hot or cold, are not recommended. The welded joint is to be preferred. In areas prone to subsidence this joint is satisfactory but flexible joints must be provided to isolate valves and branches.

When welding is adopted, plain-ended pipes may be jointed by butt welds or sleeved pipes by means of fillet welds. For laying long straight lengths of pipelines, butt joint technique may be employed. The steel pipes used for water supply include hydraulic lap welded, electric fusion welded, submerged arc welded and spiral welded pipes. The latter are being made from steel strip. For laying of welded steel pipe I.S. 5822-1986 may be referred to.

For more details on different types of steel pipes used, reference may be made to the ISI codes indicated in Appendix 'C'.

For hydraulic testing of steel pipelines, the procedure described for cast iron spun pipes and ductile iron pipes may be followed.

6.6 DUCTILE IRON PIPES

6.6.1 GENERAL

Ductile Iron is made by a metallurgical process which involves the addition of magnesium into molten iron of low sulfur content. The magnesium causes the graphite in the iron to precipitate in the form of microscopic (6.25 micron) spheres rather than the flakes found in ordinary cast iron. The spheroidal graphite in iron improves the properties of ductile iron. It possesses properties of high mechanical strength, excellent impact resistance and good casting qualities of grey cast iron. Ductile iron pipes are normally prepared using the centrifugal cast process. The ductile iron pipes are usually provided with cement mortar lining at the factory by centrifugal process to ensure a uniform thickness through out its length. Cement mortar lining is superior to bituminous lining as the former provides a smooth surface and prevents tuberculation by creating a high pH at the pipe wall and ultimately by providing a physical and chemical barrier to the water.

The Indian standard IS 8329-1994 provides specifications for the centrifugally cast ductile iron pipes(Similar to ISO:2531-1998 and EN:545-1994). These pipes are available in the range of 80 mm to 1000 mm diameter; in lengths of 5.5 to 6 m. These pipes are being manufactured in the country with ISO 9002 accreditation.

Ductile iron pipes have excellent properties of machinability, impact resistance, high wear and tear resistance, high tensile strength, ductility and corrosion resistance. DI pipes having same composition of CI pipe, it will have same expected life as that of CI pipes. The ductile iron pipes are strong, both inner and outer surfaces are smooth, free from lumps, cracks blisters and scars. Ductile Iron pipes stand up to hydraulic pressure tests as required by service regulations. These pipes are approximately 30% lighter than conventional cast iron pipes.

Ductile iron pipes are lined with cement mortar in the factory by centrifugal process and unlined ductile iron pipes are also available. For more details reference may be made to IS 8329 - 1994 for Ductile Iron Pipes.

6.6.2 Ductile Iron Fittings

The ductile iron fittings are manufactured conforming to IS 9523-1980 for Ductile Iron fittings.

6.6.3 Joints

The joints for ductile iron pipes are suitable for use of rubber gaskets conforming to IS 5383.

6.6.4 Laying And Jointing

Reference may be made to para 6.4.2 (laying and jointing of cast iron pipes).

6.6.5 Testing Of Ductile Iron Pipelines

The Ductile Iron pipelines are tested as per para 6.4.4 (testing of the pipeline) The test pressures shall be as per IS 8329 - 1994.

6.7 ASBESTOS CEMENT PIPES

6.7.1 General

Asbestos cement pipes are made of a mixture of asbestos paste and cement compressed by steel rollers to form a laminated material of great strength and density. Its carrying capacity remains substantially constant as when first laid, irrespective of the quality of water. It can be drilled and tapped for connecting but does not have the same strength or suitability for threading as iron and any leakage at the thread will become worse as time passes. However, this difficulty can be over come by screwing the ferrules through malleable iron saddles fixed at the point of service connections as is the general practice. These pipes are not suitable for use in sulphate soils. Due to expansion and contraction of black cotton soil, usage of these pipes may be avoided as far as possible in Black Cotton soils, except where the depth of B.C. soil is clearly less than 0.9 metre below ground level.

The available safety against bursting under pressure and against failure in longitudinal bending, though less than that for spun iron pipes, is nevertheless adequate and increases as the pipe ages. In most cases, good bedding of the pipes and the use of flexible joints are of greater importance in preventing failure by bending, than the strength of pipe itself. Flexible joints are used at regular intervals to provide for repairing of pipes, if necessary.

AC pipes are manufactured from classes 5 to 25 and nominal diameters of 80mm to 600mm with the test pressure of 5 to 25 Kg/cm^2.

AC pipe can meet the general requirements of water supply undertakings for rising main as well as distribution main. It is classified as class 5, 10, 15, 20 and 25, which have test pressures 5, 10, 15, 20 and 25 Kg/cm^2 respectively. Working pressures shall not be greater than 50% of test pressure for pumping mains and 67% for gravity mains.

For further details, refer to IS 1592-1989.

6.7.2 HANDLING

Utmost care must be taken while loading, transportation, unloading, stacking and retransporting to the site to avoid damage to the pipes.

6.7.2.1 Laying And Jointing

The width of the trench should be uniform throughout the length and greater than the outside diameter of the pipe by 300mm on either side of the pipe. The depth of the trench is usually kept 1 meter above the top of the pipe. For heavy traffic, a cover of atleast 1.25 meter is provided on the top of the pipe.

The AC pipes to be laid are stacked along the trenches on the side or opposite to the spoils. Each pipe should be examined for any defects such as cracks, chipped ends, crusting of the sides etc. The defective pipes should be removed forthwith from the site as otherwise they are likely to be mixed up with the good pipes. Before use the inside of the pipes will have to be cleaned. The lighter pipes weighing less than 80Kg can be lowered in the trench by hand. If the sides of the trench slope too much, ropes must be used. The pipes of medium weight upto 200Kg are lowered by means of ropes looped around both the ends. One end of the rope is fastened to a wooden or steel stack driven into the ground and the other end of the rope is held by men and is slowly released to lower the pipe into the trench. After their being lowered into the trench they are aligned for jointing. The bed of the trench should be uniform.

6.7.3 PIPE JOINTS

There are two types of joints for AC pipes.

- Cast iron detachable joint, (CID)and
- AC coupling joint.

(a) Cast Iron Detachable Joints

This consists of two cast iron flanges, a cast iron central collar and two rubber rings along with a set of nuts and bolts for the particular joint. For this joint, the AC pipes should have flush ends. For jointing a flange, a rubber ring and a collar are slipped to the first pipe in that order; a flange and a rubber ring being introduced from the jointing of the next pipe. Both the pipes are now aligned and the collar centralized and the joints of the flanges tightened with nuts and bolts.

(b) A.C. Coupling Joint

This consists of an A.C. Coupling and three special rubber rings. The pipes for these joints have chamfered ends. These rubber rings are positioned in the grooves inside the coupling, then grease is applied on the chamfered end and the pipe and coupling is pushed with the help of a jack against the pipe. The mouth of the pipe is then placed in the mouth of the coupling end and then pushed so as to bring the two chamfered ends close to each other. Wherever necessary, change over from cast iron pipe to AC pipes or vice-versa should be done with the help of suitable adapters. I.S. 6530 - 1972 may be followed for laying A.C. pipes.

6.7.4 PRESSURE TESTING

The procedure for the test as adopted is as follows:

(a) At a time one section of the pipeline between two sluice valves is taken up for testing. The section usually taken is about 500 meters long.

(b) One of the valves is closed and the water is admitted into the pipe through the other, manipulating air valves suitably.

(If there are no sluice valves in between the section, the end of the section can be sealed temporarily with an end cap having an outlet which can serve as an air relief vent or for filling the fine as may be required. The pipeline after it is filled, should be allowed to stand for 24 hours before pressure testing).

(c) After filling, the sluice valve is closed and the pipe section is isolated.

(d) Pressure gauges are fitted at suitable intervals on the crown into the holes meant for the purpose.

(e) The pipe section is then connected to the delivery side of a pump through a small valve.

(f) The pump is then operated till the pressure inside reaches the designed value which can be read from the pressure gauges fixed.

(g) After the required pressure has been attained, the valve is closed and the pump disconnected.

(h) The pipe is then kept under the desired pressure during inspection for any defect, i.e. leakages at the joints etc. The test pressures will be generally as specified in 6.7.1 and Appendix 6.4. The water will then be emptied through scour valves and defects observed in the test will be rectified.

6.8 CONCRETE PIPES

6.8.1 GENERAL

Reinforced concrete pipes used in water supplies are classified as P1, P2 and P3 with test pressures of 2.0, 4.0, and 6.0 Kg/cm^2 respectively. For use as gravity mains, the working pressure should not exceed 2/3 of the test pressure. For use as pumping mains, the working pressure should not exceed half of the test pressure.

Generally concrete pipes have corrosion resistant properties similar to those of prestressed concrete pipes although they have their own features which significantly affect corrosion performance. Concrete pipes are made by centrifugal spinning of vibratory process. Centrifugally spun pipes are subjected to high rotational forces during manufacture with improved corrosion resistance properties. The line of development most likely to bring concrete pressure pipes into more general acceptance is the use of P.S.C. pipes which are widely used to replace reinforced concrete pipes.

6.8.2 LAYING AND JOINTING

The concrete pipes should be carefully loaded, transported and unloaded avoiding impact. The use of inclined planes or chain pulley block is recommended. Free working space on either side of the pipe shall be provided in the trench which shall not be greater than 1/3 the dia of the pipe but not less than 15 cm on either side.

Laying of pipes shall proceed upgrade of a slope. If the pipes have spigot and socket joints the socket ends shall face upstream, The pipes shall be joined in such a way to provide as little unevenness as possible along the inside of the pipe. Where the natural foundation is inadequate, the pipes shall be laid in a concrete cradle supported on proper foundation or any other suitably designed structure. If a concrete cradle is used, the depth of concrete below the bottom of the pipes shall be at least 1/4 the internal diameter of pipe with the range of 10-30cm. It shall extend upto the sides of the pipe atleast to a distance of 1/4 the dia for larger than 300mm.

The pipe shall be laid in the concrete bedding before the concrete has set.

Trenches shall be back filled immediately after the pipe has been laid to a depth of 300mm above the pipe subject to the condition that the jointing material has hardened (say 12 hours at the most). The backfill material shall be free from boulders, roots of trees etc. The tamping shall be by hand or by other hand operated mechanical means. The water content of the soil shall be as near the optimum moisture content as possible. Filling of trench shall be carried on simultaneously on both sides of the pipe to avoid development of unequal pressures. The back fill shall be rammed in 150mm layers upto 900mm above the top of the pipe.

Joints may be of any of the following types

(i) Bandage joint

(ii) Spigot and socket joint (rigid and semi-flexible)

(iii) Collar joint (rigid and semi-flexible)

(iv) Flush joint. (internal and external)

For more details of jointing procedure, reference may be made to I.S. 783-1985.

In all pressure pipelines, the recesses at the ends of the pipe shall be filled with jute braiding dipped in hot bitumen. The quantity of jute and bitumen in the ring shall be just sufficient to fill the recess in the pipe when pressed hard by jacking or any other suitable method.

The number of pipes that shall be jacked together at a time depends upon the dia of the pipe and the bearing capacity of soil. For small pipe upto 250mm dia, six pipes can be jacked together at a time. Before and during jacking, care should be taken to see that there is no offset at the joint. Loose collar shall be set up over the joint so as to have an even caulking space all round and into this caulking space shall be rammed a 1 : 1.5 mixture of cement and sand just sufficiently moistened to hold together in the form of a clod when compressed in the hand. The caulking shall be so firm that it shall be difficult to drive the point of a penknife into it. The caulking shall be employed at both the ends in a slope of 1:1. In the case of non-pressure pipes the recess at the end of the pipes shall be filled with cement mortar 1: 2 instead of jute braiding soaked in bitumen. It shall be kept wet for 10 days for maturing.

6.8.3 PRESSURE TEST

When testing the pipeline hydraulically, the line shall be kept filled completely with water for a week. The pressure shall then be increased gradually to full test pressure as indicated in 6.4.4.2. and maintained at this pressure during the period of test with the permissible allowance indicated therein. For further details, reference may be made to I.S. 458-1971.

6.9 PRESTRESSED CONCRETE PIPES

6.9.1 GENERAL

While RCC pipes can cater to the needs where pressures are upto 6.0 kg/cm^2 and CI and steel pipes cater to the needs of higher pressures around 24 kg/cm^2, the Prestressed Concrete (PSC) pipes cater to intermediate pressure range, while RCC pipes would not be suitable.

The strength of a PSC pipe is achieved by helically binding high tensile steel wire under tension around a concrete core thereby putting the core into compression. When the pipe is pressurized the stresses induced relieve the compressive stress but they are not sufficient to subject the core to tensile stresses. The prestressing wire is protected against corrosion by a surround of cementatious cover coat giving atleast 25mm thick cover.

The PSC pipes are suited for water supply mains where pressures in the range of 6 kg/cm^2 to 20 kg/cm^2 are encountered.

Two types of P.S.C. pipes are in use today:

(i) Cylinder type: Consists of a concrete lined steel cylinder with steel joint rings welded to its ends wrapped with a helix of highly stressed wire and coated with dense cement mortar or concrete.

Recommended specifications for above pipe are covered by Indian and foreign codes IS: 784- 1978 AWWA C-301 EN-639 and EN-642.

(a) Steel Cylinder Prestressed Concrete Pipes are used in America and Europe Confirming to AWWA C-301 and in Europe EN - 642.

Prestressed Concrete Cylinder pipe has the following two general types of construction : (1) a steel cylinder lined with a concrete core or (2) a steel cylinder embedded in a concrete core. In either type of construction, manufacturing begins with a full length welded steel cylinder. Joint rings are attached to each end and the pipe is hydrostatically tested to ensure water-tightness. A concrete core with a minimum thickness of one-sixteenth times the pipe diameter is placed either by the centrifugal process, radial compaction, or by vertical casting. After the core is cured, the pipe is helically wrapped with high strength, hard drawn wire using a stress of 75 percent of the minimum specified tensile strength. The wrapping stress ranges between 150,000 and 189,000 psi (1034 and 1303 Mpa)depending on the wire size and class. The wire spacing is accurately controlled to produce a predetermined residual compression in the concrete core. The wire is embedded in a thick cement slurry and coated with a dense mortar that is rich in cement content.

Size Range : AWWA C-301 covers prestressed concrete cylinder pipe 16 in. (410 mm) in inside diameter and larger. Lined cylinder pipe is commonly available in inside diameters ranging from 16 to 48 in. (410 to 1,220 mm). Sizes upto 60 in. are available from some manufacturers. Embedded cylinder pipe has been manufactured larger than 250 in. (6,350 mm) in diameter and is commonly available in inside diameters of 48 in. (1,220 mm) and larger. Lengths are generally 16 - 24 ft (4.9-7.3 m), although longer units can be furnished.

The technology for manufacture of these pipes is now available with some of the Indian manufacturers.

(ii) **Non cylinder type** : Consists of a concrete core which is pre-compressed both in longitudinal and circumferential directions by a highly stressed wire. The wire wrapping is protected by a coat of cement mortar or concrete.

Physical behaviour of PSC pipes under internal and external load is superior to RCC pipes. The PSC pipe wall is always in a state of compression which is the most favourable factor for impermeability. These pipes can resist high external loads. The protective cover of cement and mortar which covers the tensioned wire wrapping by its ability to create and maintain alkaline environment around the steel inhibits corrosion. PSC pipes are jointed with flexible rubber rings.

The deflection possible during laying of main is relatively small and the pipes cannot be cut to size to close gaps in the pipeline. Special closure units (consisting of a short double spigot piece and a plain ended concrete lined steel tube with a follower-ring assembled at each end) are manufactured for this purpose. The closure unit (minimum length 1.27m) must be ordered specially to the exact length.

Specials such as bends, bevel pipes, flanged tees, tapers and adapters to flange the couplings are generally fabricated as mild steel fittings lined and coated with concrete.

It is worth while when designing the pipeline to make provision for as many branches as are likely to be required in the future and then to install sluice valves or blank flanges on these branches. It is possible to make connections to the installed pipeline by emptying, breaking out and using a special closure unit but this is a costly item.

6.9.2 Laying and Jointing

PSC pressure pipes are provided with flexible joints, the joints being made by the use of rubber gasket. They have socket spigot ends to suit the rubber ring joint. The rubber gasket is intended to keep the joint water tight under all normal conditions of service including expansion, contraction and normal earth settlement. The quality of rubber used for the gasket should be waterproof, flexible and should have a low permanent set. Refer to IS 784 -1978, for laying of PSC pipes.

6.9.3 Pressure Testing

Testing of PSC pipe is the same as given in the para 6.4.4.2.

However the quantity of water added in order to re-establish the test pressure should not exceed 3 litres (instead of 0.1 litres) per mm dia, per km per 24 hours per 30m head for non-absorbent pipes as per the IS 783 (para 15.5.3 pages 28 & 29).

6.9.4 Bar Wrapped Steel Cylinder Concrete Pressure Pipes

6.9.4.1 General

Bar Wrapped Steel Cylinder Concrete Pressure Pipes (confirming to AWWA C 303 and EN639 & EN 641) are reported to be manufactured in India. No Indian Standard is presently available for these pipes. Bar Wrapped Steel Cylinder Concrete Pressure Pipes are available in diameters of 250 mm to 1500 mm and higher diameter pipes can be designed for working pressures upto 25 kgs per sq. cm.. Standard lengths are generally 5 to 6m. Longer length pipes can also be custom made.

6.9.4.2 Manufacture

Manufacture of Bar Wrapped Steel Cylinder Concrete Pressure Pipes begins with fabrication of a thin steel pipe cylinder. Thicker steel joint rings are welded at both ends. Each pipe is hydrostatically tested. A cement mortar lining is placed by centrifugal process inside the cylinder. The lining varies from 12mm to 25 mm. After the lining is cured by steam or water, mild steel rod is wrapped on the cylinder using moderate tension in the bar. The wrapping is to be done under controlled tension ensuring intimate contact with the cylinder. The cylinder and bar wrapping are covered with a cement slurry and a dense mortar coating that is rich in cement. The coating is cured by steam or water.

6.9.4.3 Joints

The standard joint consists of steel joint rings and a continuous solid rubber ring gasket. The field joint can be over lapping/sliding, butt welded or with confined rubber ring as per the clients requirement. In the case of welded & rubber joints, the exterior joint recess is normally grouted and the internal joint space may or may not be pointed with mortar. The AWWA C-303 provides for use of elastomeric sealing ring (rubber joint), and EN 641 provides both elastomeric sealing ring and steel end rings welded together on site. At present the pipes available in India use steel end rings welded at site.

6.10 PLASTIC PIPES

6.10.1 GENERAL

Plastic pipes are produced by extrusion process followed by calibration to ensure maintenance of accurate internal diameter with smooth internal bores. These pipes generally come in lengths of 6 meters. A wide range of injection moulded fittings, including tees, elbows, reducers, caps, pipe saddles, inserts and threaded adapters for pipe sizes upto 200mm are available.

6.10.2 PVC PIPES

The chief advantages of PVC pipes are

- Resistance to corrosion
- Light weight
- Toughness
- Rigidity
- Economical in laying, jointing and maintenance
- Ease of fabrication

The PVC pipes are much lighter than conventional pipe materials. Because of their lightweight, PVC pipes are easy to handle, transport, and install. Solvent cementing technique for jointing PVC pipe lengths is cheaper, more efficient and far simpler. PVC pipes do not become pitted or tuberculated and are unaffected by fungi and bacteria and are resistant to a wide range of chemicals. They are immune to galvanic and electrolytic attack, a problem frequently encountered in metal pipes, especially when burried in corrosive soils or near brackish waters. PVC pipes have elastic properties and their resistance to deformation resulting from earth movements is superior compared to conventional pipe materials specially AC. Thermal conductivity of PVC is very low compared to metals. Consequently water transported in these pipes remain at a more uniform temperature.

Rigid PVC pipes weigh only $1/5^{th}$ of conventional steel pipes of comparable sizes. PVC pipes are available in sizes of outer dia, 20, 25, 32, 50, 63, 75, 90, 110, 140, 160, 250, 290, and 315mm at working pressures of 2,5,4,6, 10 Kg/cm^2 as per IS 4985 - 1988.

Since deterioration and decomposition of plastics are accelerated by ultraviolet light and frequent changes in temperature which are particularly severe in India, it is not advisable to use PVC pipes above ground. The deterioration starts with discolouration, surface cracking and ultimately ends with brittleness, and the life of the pipe may be reduced to 15-20 years.

6.10.3 Precautions in Handling and Storage

Because of their lightweight, there may be a tendency for the PVC pipes to be thrown much more than their metal counterparts. This should be discouraged and reasonable care should be taken in handling and storage to prevent damage to the pipes. On no account should pipes be dragged along the ground. Pipes should be given adequate support at all times. These pipes should not be stacked in large piles, specially under warm temperature conditions, as the bottom pipes may be distorted thus giving rise to difficulty in pipe alignment and jointing. For temporary storage in the field, where racks are not provided, care should be taken that the ground is level, and free from loose stones. Pipes stored thus should not exceed three layers and should be so stacked as to prevent movement. It is also recommended not to store one pipe inside another. It is advisable to follow the practices mentioned as per IS 7634 – Part I.

6.10.4 Laying and Jointing Procedure

6.10.4.1 Trench Preparation

The trench bed must be free from any rock projections. The trench bottom where it is rocky and uneven a layer of sand or alluvial earth equal to 1/3 dia of pipe or 100mm whichever is less should be provided under the pipes.

The trench bottom should be carefully examined for the presence of hard objects such as flints, rock, projections or tree roots. In uniform, relatively soft fine grained soils found to be free of such objects and where the trench bottom can readily be brought to an even finish providing a uniform support for the pipes over their lengths, the pipes may normally be laid directly on the trench bottom. In other cases, the trench should be cut correspondingly deeper and the pipes laid on a prepared under-bedding, which may be drawn from the excavated material if suitable.

6.10.4.2 Laying And Jointing

As a rule, trenching should not be carried out too far ahead of pipe laying. The trench should be as narrow as practicable. This may be kept from 0.30m over the outside diameter of pipe and depth may be kept at 0.60 -1.0m depending upon traffic conditions. Pipe lengths are placed end to end along the trench. The glued spigot and socket jointing technique as mentioned later is adopted. The jointed lengths are then lowered in the trench and when sufficient length has been laid, the trench is filled.

If trucks, lorries, or other heavy traffic will pass across the pipeline, concrete tiles 600 x 600mm of suitable thickness and reinforcement should be laid about 2m above the pipe to distribute the load. If the pipeline crosses a river, the pipe should be buried at least 2m below bed level to protect the pipe.

For bending, the cleaned pipe is filled with sand and compacted by tapping with wooden stick and pipe ends plugged. The pipe section is heated with flame and the portion bent as required. The bend is then cooled with water, the plug removed, the sand poured out and the pipe (bend) cooled again. Heating in hot air over hot oil bath, hot gas or other heating devices are also practiced. Joints may be heat welded, or flamed or with rubber gaskets or made with solvent cement. Threaded joints are also feasible but are not recommended. Jointing of PVC pipes can be made in following ways:

i) Solvent cement

ii) Rubber ring joint

iii) Flanged joint

iv) Threaded joint

For further details on laying & jointing of PVC pipes, reference can be made to IS 4985 – 1988, IS 7634 – Part 1-3.

Socket and spigot joint is usually preferred for all PVC pipes upto 150mm in dia. The socket length should at least be one and half times the outer dia for sizes upto 100mm dia and equal to the outer dia for larger sizes.

For pipe installation, solvent gluing is preferable to welding. The glued spigot socket connection has greater strength than can ever be achieved by welding. The surfaces to be glued are thoroughly scoured with dry cloth and preferably chamfered to 30^0. If the pipes have become heavily contaminated by grease or oil, methylene cement is applied with a brush evenly to the outside surface of the spigot on one pipe and to the inside of the socket on the other. The spigot is then inserted immediately in the socket upto the shoulder and thereafter a quarter (90^0) turn is given to evenly distribute the cement over the treated surface. The excess cement which is pushed out of the socket must be removed at once with a clean cloth. Jointing must be carried out in minimum possible time, time of making complete joint not being more than one minute. Joints should not be disturbed for at least 5 minutes. Half strength is attained in 30 minutes and full in 24 hours. Gluing should be avoided in rainy or foggy weather, as the colour of glue will turn cloudy and milky as a result of water contamination.

6.10.4.3 Pre-Fabricated Connections

In laying, long lengths of pipe, prefabricated double socketed connections are frequently used to join successive pipe lengths of either the same or one size different. The socket in this case must be formed over a steel mandrel. A short length of pipe is flared at both ends and used as the socket connection. The mandrel used is sized such that the internal dia of the

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flared socket matches the outer dia of the spigot to be connected. By proper sizing of the two ends of a connector, it is possible to achieve reduction (or expansion) of pipe size across the connector.

6.10.4.4 Standard Threaded Connections

Normally PVC pipes should not be threaded. For the connections of PVC pipes to metal pipes, a piece of a special thick wall PVC connecting tube threaded at one end is used. The other end is connected to the normal PVC pipe by means of a glued spigot and socket joint. Before installation, the condition of the threads should be carefully examined for cracks and impurities.

Glue can be used for making joints leak proof. Yarn and other materials generally used with metal pipe and fittings should not be used. Generally, it is advisable to use PVC as the spigot portion of the joint.

6.10.5 Pressure Testing

The method which is commonly in use is filling the pipe with water, taking care to evacuate any entrapped air and slowly raising the system to appropriate test pressure. The pressure testing may be followed as in 6.4.4.2.

After the specified test time has elapsed, usually one hour, a measured quantity of water is pumped into the line to bring it to the original test pressure, if there has been loss of pressure during the test. The pipe shall be judged to have passed the test satisfactorily if the quantity of water required to restore the test pressure of 30m for 24 hours does not exceed 1.5 litres per 10 mm of nominal bore for a length of 1 Km.

6.11 POLYETHYLENE PIPES

Rigid PVC and high-density polyethylene pipes have been used for water distribution systems mostly ranging from 15 -150mm dia and occasionally upto 350mm.

Among the recent developments is the use of High-Density Polyethylene pipes. These pipes are not brittle and as such a hard fall at the time of loading and unloading etc. may not do any harm to it. HDPE pipes as per IS 4984 - 1987 can be joined with detachable joints and can be detached at the time of shifting the pipeline from one place to another. Though for all practical purposes HDPE pipes are rigid and tough, at the same time they are resilient and conform to the topography of land when laid over ground or in trenches. They are coilable, easily be bent in installation, eliminating the use of specials like bends, elbows etc., there by reducing fitting and installation costs. HDPE pipes are easy to carry and install. They are lighter in weight and can be carried to heights as on hills. They can withstand movement of heavy traffic. This would not cause damage to the pipes because of their flexural strength. HDPE has excellent free flowing properties. They have non-adherent surface which reject (not attract) any foreign materials which would impede the flow. HDPE pipes are anti-corrosive, have smooth inner surface so that there is less friction and pressure loss is comparatively less.

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HDPE pipes can be jointed by welding.

For further details of PVC and HDPE pipes refer to:

IS 7834 - 1975 Parts1-8

IS 8008 - 1976 Parts1-7

IS 7634 - 1975 Parts1-3

IS 3076 - 1985

IS 4984 - 1987

6.11.1 Medium Density Polyethylene (MDPE) Pipes

The medium density Polyethylene Pipes (MDPE) are now being manufactured in India conforming to ISO specifications (ISO 4427 and BS 6730 - 1986) for carrying potable water. However no BIS is available for these pipes. The MDPE pipes are being used for consumer connection pipes as an alternative to GI pipes. The Polyethylene material used for making the MDPE pipes conforms to PE 80 grade and the MDPE pipes when used for conveying potable water does not constitute toxic hazard and does not support any microbial growth. Further, it does not impart any taste, odour or colour to the water.

The Polyethylene material conforms to PE 80 grade. The MDPE pipes are colour coded black with blue strips in sizes ranging from 20 mm to 110 mm dia for pressure class of PN3.2, PN4, PN6, PN10 and PN16. The maximum admissible working pressures are worked out for temperature of 20 degrees centigrade as per ISO 4427. The pipes are supplied in coils and minimum coil diameter is about 18 times diameter of the pipe.

MDPE compression fittings made of PP, AABS, UPVC are also available in India for use with MDPE pipes. The materials used for the fittings are also suitable for conveying potable water like MDPE pipes. The jointing materials of fittings consists of thermoplastic resins of Polyethylene type, NBR 'O' ring of Nitrile and clamp of Polypropylene, copolymer body, Zinc plated steel reinforcing ring, nuts and balls of special NBR gasket.

The MDPE pipes are lightweight, robust and non-corrodible and hence can be used as alternative material for consumer connections. Since the pipes are supplied in coils, there will be no joints under the roads and bends are avoided resulting in fast, simple and efficient jointing.

6.12 GLASS FIBRE REINFORCED PLASTIC PIPES (G.R.P. PIPES)

Glass fibre Reinforced Plastic (GRP) pipes are now being manufactured in India conforming to IS 12709. The diameter range is from 350 mm to 2400 mm. The pressure class is 3,6,9,12 & 15 kgs/sq. cm. The field test pressures are 4.5,9,13.5,18,22.5 kgs/sq cm. The factory test pressures are 6,12,18,24& 30 kgs/sq cm. Depending on the type of installation, overburden above the crown of the pipe and the soil conditions, four types of stiffness class pipes are available. Standard lengths are 6 & 12 metres, however custom made lengths can also be made. The specials are made out of the same pipe material i.e. Glass fibre Reinforced Plastic (GRP).

The pipes are jointed as per the techniques; Double bell coupling (GRP) for GRP to GRP; Flange Joint (GRP) for GRP to valves, CI pipes or flanged pipes.

Mechanical Coupling (Steel) for GRP to GRP / steel pipe and Butt - strap joint (GRP) for GRP to GRP.

GRP pipes are corrosion resistant, have smooth surface and high strength to weight ratio. It is lighter in weight compared to metallic and concrete pipes. Longer lengths and hence minimum joints enable faster installation.

G.R.P. pipes are widely used in other countries where corrosion resistant pipes are required at reasonable costs. GRP can be used as a lining material for conventional pipes, which are subject to corrosion. These pipes can resist external and internal corrosion whether the corrosion mechanism is galvanic or chemical in nature.

6.12.1 Pipe Installation

GRP pipes being light in weight, can be easily loaded or unloaded by slings, pliable stripes or ropes. A pipe can be lifted with only one support point or two support points, placed about 4 metre apart. Excavation of trench and back filling of materials is similar to that in the case of CI and MS pipes.

Pipes are joined by using double bell couplings in following manner.

(i) Double bell coupling grooves and rubber gasket rings should be thoroughly cleaned to ensure that no dirt or oil is present.

(ii) Lubricate the rubber gasket with the vegetable oil based soap which is supplied along with the pipes and insert it in the grooves.

(iii) With uniform pressure, push each loop of the rubber gasket into the gasket groove. Apply a thin film of lubricant over the gaskets.

(iv) Apply a thin film of lubricant to the pipe from the end of the pipe to the back-positioning stripe.

(v) Lift manually or mechanically the double bell coupling and align with the pipe section.

(vi) Push the coupling onto the pipe by using levers. For large dia pipe, the coupling may be pushed mechanically with even force on the coupling ring.

(vii) Apply a thin film of lubricant over the pipe to be pushed into the coupling just assembled until the stripes on the pipe are aligned between the edge of the coupling.

Thus pipes are coupled together and the rubber gasket acts as a seal making the joint leak-proof. Joint types are normally adhesive bonded, however reinforced overlay and mechanical types such as flanged, threaded, compressed couplings or commercial/ proprietary joints are available.

6.13 STRENGTH OF PIPES

The stresses in a pipe are normally induced by internal pressure, external loading, surge forces and change of temperature, although torsional stresses can also arise. Internal pressure induces circumferential and longitudinal stresses, the latter developing where the line changes in size or direction, or has a closed end. A pipe is usually chosen so as to carry the circumferential stress without extra strengthening or support but if the joints cannot safely transmit the longitudinal stress, anchorages or some other means of taking the load must be provided. Longitudinal stress is absorbed by friction between the outside surface of the pipe and the material in which the line is buried.

A pipe must withstand the highest internal pressure it is likely to be subjected, the general provisions for which have been discussed in section 6.4.4, while surging or water hammer is discussed in 6.17.

External loads generally arise from the weight of the pipe and its contents and that of the trench filling from superimposed loads, including impact from traffic, from subsidence and from wind loads in the case of pipes laid above ground. If a pipe is laid on good and uniform continuous bed and the cover does not greatly exceed the normal, no special strengthening to resist external loading is generally necessary. Loading likely to arise from subsidence is best dealt with by the use of flexible joints and steel pipes. External loading becomes important usually when a line is laid on a foundation providing uneven support (e.g. across a sewer, trench or in rock under deep cover) or is subjected to heavy superimposed surface loads at less than normal cover. The necessity of stronger pipes can often be avoided by careful bedding and trench filling to give additional support. The importance of good bedding under and around the pipe upto at least the horizontal diameter cannot be overemphasized and in some cases concreting may be required.

Excessive distortion of a steel pipe may cause failure of its protective coating but can be limited by the use of strengthening rings. This problem is only likely to arise in very large mains. Distortions at flexible joints can cause leakage.

When a pipeline has to be laid above ground over some obstruction, such as waterway or railway, it may either be carried on a pipe-bridge or be supported on pillars. In the latter case, the pipe ends must be properly designed to resist shear, if the full strength of the pipe as a beam is to be realized. A small diameter pipe is usually thick enough to span short lengths with its ends simply supported, but as diameter and lengths of span increase, the problem becomes more complex and the ends must be supported in saddles or restrained by ring girders. For pipes of more than 900mm in dia the ring girder method will probably provide the most economical design. Structural design of buried pipes is discussed in detail in the companion volume "Manual on Sewerage and Sewage Treatment".

The temperature of the water in a transmission main varies during the year. If the water is derived from underground sources the variation is relatively small, but if it is obtained from surface sources and is filtered through slow sand filters, the variation may be as much as 20^0C during the year. Furthermore, the temperature changes may take place fairly quickly and for these and other reasons, long lengths of rigid mains are to be avoided. Provision of expansion joints to take care of these stresses is necessary. Thrust and anchor blocks are provided to keep the pipe curve in position. In small mains, i.e. the mains with spigot and socket lead joints, the joints themselves allow sufficient movement, although some recaulking may be occasionally necessary. On large steel pipelines with welded joints expansion can be allowed to give a longitudinal stress in the pipes, when first laid. In about four years or so, the ground normally consolidates sufficiently around the pipe so that the stress is transferred to the ground. Valves require to be bridged by steel or reinforced concrete blocks so that the valve bodies are not stressed, as this could affect their water tightness.

In case of PVC pipelines, it should be noted that the coefficient of expansion of PVC is eight times greater than steel and considerable movement can take place in long lengths of rigidly jointed pipelines.

6.13.1 STRUCTURAL REQUIREMENTS

Structurally, closed conduits must resist a number of different forces singly or in combination.

(a) Internal pressure equal to the full head of water to which the conduit can be subjected (see Appendix 6.4).

(b) Unbalanced pressures at bends, contractions, and closures which have been discussed in 6.16. 18.

(c) Water hammer or increased internal pressure caused by sudden reduction in the velocity of the water; by the rapid closing of a gate or shut down of a pump, for example, which has been discussed in 6.17.

(d) External loads in the from of backfill, traffic, and their own weight between external supports (piers or hangers). A reference may be made to the Manual on Sewerage and Sewage Treatment.

(e) Temperature induced expansion and contraction, which is discussed in 6.13.2.

Internal pressure, including water hammer, creates transverse stress or hoop tension. Bends and closures at dead ends or gates produce unbalanced pressures and longitudinal stress. When conduits are not permitted to change length, variations in temperature likewise create longitudinal stress. External loads and foundation reactions (manner of support), including the weight of the full conduit, and atmospheric pressure (when the conduit is under vacuum) produce flexural stress.

6.13.2 Temperature Induced Expansion and Contraction

When the conduits are not permitted to change length due to variations in temperature, longitudinal stresses are created in the conduits, which is calculated as shown below:

(i) Change in pipe length with temperature

$$\Delta = C\,\theta\,L \qquad (6.12)$$

Where θ = change in temperature and C = coefficient of expansion of conduit per Degree Centigrade and is equal to 11.9×10^{-6} for steel, 8.5×10^{-6} for Cast Iron, and 10×10^{-6} for concrete, L = length of pipe.

(ii) Resulting longitudinal stress, $S = C\,\theta\,E$ (6.13)

for pipeline with fixed ends, and E = Young's Modulus of Elasticity, 2,10,000 N/m² for steel, 1,00,000 N/m² for cast iron, and 1400 - 40,000 N/m² for concrete.

(iii) Resulting longitudinal force $P = \pi\,(d+t)\,t\,s$ (6.14)

6.13.3 Cross Section

The selection of the optimum cross section of a transmission main depends upon both hydraulic performance and structural behaviour because hydraulic capacity is a direct function of the hydraulic radius, full circles or half circles posses the highest hydraulic capacity, by virtue of their largest hydraulic radius or smallest frictional surface for a given area. Hence circular cross sections are preferred for closed conduits and the semicircular ones for open conduits whenever structural conditions permit. The cross sections preferred next are those in which circles or semicircles can be inscribed. The following cross sections are generally used:

(a) Trapezoids approaching half a hexagon as nearly as maintainable slopes permit, for canals in earth,

(b) Rectangles with widths equal to twice the depths for canals in rock and flumes of masonry or wood,

(c) Semi-circles for flumes of wood staves or steel,

(d) Horse shoe for grade aqueducts and grade tunnels.

Material high in tensile strength with circular cross sections withstand satisfactorily the internal pressures; external pressures due to earth or rock, not counterbalanced by internal pressures are resisted best by horse shoe sections of materials possessing high compressive strength. The hydraulic properties of horse shoe sections are only slightly poorer than those of circles. Moreover their relatively flat invert makes for easy transport of excavation and construction material, in and out of the aqueduct.

6.13.4 DEPTH OF COVER

One metre cover on pipeline is normal and generally sufficient to protect the lines from external damage. When heavy traffic is anticipated, depth of cover has to be arrived at taking into consideration the structural and other aspects as detailed in 6.13.2. When freezing is anticipated, 1. 5m cover is recommended as discussed in 10.12.

6.14 ECONOMICAL SIZE OF CONVEYING MAIN

6.14.1 GENERAL CONSIDERATIONS

When the source is separated by a long distance from the area of consumption, the conveyance of the water over the distance involves the provision of a pressure pipeline or a free flow conduit entailing an appreciable capital outlay. The most economical arrangement for the conveyance is therefore of importance.

The available fall from the source to the town and the ground profile in between should generally help to decide if a free flow conduit is feasible. Once this is decided, the material of the conduit is to be selected keeping in view the local costs and the nature of the terrain to be traversed. Even when a fall is available, a pumping or force main independently or in combination with gravity main could also be considered. Optimization techniques need to be adopted to help decisions. The most economical size for the conveyance main will be based on a proper analysis of the following factors:

(a) The period of design considered or the period of loan repayment if it is greater than the design period for the project and the quantities to be conveyed during different phases of such period.

(b) The different pipe sizes against different hydraulic slopes which can be considered for the quantity to be conveyed.

(c) The different pipe materials which can be used for the purpose and their relative costs as laid in position.

(d) The duty, capacity and installed cost of the pump sets required against the corresponding sizes of the pipelines under consideration.

(e) The recurring costs on

  (i) Energy charges for running the pump sets,

  (ii) Staff for operation of the pump sets,

  (iii) Cost of repairs and renewals of the pump sets,

  (iv) Cost of miscellaneous consumable stores, and

  (v) Cost of replacement of the pumpsets installed to meet the immediate requirements, by new sets at an intermediate stage of design period. The full design period or the repayment period may be 30 years or more while the pumpsets are designed to serve a period of 15 years.

6.14.2 Evaluation Of Comparable Factors

Every alternative, when analyzed on the above lines, could be evaluated in terms of cost figures on a common comparable basis by:

(i) Capital cost of the most suitable pipe material as laid and jointed and ready for service, including cost of valves and fittings and all ancillaries to the pipeline.

(ii) (a) Capital cost, as installed, of the necessary pump sets corresponding to the pipeline size in (i) above.

(b) The amount which should be invested at present such as would yield with compound interest, the amount necessary to replace the pumpsets in (ii) (a) at the end of their useful life with bigger pumpsets for once or often to cater to the requirements during the design period or the loan repayment period.

(iii) Energy charges; if the pumpsets in (ii) (a) are designed to serve for, say 15 years, the daily pumpage will vary from the initial requirements to the intermediate demand after 15 years. The energy charges will be based on the average of these two daily pumpages, leading to an average annual expenditure on energy charges on such basis.

The replacing of pumps under (ii) (b) will, likewise, involve annual recurring energy charges for the average of the demands during the subsequent 15 years period for the project design or the loan repayment period whichever is greater.

The two annual recurring costs should be capitalized for inclusion as a part of the present investment. For this purpose it is necessary to derive:

(a) The amount of the present investment which would yield an annuity for 15 years equal to the annual energy charges on the initial pump sets, and

(b) The amount of present investment which would commence to yield, over the subsequent 15 years period, the annual energy charges for the replaced pumpsets in (ii) (b),

(c) Apart from the energy charges, the other recurring annual charges comprising the cost of operation and maintenance staff, ordinary repairs and miscellaneous consumable stores.

The present investment which would yield an annuity equal to such annual recurring charges throughout the design period, or loan repayment period (if it exceeds the former), would represent the capitalized cost, for inclusion as part of the total investment now required.

(iv) The addition of the present investment figures as worked out under (i), (ii), (a), (ii) (b), (iii) and (iv) would represent the total capital investment called for in respect of each alternative involving a specific pipeline size and the corresponding pumpsets. A comparison of the total investment so required in respect of the several alternatives examined would indicate the most economical pipeline size to be adopted for any particular project.

(v) In all the above computations, the rate of interest plays an important role and for proper comparison, it may be taken as the rate demanded for the loan repayment.

6.14.3 Scope Of Sinking Fund

In the methods of comparison outlined above, any provision for a sinking fund to replace the pipeline or the pumpsets at the end of the design or loan repayment period where needed has been advisedly not included. It would tantamount to the present generation paying in advance for the amenities for the next generation, in addition to paying for its own amenities through the design period of 30 years. Such a procedure is neither equitable nor expedient, particularly when local finances are unable to shoulder the financial commitments even against the initial installations of such projects.

6.14.4 Pipeline Cost Under Different Alternatives

There are three independent factors bearing on the problem viz., the design period usually limited to a maximum of 30 years, the loan repayment period of 30 - 40 years and the life of the pipeline which may be anything from 50 to 100 years. There is one particular pipe size for which cost should be minimum, considering its capital and maintenance charge, for the loan repayment period. The size of the pipe will be larger if the period considered is the life of the pipeline and this larger size would appear to be less economical if the period is restricted to the loan repayment period.

The issue, therefore, hinges on which size to choose out of the two in a particular project. Whichever size is adopted, the loan therefore has to be repaid, within the specified period, long before the pipeline ceases to be of use. For the investor, the pipe size which will cost him the minimum is the criterion, pipe costs and maintenance being considered over the loan repayment period. The other size based on the life of the pipe material would cost him additional financial burden although it may be the cheapest when considered over the life period of the pipeline.

The sale price for the water will have to be based on the financial obligations on the repayment of the loan and the maintenance costs. The period of repayment of the loan thus enters into the question and the consumer will have to pay a higher price if the comparison is based on the life time of the pipe and not on the loan repayment period or the design period, as the case may be.

The life period of the pipeline as also other components would become a more rational factor when the project is financed entirely from perpetual public debts to be incurred by the promoters and the community pays back in perpetuity against loans raised from time to time for additions, alternations and expansions needed.

Whether the pipe size is based on the loan repayment period or the lifetime of the pipe, its utility to the community will be there even after repayments of the loan. Since the incidence of the financial burden on the consumer will be less in the former case, the method is to be preferred.

6.14.5 Recurring Charges-Design Period Vs. Perpetuity

Annual recurring charges on energy and operation and maintenance are perpetuity, irrespective of the design period or the lift of the pipeline. Their capitalized value is restricted to the design period or the loan repayment period whichever is greater, as reflecting the commitment involved relevant to such period for a proper comparison between alternatives. Otherwise a possible method may be to consider an initial investment which would yield an interest to meet such recurring charges in perpetuity. It is, however, more rational to consider capitalization of the recurring charges over the design or loan repayment period.

6.14.6 Capitalisation V/s Annuity Methods

In 6.14.2(v), the comparison suggested was on the basis of present capitalized value. In the alternative, the capital installation cost of the pipeline could be converted into an annuity for the design period, or loan repayment period whichever is greater, in the same way as a loan discharged through annuities and such annuity added on to the other annual recurring charges for a total comparison between the alternatives.

6.14.7 Selection

The method suggested in 6.14.2 would give a comparative idea of the total capital investment involved whereas the method suggested in 6.14.6 would indicate the annuities involved as between the alternatives. A better concept is perhaps afforded by the former method.

The most economical size of a main can be arrived by evaluating the capital and maintenance cost (capitalized value) for different diameters. Mathematical solution is also possible. The objective (cost) function is formulated to ensure desired system performance, Several optimization techniques are available for minimizing the objective function. One of the simpler methods is one in which its (objective function) first partial derivatives with respect to the several decision variables are set equal to zero. The resulting system of equations is solved exactly or approximately and the principal minors of the determinant of second partial derivatives are investigated to ascertain whether a maximum or minimum is involved (see Appendix 6.5).

While determining the type of the pipe material to be used, alternative alignments, cost of cross drainage works, cost of valves, specials and other appurtenances, should all be considered to determine the most economical size for the conveying main.

6.15 CORROSION

Causes of corrosion and the protective and preventive measures have been discussed in 9.8.

6.16 APPURTENANCES

To isolate and drain pipe sections for test, installation, cleaning and repairs, a number of appurtenances or auxiliaries are generally installed in the line.

6.16.1 LINE VALVES

Main line valves are provided to stop and regulate the flow of water in the course of ordinary operations and in an emergency. There are many types of valves for use in pipeline, the choice of which depends on the duty. The spacing varies principally with the terrain traversed by the line. In urban areas with connections in the distribution system, main aim is to sectionalise the line in order to maintain reasonable service. In larger lines isolating valves are frequently installed at intervals of 1 to 5 Km. The principle considerations in location of the valves are accessibility and proximity to special points such as branches, stream crossings etc. The spacing of valves is a function of economics and operating problems. Sections of the pipeline may have to be isolated to repair leaks. The volume of water which would have to be drained to waste would be a function of spacing of isolating valves.

These valves are usually placed at major summits of pressure conduits. Summits identify the sections of the line that can be drained by gravity, and pressures are least at these points permitting cheaper valves and easier operation. Gravity conduits are provided with valves at points strategic for the operation of supply points, at the two ends of sag pipes and wherever it is convenient to drain the given section.

Normally valves are sized slightly smaller than the pipe diameter and installed with a reducer on either side. In choosing the size; the cost of the valve should be weighed against the cost of head loss through it, although in certain circumstances it may be desirable to maintain the full pipe bore (to prevent erosion or blockage).

It is sometimes advisable to install small diameter bypass valves around large diameter inline valves to equalize pressures across the gate and thus facilitate opening.

6.16.1.1 Sluice Valves

Sluice valves or gate valves are the normal type of valves used for isolating or scouring. They seal well under high pressures 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 hand wheel, and a non rising spindle which is rotated in a screwed attachment in the gate. The rising spindle 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 pressure, resilient or gunmetal scaling faces may be used. For high pressure, stainless steel seals are preferred.

Sluice valves are not intended to be used for continuous throttling, as erosion of the seats and body cavitation may occur. If small flows are required the bypass, valve is more suitable for this duty.

Despite sluice valve's simplicity and positive action, they are sometimes troublesome to operate. They need a big force to unseat them against high unbalanced pressure and large valves take many minutes to turn open or closed, for which power operated or manual operated actuators are also used. Some of these problems can be over come by installing a valve with a smaller bore than the pipeline diameter.

In special situations variations of sluice valves suited to the needs are used; needle valves are preferred for fine control of flow, butterfly valves for ease of operation and cone valves for regulating the time of closure and controlling water hammer.

6.16.1.2 Butterfly Valves

Butterfly valves are used to regulate and stop the flow especially in large size conduits. They are sometimes cheaper than sluice valves for larger sizes and occupy less space. Butterfly valves with no sliding parts have the advantages of ease of operation, compact size, reduced chamber or valve house and improved closing and retarding characteristics.

These would involve slightly higher head loss than sluice valves and also are not suitable for continuous throttling. 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 fully open state, because the thickness of the disc obstructs the flow even when it is rotated to 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. Both types require high torques to open them against high pressure, they often have geared hand wheels or power driven actuators.

Butterfly Valves with loose sealing ring are sometimes not effective, especially at higher pressures. Butterfly valves with fixed liner can overcome this shortcoming, further the butterfly valves with fixed liner needs no frequent maintenance for replacement of sealing ring as in the case of butterfly valves with loose sealing ring. The fixed liner design butterfly valves are now available in India suitable for working pressures up to 16 kgs/sq cm. Presently there is no IS for the fixed liner Butterfly valves.

6.16.1.3 Globe Valves

Globe valves have a circular seal connected axially to a vertical spindle and hand wheel. The seating is a ring perpendicular to the pipe axis. The flow changes direction through 90^0 twice thus resulting in high head losses. These valves are normally used in small bore pipe work and as taps, although a variation is used as a control valve.

6.16.1.4 Needle And Cone Valves

Needle valves are more expensive than sluice and butterfly valves but are well suited for throttling flow. They have a gradual throttling action as they close, whereas sluice valves and butterfly valves offer little flow resistance until practically shut and may suffer cavitation damage. Needle valves may be used with counter balance weights, springs, or actuators to maintain constant pressure conditions either upstream or downstream of the valve or to maintain a constant flow. They are resistant to wear even at high flow velocities. The method of sealing is to push an axial needle or spear shaped cone into a seat. There is often a pilot needle which operates first to balance the heads before opening. The cone valve is a variation of the needle valve but the sealing cone rotates away from the pipe axis instead of being withdrawn axially.

The needle and cone valves are not commonly used in water supply but are occasionally used as water hammer release valves when coupled to an electric or hydraulic actuator.

6.16.2 Scour Valves

In pressure conduits, small gate off-take known as blow-off or scour valves are provided at low points above line valves situated in the line on a slope such that each section of the line between valves can be emptied and drained completely. They discharge into natural drainage channel or empty into a sump from which water can be pumped to waste.

The exact location of scour valves is frequently influenced by opportunities to dispose off the water. Where a main crosses a stream or drainage structure, there will usually be a low point in the line, but if the main goes under the stream or drain, it cannot be completely drained into the channel. In such a situation it is better to locate a scour connection at the lowest point that will drain by gravity and provide for pumping out the part below the drain pipeline.

There should be no direct connection to sewers or polluted watercourses except through a specially designed trap chamber or pit. For safety, two blow off valves are placed in series. The outlet into the channel should be above the high water line. If the outlet must be below high water, a check valve must be placed to prevent back flow.

The size depends on local circumstances especially upon the time in which a given section of line is designed to be emptied and upon the resulting velocity of flow. Calculations are based upon orifice discharge under a falling head, equal to the difference in elevation of the water surface in the conduit and the blow off less the friction head. Frequency of operation depends upon the quality of the water carried, especially on silt loads.

6.16.3 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. It is also undesirable to have air pockets in the pipe as they may cause water hammer pressure fluctuations during operation of the pipeline. Other problems due to air include corrosion, reduced pump efficiency, malfunctioning of valves or vibrations. Air valves are fitted to release the air automatically when a pipeline is being filled and also to permit air to enter the pipeline when it is being emptied. Additionally air valves have also to release any entrained air, which might be accumulated at high points in the pipeline during normal operations.

Without air valves, vacuum may occur at peaks and the pipe could collapse or it may not be possible to drain the pipeline completely.

Air valves require care in selection and even more care in siting and it is good practice to plan the pipeline alignment to avoid air troubles altogether. A special study of the possible air problems is necessary at the design stage itself and provision should be made for suitable corrective measures rather than positioning arbitrary air valves at pipeline peaks.

Locations of air valves can be at both sides of gates at summits, the downstream side of other gates and changes in grade to steeper slopes in sections of line not otherwise protected by air valves.

The valve usually takes the form of a rigid buoyant vulcanite or rubber-covered ball seated on a rubber or metal ring. The sealing element i.e., the ball is slated against an opening at the top of the valve when the pipe is full and seals the opening. When the pressure inside the pipe falls below external pressure, the ball drops thereby permitting air to be drawn into the pipe. The valves are mainly available in two forms, either single-ball or double ball. The single ball type can have either a large orifice or a small orifice, the former being only suitable for emptying and filling of pipelines and latter for discharging small quantities of entrained air. Double air valves are available which can be classified as dual purpose with a large orifice and small orifice in one unit, with a common connection to the main. For large aqueduct pipelines, a triple orifice air valve is available with two large orifices and one small. For high pressures, stainless steel floats are used instead of the vulcanite-covered balls.

Special designs of air valves are also available which operate satisfactorily with high-velocity air discharges. If normal air valves are used under these conditions, there is a danger that the ball might be carried on to its seat by the air stream before the accumulated air has been fully released.

Air valves can be provided with an integral stop valve or alternatively and preferably, a standard sluice valve can be bolted to the inlet flange, which must be of adequate size for its duty. Regular maintenance checks on atleast an annual basis should be carried out to ensure that the balls are free to move and that the seats do not leak. If an air valve is isolated for any reason in very cold weather, the body should be drained to prevent frost damage; a plug cock can be fitted at the base of the body for this purpose. Trapped chamber drainage is essential to prevent any possibility of stagnant or polluted water or air entering the pipeline.

Automatic air valves in urban streets present a serious contamination risk, since they must have air vents that could, in some circumstances, admit polluted surface water. Constructing an air valve chamber as water tight as possible and fitting a ball valve interceptor as on outlet to a storm water sewer is a practice to obviate this possibility. Using manually operated air valves in the streets, it being the routine duty of a turncock in the area to air the main, to minimize the risk of serious contamination, is yet another practice.

The following ratios of air valves to conduit diameter provide common but rough estimates of needed sizes:

For release of air only	1:12
For admission as well as release of air	1:8

An analysis of air-inlet valves for steel pipelines, Parmakian takes the compressibility of air into account and combines equations for safe differential pressures of cylindrical steel pipe, pipeflow, and air flow, in the following approximate relationships:

$$d_a / d = 1.99 \times 10^{-2} \sqrt{\Delta V / C} \left[1 - P_2 / P_1 \times 0.288 \right] - 0.25 \qquad (6.15)$$

for $P_2 > 0.53\ P_1$ and as

$$d_a / d = 3.91 \times 10^{-2} \sqrt{\Delta V / C} \left(P_2 / P_1 \right)^{0.356} \qquad (6.16)$$

for $P_2 \leq 0.53\ P_1$, because air flow cannot increase beyond a critical differential of 0.488 Kg/cm^2.

In these equations, d_a and d are respectively the diameter of the air orifice and pipe, ΔV is the difference in the velocities of flow on each side of the inlet valve, C is the coefficient of discharge of the valve, and $P_2$and P_1, are the pressures inside and outside the pipe respectively, with $P_1 - P_2$ not exceeding one half the collapsing pressure as a matter of safety.

The equations apply strictly only to elevations of 304.8m above mean sea level at 40 degrees latitude (g = 9.81 mps) temperatures of 25.32^{0}C, 20% humidity, an adiabatic expansion for which $pv^n = pv^{1.40}$, the air occupying a volume of 0.87 cum/Kg.

6.16.3.1 Air Release Valves

Air Release valves are designed specifically to vent, automatically and when necessary, air accumulations from lines in which water is flowing. Such accumulations of air tend to collect at high points in the pipeline. Air which accumulates at such peaks, reduces the useful cross sectional area of the pipe, and therefore induces a friction head factor that lowers the pumping capacity of the entire line. The use of air release valves eliminates the possibility of this air binding and permits the flow of water without damage to pipeline.

Small orifice air valves are designated by their inlet connection size, usually 12 to 50 mm diameter. This has nothing to do with the air release orifice size which may be from 1 to 10 mm diameter. The larger the pressure in the pipeline, the smaller need be the orifice size. The volume of air to be released will be a function of the air entrained which is on the average 2% of the volume of water (at atmospheric pressure).

The small orifice release valves are sealed by a floating ball, or needle which is attached to a float. When a certain amount of air has accumulated in the connection on top of the pipe, the ball will drop or the needle valve will open and release the air. Small orifice release valves are often combined with large orifice air vent valves on a common connection on top of the pipe. The arrangement is called a double air valve. An isolating sluice valve is normally fitted between the pipe and the air valves.

Double air valves should be installed at peaks in the pipeline, both with respect to the horizontal and the maximum hydraulic gradient. They should also be installed at the ends and intermediate points along a length of pipeline which is parallel to the hydraulic grade line. It should be borne in mind that air may be dragged along in the direction of flow in the pipeline and may even accumulate in sections falling slowly in relation to the hydraulic gradient. Double air valves should be fitted every 1/2 to 1 KM along descending sections, especially at points where the pipe dips steeply.

Air release valves should also be installed all along ascending lengths of pipeline where air is likely to be released from solution due to the lowering of the pressure, again especially at points of decrease in gradient. Other places where air valves are required are on the discharge side of pumps and at high points on large mains and upstream of orifice plates and reducing tapers.

Air-Relief towers are provided at the first summit of the line to remove air that is mechanically entrained as water is drawn into the entrance of the pipeline.

6.16.3.2 Air Inlet Valves

In the design and operation of large steel pipelines, where gravity flow occurs, considerations must be given to the possibility of collapse in case the internal pressure is reduced below that of atmosphere. Should a break occur in the line at the lower end of a slope, a vaccum will in all probability be formed at some point upstream from the break due to the sudden rush of water from the line. To prevent the pipe from collapsing, air inlet (vaccum breaking) valves are used at critical points.

These valves, normally held shut by water pressure, automatically open when this pressure is reduced to slightly below atmosphere, permitting large quantities of air to enter the pipe, thus effectively preventing the formation of any vaccum. In addition to offering positive protection against extensive damage to large pipelines, by prevention of vacuum, they also facilitate the initial filling of the line by the expulsion of air wherever the valves are installed.

Air inlet valves should be installed at peaks in the pipeline, both relative to the horizontal and relative to the hydraulic gradient. Various possible hydraulic gradients, including reverse gradients during scouring, should be considered. They are normally fitted in combination with an air release valve.

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Often air release valves are used in conjunction with them, the purpose of these being to vent air accumulations that may occur at the peaks after the line has been put into operation. Please refer to 6.17.3 also for more information.

6.16.4 Kinetic Air Valves

In case of ordinary air valve, single orifice (small or large) type, the air or water from the rising main is admitted in the ball chamber of the air valve from one side of the ball. The disadvantages with this type of valve are that (a) once the ball goes up, it does not come down even when air accumulates in the ball chamber and (b) due to air rushing in, it stirs the ball making it stick to the upper opening which does not fall down unless the pressure in the main drops. The Kinetic air valve, overcomes these deficiencies since the air or water enters from the bottom side of the ball and the air rushing around ball exerts the pressure and loosens the contact with the top opening and allows the ball to drop down.

6.16.5 Pressure Relief Valves

These, also called as over-flow towers, are provided in one or more summits of the conveyance main to keep the pressure in the line below given value by causing water to flow to waste when the pressure builds up beyond the design value. Usually they are spring or weight loaded and are not sufficiently responsive to rapid fluctuations of pressure to be used as surge protection devices. The latter are dealt in 6.17.4.

6.16.6 Check Valves

Check valves, also called non-return valves or reflux valves, automatically prevent reversal of flow in a pipeline. They are particularly useful in pumping mains when positioned near pumping stations to prevent backflow, when pumps shut down. The closure of the valve should be such that it will not set up excessive shock conditions within the system. The remedial measures are discussed in 6.17.4. For more details of swing check reflux valves, reference may be made to IS 5312 - Pt I-1984 & Pt II-1986.

6.16.6.1 Dual Plate Check Valves

Dual plate check valves employ two spring loaded plates hinged on a central hinge pin. When the flow decreases, the plates close by torsion spring action without requiring reverse flow. As compared to conventional swing check valve which operates on mass movement, the Dual plate check valve are provided with accurately designed and tested torsion springs to suit the varying flow conditions. The Dual plate check valves are of non-slamming type and arrest the tendency of reversal of flow. Presently there is no IS for the Dual Plate Check Valves.

6.16.7 Surge Tanks

These are provided at the end of the line where water hammer is created by rapid closing of a valve and are discussed in detail in 6.17.

6.16.8 Pressure-Reducing Valves

These are used to automatically maintain a reduced pressure within reasonable limits in the downstream side of the pipeline. This type of valve is always in movement and requires scheduled maintenance on a regular basis. This work is facilitated if the valve is fitted on a bypass with isolating valves to permit work to proceed without taking the main out of service. If the pressure reducing valve is fitted on the main pipeline, a bypass can be provided for emergency use. Needle type valves which can be hydraulically controlled or motor operated with a pressure regulator are used for large aqueduct mains.

6.16.9 Pressure Sustaining Valves

Pressure sustaining valves are similar in design and construction to pressure reducing valves and are used to maintain automatically the pressure on the upstream side of the pipeline.

6.16.10 Ball Valves Or Ball Float Valves

Ball valves or ball float valves are used to maintain a constant level in a service reservoir or elevated tank or standpipe. The equilibrium type of valve is the most effective and it is designed to ensure that the forces on each side of the piston are nearly balanced. For severe operating conditions, a more expensive needle type of valve will give better service.

In both cases the float follows the water level in the reservoir and permits the valve to admit additional water on a falling level and less water on a rising level and to close entirely when the overflow level is reached. The disadvantage of this system is that the valve may operate for long periods in a throttled condition, but this can be avoided by arranging for the float to function in a small auxiliary cylinder or a tank. When the water reaches the top of the auxiliary tank, the ball will rise fairly quickly from the fully open position to the closed position without shock. The valve will not open again until the water level in the reservoir reaches the base of the auxiliary tank, at which point the water will drain away and the ball valve will move to the fully open position. With this method the valve is not in a state of almost continuous movement and throttling and erosion of the seats are avoided.

6.16.11 Automatic Shut-Off Valves

These are used on the mains to close automatically when the velocity in the mains exceeds a predetermined valve in case of accident to the line.

6.16.12 Automatic Burst Control

With large steel mains suitably protected against corrosion and laid properly, particularly at change of direction and the ground is not liable to subsidence, the possibility of a major burst is ruled out.

The simplest arrangement as explained in 6.16.14 is to insert an interrupter timer in the motor circuit so arranged that the final quarter travel of a sluice valve occurs in slow steps to the point of closure. The costlier arrangement will be insertion of a smaller power operated bypass valve alongside the main valve and provision of automatic control arrangements for the main valve to close first at a fairly rapid rate, followed by the smaller bypass valve at a much lower speed.

6.16.13 VENTURIMETERS

These are used to measure the flow in line and are discussed in 4.3.1.1.

6.16.14 SPACING OF VALVES AND INTERCONNECTIONS

The pipeline should be divided into sections by valves to avoid the necessity of emptying the whole pipeline in case of repair, each section being provided with an air valve and scouring facilities. The need for scour should be particularly borne in mind when layout of the pipeline and siting of the valves is finalized, as they cannot always be arranged in the best position due to likely difficulty in disposing of the discharge. They are necessary for scouring the mains and hence should be in proportion to the size of the main.

It is desirable to have valves close together in more densely built up areas. Ease of access to the valves is also important as the time taken in shutting of a valve in an emergency may be mostly spent in reaching it. In gravity mains, automatic valves, self-closing if pipe bursts, may also be provided for protection to property as well as to prevent excessive wastage of water.

Where there is more than one pipeline, they should be interconnected at each site of main valves, so that only shortest possible length of one pipeline need be put out of commission at a time. The interconnection will entail only negligible loss of head if its area is not less than two-thirds that of the largest main.

Also, when two or more mains are connected in parallel, the scours may be interconnected so that either main can be refilled from the other while the master valve is shut. Charging through a scour can be done speedily with less risk than charging over a summit, the danger of surging from trapped air being much reduced.

Bypasses around the main valves are convenient for regulating the flow during the charging or emptying of a pipeline and may be a part of the main valve itself, or arranged as a connecting between tees on each side of the valve. Bypasses may also be essential in order to balance up pressures on each side of the main valve before attempting to open it up.

6.16.15 MANHOLES

Access manholes are spaced 300 to 600m apart on large conduits. They are helpful during construction and serve later on for inspection and repairs. Their most useful positions are at summits, discharges and downstream of main valves. They are less common on cast iron and asbestos cement lines than on steel and concrete lines.

6.16.16 INSULATION JOINTS

They are used to introduce resistance to the flow of stray electric currents along metallic pipelines and may help in the control of electrolysis. Modern insulation joints make use of rubber gaskets or rings and of rubber covered sections of pipe if they are sufficiently long to introduce appreciable resistance.

6.16.17 EXPANSION JOINTS

Expansion joints are not needed if the pipe joints themselves take care of the pipe movements induced due to temperature changes, which is mostly the case for long buried pipes without any bend or dip. Steel pipes laid with rigid transverse joints particularly in the open, must either be allowed to expand at definite points or its motion be rigidly restrained by anchoring the line.

6.16.18 ANCHORAGES

Anchorages are necessary to resist the tendency of the pipes to pull apart at bends or other points of unbalanced pressure, or when they are laid on steep gradients and the resistance of their joints to longitudinal (shearing) stresses is either exceeded or inadequate. They are also used to restrain or direct the expansion and contraction of rigidly joined pipes under the influence of temperature changes. The unbalanced static pressure at ends computed by the expression $1/2\ \pi\ d^2\ p \sin \alpha/2$ with the two component pressures in the direction of each pipe leg being $1/4\ \pi d^2\ \alpha\ p$ (where d = dia of pipe, α = degree of bend and p the water pressure in the pipeline) is compared with the magnitude of the resistance of the pipe joint (which is 14.06 Kg/cm^2 for lead joint) and anchorages are designed to resist the balance force. Horizontal thrust F at Bend = $2\ A\ \beta \sin \alpha/2$, where β = internal pressure in Kg/cm^2, A area of pipe in sq cms, and α is angle of deviation of pipe in degrees.

Anchorages take many forms. For bends, both horizontal and vertical they may be designed as concrete buttresses or 'Kick blocks' that resist the unbalanced pressure by their weight, in much the same manner as a gravity dam resists the pressure of the water that it impounds. The resistance offered by the pipe joints themselves, by the friction of the pipe exterior and by the bearing value of the soil in which the block is buried may be taken into consideration if the cost of the block is to be a minimum. Steel straps attached to heavy boulders or to bedrock are used in place of buttresses where it is possible and convenient to do so.

The unbalanced thrust may be counteracted by longitudinal tension in an all-welded pipeline, or by a concrete thrust block bearing against the foundation material. In the case of a jointed pipeline the size of the block may be calculated using soil mechanics theory. In addition to frictional resistance on the bottom of the thrust block and the circumference of the pipeline, there is a lateral resistance against the outer face of the pipe and block. The maximum resisting pressure a soil mass will offer is termed the passive resistance and is given by

$$fp = \gamma_s \cdot h \left[\frac{1+\sin\theta}{1-\sin\theta}\right] + 2C\sqrt{\frac{1+\sin\theta}{1-\sin\theta}} \qquad (6.17)$$

The lateral resistance of soil against the thrust block

$$Fp = \gamma_s \frac{H^2}{2} L\left[\frac{1+\sin\theta}{1-\sin\theta}\right] + 2CHL\sqrt{\frac{1+\sin\theta}{1-\sin\theta}} \quad (6.18)$$

This maximum possible resistance will only be developed if the thrust block is able to move into the soil mass slightly. The corresponding maximum soil pressure is termed the passive pressure. The minimum pressure which may occur on the thrust block is the active pressure, which may develop if the thrust block were free to yield away from the soil mass.

$$fa = \gamma_s h\left[\frac{1-\sin\theta}{1+\sin\theta}\right] - 2C\sqrt{\frac{1-\sin\theta}{1+\sin\theta}} \quad (6.19)$$

F_P, f_p = Lateral resistance of soil against the thrust block in tons,

f_a = Lateral resistance of soil against the projection of pipe in tons/m^2,

γ_s = soil density in T/m^3,

h = depth in m,

θ = angle of friction in degrees,

C = cohesion of soil in N/m^2,

(C = 0 for gravel and sand, 0.007 for silt, 0.035 for dense clay, and 0.15 for soft saturated clay),

H = height of thrust block in m,

L = the length of thrust block in m,

The active pressure is considerably less than the passive pressure and will only be developed if the force on which it is acting is free to move away from the soil exerting the pressure.

A thrust block should be designed so that the line of action of the resultant of the resisting forces coincides with the line of thrust of the pipe. This will prevent overturning or unbalanced stresses. This may best be done graphically or by taking moments about the centre of the pipe. Anchor blocks for expansion joints can also be designed on the basis of I.S.5330-1984.

Thrust blocks are needed not only at changes in vertical or horizontal alignment of the pipeline, but also, at fittings that may not be able to transmit longitudinal forces such as flexible couplings.

When laying a pipe parallel to an existing pipe, the trench excavation for a bend would deprive the existing pipe of the needed support. The simplest solution is to stop the flow in the original pipe while the work is carried out and a new thrust block is constructed, but where this is not possible, one alternative is to anchor the thrust block of the original pipe to an anchor block of concrete by means of steel bars. Another method is to provide additional support by piling on the outside of the bend before excavation commences. It is advisable to avoid sharp bends above 45^0 and in soft ground it is better not to put two bends together but to separate them by atleast a length of straight pipe. Cast iron pipes and fittings can be cast with lugs through which tie rods are passed when it is desired to prevent movement of the pipe. Where steel pipes with welded joints are used, full anchorage is not generally necessary, since the longitudinal continuity of the pipe is capable of distributing the forces into the ground. If the pressures are high enough to merit it and sleeve joints are being used, the joints on the bends and on two pipes either side of them should be fully welded inside and outside and the trench refilled with concrete to 150mm above these pipes and bends. About half the thrust will be taken by the weight of the concrete and the remainder by the longitudinal stress in the pipes. In order to restrain the motion of steel pipes or force it to take place at expansion joints that have been inserted for that purpose, the pipe may be anchored in much the same way as described for bends. Due attention must be paid to the bounding of the pipe to the anchors. Pipes laid on steep inclines should be anchored by transversed blocks or other precautions taken to prevent slippage and measures to overcome unbalanced pressures provided.

In the absence of expansion joints, steel pipes must be anchored at each side of gates and meters in order to prevent their destruction. Where gate chambers are used, they may be so designed, of steel and concrete, that they hold the two ends of the steel line rigidly in place. In the absence of anchors, flanged gates are sometimes bolted on one side and the other side to a cast iron nipple that is connected to the pipe by means of a sleeve or expansion joint. See Figure 6.1 for typical Thrust Block and Appendix 6.6 for a worked out example.

6.17 WATER HAMMER

6.17.1 OCCURENCE

If the velocity of water flowing in a pipe is suddenly diminished, the energy given up by the water will be divided between compressing the water itself, stretching the pipe walls and frictional resistance to wave propagation. This pressure rise or water hammer is manifest as a series of shocks, sounding like hammer blows, which may have sufficient magnitude to rupture the pipe or damage connected equipment.

It may be caused by the nearly instantaneous or too rapid closing of a valve in the line, or by an equivalent stoppage of flow such as would take lace with the sudden failure of electricity supply to a motor driven pump. The shock pressure is not concentrated to the valve and if rupture occurs, it may take place near the valve simply because it acts there first. The pressure wave due to water hammer travels back upstream to the inlet end of the pipe, where it reverses and surges back and forth through the pipe, getting weaker on each successive reversal. The velocity of the wave is that of acoustic wave in an elastic medium, the elasticity of the medium in this case being a compromise sound between that of the liquid and the pipe. The excess pressure due to water hammer is additive to the normal hydrostatic pressure in the pipe and depends on the elastic properties of the liquid and pipe and on the magnitude and rapidity of change in velocity. Complete stoppage of flow is not necessary to produce water hammer, as any sudden changes in velocity will create it to a greater or lesser degree depending on the conditions mentioned above.

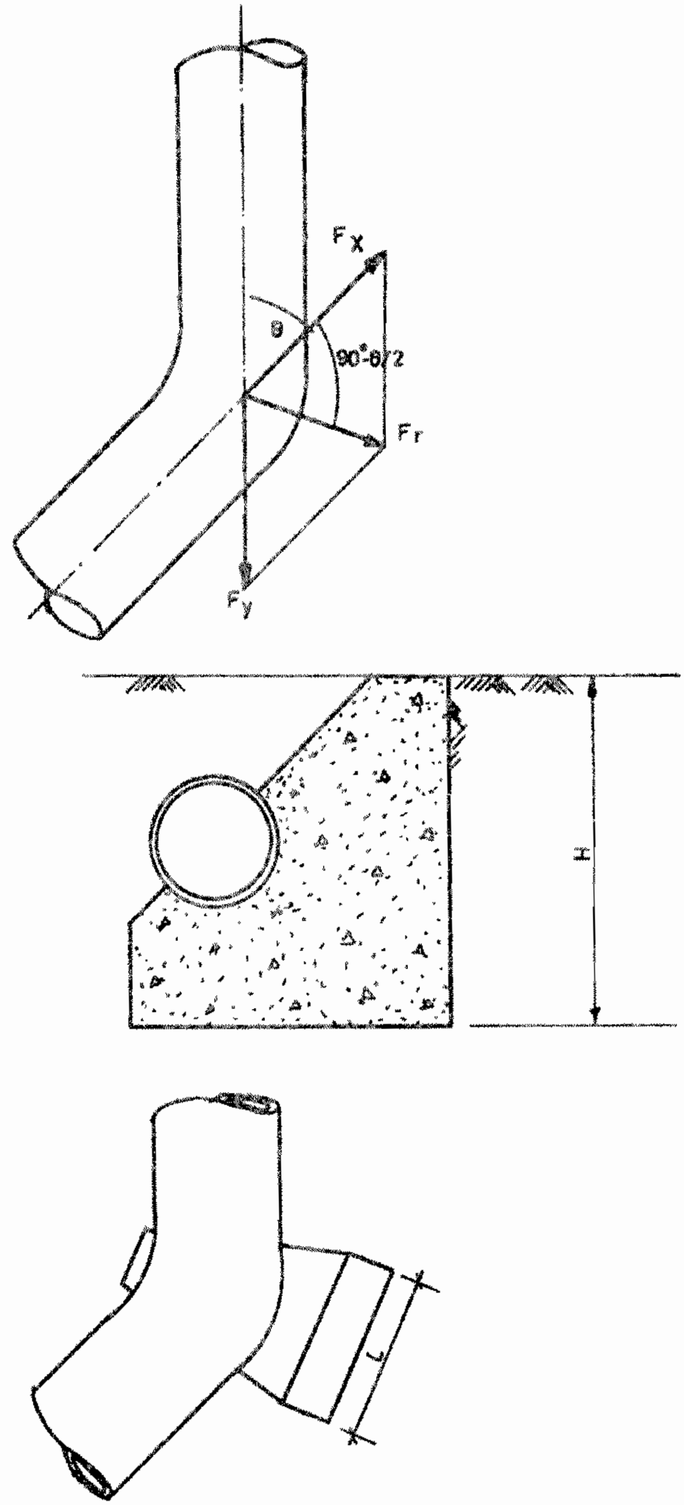

FIGURE 6.1 : THRUST AT A BEND & THRUST BLOCK

6.17.2 COMPUTATIONS

Maximum water hammer pressure (which occur at the critical time of closure T_c or any time less than T_c) is given by the expression,

$$Hmax = \frac{C.\ V_0}{g} \tag{6.20}$$

Where,

H_{max} = maximum pressure rise in the closed conduit above the normal pressure in m,

C = velocity of pressure wave travel in m/s,

g = acceleration due to gravity in m/s^2

V_0 = normal velocity in the pipeline, before sudden closure in m/s

$$C = \frac{1425}{\sqrt{1 + \frac{kd}{EC_t}}} \tag{6.21}$$

Where,

k = bulk modulus of water (2.07×10^8 kg/m^2)

d = diameter of pipe in m,

C_t = wall thickness of pipe in m and

E = modulus of elasticity of pipe material in kg/m^2

Table 6.7 gives values of E that may be adopted for different materials:

TABLE 6.7 : VALUES OF E FOR DIFFERENT MATERIALS

Material	E (Kg/m^2)
Polyethylene – soft	1.2×10^7
Polyethylene – hard	9×10^7
P V C	3×10^8
Concrete	2.8×10^9
Asbestos Cement	3×10^9
Reinforced Cement Concrete	3.1×10^9
Prestressed Concrete	3.5×10^9
Cast Iron	7.5×10^9
Ductile Iron	1.7×10^{10}
Wrought Iron	1.8×10^{10}
Steel	2.1×10^{10}

If the actual time of closure T is greater than the critical time T_c , the actual water hammer is reduced approximately in proportion to T_c/T.

Water hammer wave velocity may be as high as 1370 m/s for a rigid pipe or as low as 850 m/s for a steel pipe and for plastic pipes may be as low as 200 m/s.

6.17.3 Control Measures

The internal design pressure for any section of a pipeline should not be less than the maximum operating pressure or the pipeline static pressure obtaining at the lowest portion of the pipeline considered including any allowance required for surge pressure. The maximum surge pressure should be calculated and the following allowances made.

(a) If the sum of the maximum operating pressure or the maximum pipeline static pressure whichever is higher and the calculated surge pressure does not exceed 1.1 times the internal design pressure, no allowance for surge pressure is required,

(b) If the sum exceeds 1.1 times the internal design pressure, then protective devices should be installed and

(c) In no case the sum of the maximum operating pressure and the calculated surge pressure should exceed the field hydrostatic test pressure.

Depending upon the layout of the plant, the profile and the length of the pipeline, surging in pipelines can be counteracted in two fundamentally different ways (1) by checking the formation of the initial reduced pressure wave itself by means of flywheels (which lengthen the slowing down time of the pump) and air vessels (which continues to feed water into the pipeline until the reflected pressure wave again reaches the pump) and (2) by neutralizing the reflected wave from the reservoir by installing special devices in the pipelines, some of which are automatically controlled quick closing valves, automatically controlled bypasses and pressure relief valves. To obtain greatest effectiveness, the relief valve or other form of suppressor should be located as close as possible to the source of disturbance.

Since the maximum water hammer pressure in metres is about 125 times the velocity of flow in mps and the time of closure of gate valves varies inversely with the size of the main, water hammer is held within bounds in small pipelines by operating them at moderate velocities of 1 to 2 mps. In larger mains, the pressure is held down by changing velocities at sufficiently slow rate so that the relief valve returns to position of control before excessive pressures are reached. If this is not practicable, pressure relief or surge valves are used. For mains larger than 1.75m, which operate economically at relatively high velocities of 2 to 3 mps and cannot be designed to withstand water hammer without prohibitive cost, the energy is dissipated slowly by employing surge tanks. In its simplest form, a surge tank is a standpipe placed at the end of the line next to the point of velocity control. If this control is a gate, the surge tank accepts water and builds up backpressure when velocities are regulated downward. When the demand on the line increases, the surge tank affords an immediate supply of water and, in so doing, generates the excess hydraulic gradient needed to accelerate the flow through the conduit following a change in the discharge rate. The water level in a surge tank oscillates slowly till the excess energy is dissipated by hydraulic friction through the system.

6.17.3.1 Causes Of Water Hammer And Remedial Measures

The three common causes of water hammer encountered in water supply systems are: (1) rapid closure of valves (2) sudden shut off or unexpected failure of power supply to centrifugal pumps and (3) pulsation problems due to hydraulic rams and reciprocating pumps.

6.17.3.2 Rapid Closure Of Valves

Gate valves are to be preferred to stop valves. The valve closure period should be slowed down to take longer than the critical time of closure. The first 80% of valve travel can be executed as quickly as convenient, but the last 20% (which is effective in shutting off approximately 80% of the flow) should be done as deliberately as possible. This not only tends to minimize water hammer but is expedient owing to the greater resistance to closure offered as shut-off is approached. Where power driven operating devices are used, similar precautions should be taken. For geared gate valves, closure may have to be considerably slower in the initial period than in the case of valves without gears; the mechanical advantage available is of great assistance in effecting the last 20% of closure, particularly with large gate valves at high rates of flow. Similar measures have to be adopted for prevention of rapid opening of valves. By passes are help in closing or opening of large valves and should be closed last. Care should also be taken to avoid setting up excessive water hammer through too rapid operation of fire hydrants.

6.17.3.3 Remedial Measures For Sudden Shut Off Of Pumps

When the power supply to centrifugal pump is suddenly shut off for some reason or fails unexpectedly, severe water hammer may be set up in either the pump discharge or suction piping or both, depending upon the layout due to the momentum of the column of water flowing through the pipe which tries to continue towards its destination even after the power interruption.

(a) Shut- off Effects on Discharge Line

Immediately after interruption, the impeller slows down and the column of water coasts along the discharge pipe away from the pump with an ensuing drop in pressure at the pump. The column then slows down and reverses its direction of flow so as to come back towards the point of low pressure at the pump. If there is a check valve at the pump, on continued reversal of flow is possible and a back pressure builds up against the check valve which, in general, will be about equal to the preceding drop below normal. The shock pressure may reach twice the normal head if there has been no breaking of the water column or, if the column has broken, the pressure may rise to a much higher value than twice the normal head. Increased flywheel capacity of a pump will, in cases of power failure, maintain pumping action to an extent sufficient to prevent excessive fall of pressure.

In some cases a surge relief is called for, at or near the outlet end of the line. Depending on the size of the line and the pressures involved, this may be an air chamber, a relief valve, or an open overflow which lets the oncoming water spill out of the pipeline above some pre-determined elevation. Although provision for surge at the outlet end of the line will not necessarily prevent all reversal of flow, it does tend to cushion shock there and at the same time decreases the magnitude of the reversal that is thrown back toward the pump.

Two devices can be provided for cushioning shock in the discharge line at a pump. The first concerns the check or other form of non-return valve at the pump. The ordinary swing-check valve tends to salm shut on reversal of flow, thus causing unnecessarily severe shock pressure. This trouble can be reduced by using a non-slam filling-disc check-valve, or some form of power operated valve which is controlled by a relay actuated from the power circuit or discharge pressure at the pump. Immediately on power interruption, the relay acts to start closing the valve whose operating mechanism can be timed to complete closure before reversal of flow can take place. In some cases a spring closing device can be used successfully on a swing-check valve to ensure having the tap close before back surge can start.

Although the aforesaid devices obviate the slamming to be expected with an ordinary swing check, they cannot prevent a considerable rise in pressure when reveral of flow is stopped at the closed valve. Hence, a sound device in the nature of a supplement is required which may take the form of an air chamber or a relief valve of ample size. Air chambers sometimes perform more effectively when damped with orifices or check-valves in the connecting pipe.

The necessity of replenishing the air in air chambers should be recognized in considering their use as water hammer suppressors. In some cases, restricting the passages between the pipeline and the air chambers increases the effectiveness of a given size of air chamber. Suppressors as a general rule, do not eliminate shock entirely but will reduce it by 10% to 40%, which often is sufficient to remove the clanking sound.

A pressure vessel with air cushion can serve as an automatic water accumulator. The effective volume that can be taken from the vessel depends on the switching on and switching-off pressures. Owing to the fact that water absorbs some of the compressed air that forms the aircushion, fresh air has to be introduced into the vessel from time to time. This can be done by means of a small compressor or, in the case of small units, a self-priming pump which is capable of dealing with water and air, the latter entering through a small adjustable intake in the pump suction branch.

The effective capacity of a pressure reservoir necessary for an automatic pumping plant is governed by the permissible switching frequency of the electric equipment and by the pump capacity. As a rule, the pump capacity must be such as to cover, by itself, the highest consumption expected.

Pumps with steep head/flow characteristic often induce high starting pressures when the power is switched on. This is because the flow is small (or zero) when the pump is switched on, so a wave with a head equal to the closed valve head is generated.

By partly closing the pump delivery valves during starting, the starting pressures can be reduced.

If the pumps supplying an unprotected pipeline are stopped suddenly the flow will also stop. If the pipeline profile is relatively close to the hydraulic grade line, the sudden deceleration of the water column may cause the pressure to drop to a value less than atmospheric pressure. The lowest value to which pressure could drop is vapour pressure. Vaporization or even water column separation may thus occur at peaks along the pipeline. When the pressure wave is returned as a positive wave the water columns will rejoin giving rise to water hammer over pressures.

Unless some method of water hammer protection is installed, a pumping pipeline system will normally have to be designed for a water hammer head. This is often done with high pressure lines where water hammer heads may be small in comparison with the pumping head. For short lines this may be an economic solution. Suitable locations for various water hammer protection devices are shown in Fig.6.2.

The philosophy behind the design of most methods of protection against water hammer is similar. The objective in most cases is to reduce the down surge in the pipeline caused by stopping the pumps. The upsurge will then be correspondingly reduced, or may even be entirely eliminated. The most common method of limiting the downsurge is to feed water into the pipe as soon as the pressure tends to drop.

The sudden momentum change of the water column beyond the tank is prevented so the elastic water hammer phenomenon is converted to a slow motion surge phenomenon. Part of the original kinetic energy of the water column is converted into potential energy instead of elastic energy. The water column gradually decelerates under the effect of the difference in heads between the ends. If it is allowed to decelerate the water column would gather momentum in the reverse direction and impact against the pump to cause water hammer overpressures. If, however, the water column is arrested at its point of maximum potential energy, which coincides with the point of minimum kinetic energy, there will be no sudden change in momentum and consequently no water hammer overpressure. The reverse flow may be stopped by installing a reflux valve or throttling device at the entrance to the discharge tank or air vessel, or in the pipeline. A small orifice bypass to the reflux valve would then allow the pressures on either side to gradually equalize.

Charts are available for the design of air vessels and for investigation of the pump inertia effects, so that a water hammer analysis is not normally necessary. Rigid water column theory may be employed for the analysis of surge tank action, and in some cases, of discharge tanks.

If the pipeline system incorporates in line reflux valves or a pump by pass valve, an elastic water hammer analysis is usually necessary. The analysis may be done graphically or, if a number of solutions of similar systems are envisaged, a computer program could be developed. Normally the location, size and discharge characteristics of a protective device such as a discharge tank have to be determined by trial and error. The location and size of inline or bypass reflux valves may similarly have to be determined by trial. In these instances a computer program is usually the most economical method of solution, as a general program could be developed and by varying the design parameters methodically, an optimum solution arrived at.

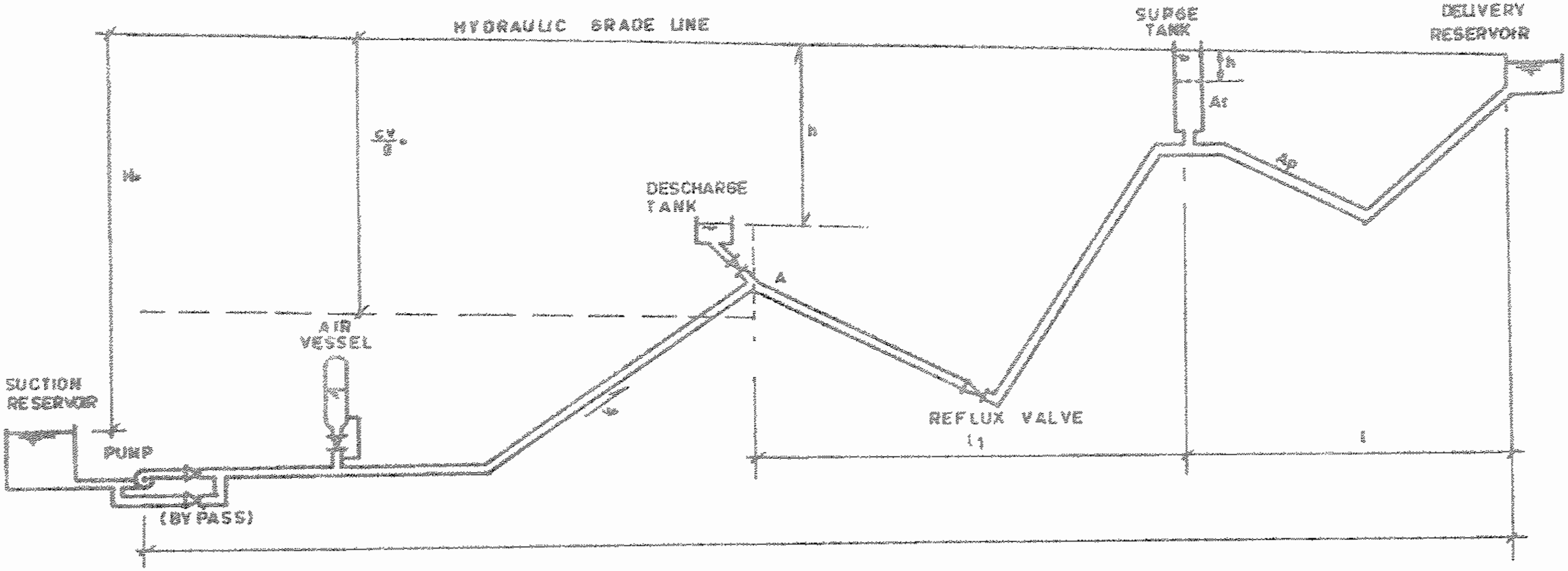

FIG.6.2 : PIPELINE PROFILE ILLUSTRATTING SUITABLE LOCATION FOR VARIOUS DEVICES FOR WATE HAMMER PROTECTION

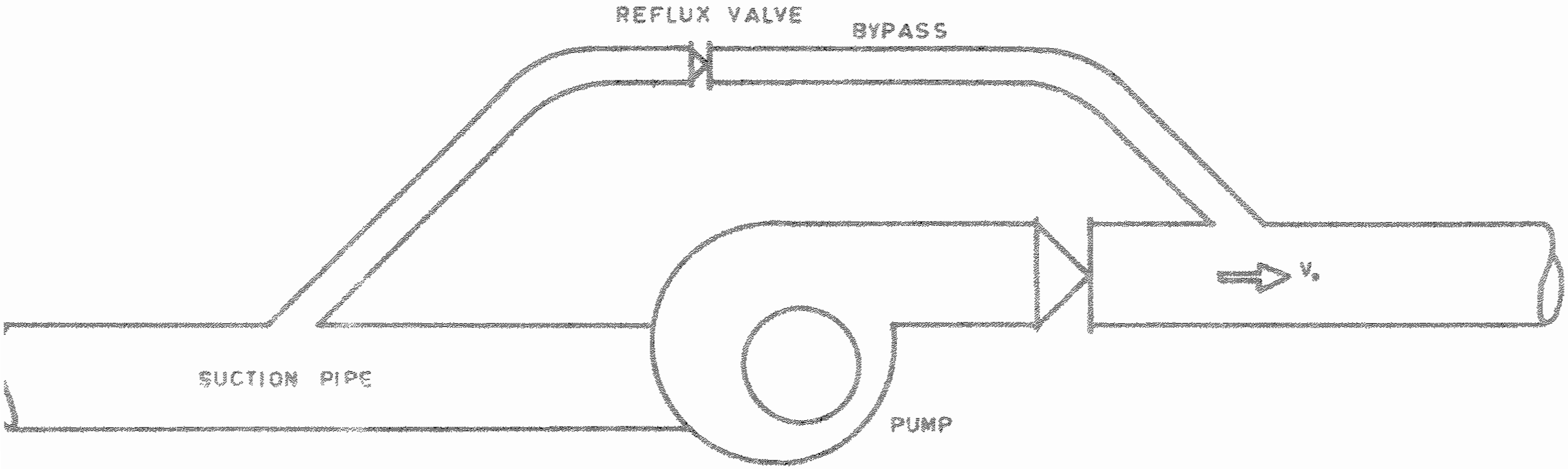

FIG.6.3: PUMP WITH BYPASS REFLUX VALUE

If the rotational inertia of a centrifugal pump and motor continue to rotate the pump for while after power failure, water hammer pressure transients may be reduced. The rotating pump, motor and entrained water will continue to feed water into the potential vacuum on the delivery side, thereby alleviating the sudden deceleration of the water column. The effect is most noticeable on low head, short pipelines.

After the power supply to the motor is cut off, the pump will gradually slow down until it can no longer deliver water against the delivery head existing at the time. It the delivery head is still higher than the suction head it will then force water through the pump in the reverse direction, with the pump still spinning in the forward direction, provided there is no reflux or control valve on the delivery side of the pump. The pump will rapidly decelerate and gather momentum in the reverse direction, and will act as a turbine under these conditions. The reverse speed of the pump will increase until it reaches runaway speed. Under these conditions there is a rapid deceleration of the reverse flow and water hammer overpressures will result.

If there is a reflux valve on the delivery side of the pump, the reverse flow will be arrested, but water hammer overpressures will still occur. The pressure changes at the pump following power failure may be calculated graphically or by computer.

The upsurge could be reduced considerably if reverse flow through the pump was permitted. If flow reversal was prevented, the maximum head-rise above operating head (H) would be approximately equal to the lowest head-drop below H_0.

A simple rule of the thumb for ascertaining whether the pump inertia will have an effect in reducing the water hammer pressures is:

If the inertia parameter $I = MN^2/WALH_0^2$ exceeds 0.01, the pump inertia may reduce the down surge by at least 10%. Here M is the moment of inertia of the pump, N is the speed in rpm and AL is the volume of water in the pipe.

Some installations have a flywheel fitted to the pump to increase the moment of inertia. In most cases the flywheel would have to be impracticably heavy, also it should be borne in mind that starting currents may thereby be increased. The effect of pump inertia can be neglected and the pumps assumed to stop instantaneously.

(i) Pump Bypass Reflux Valve

One of the simplest arrangements for protecting a pumping main against water hammer is a reflux valve installed in parallel with the pump (Fig. (6.3)). The reflux or non-return valve would discharge only in the same direction as the pumps. Under normal pumping conditions the pumping head would be higher than the suction head and the pressure difference would maintain the reflux valve in a closed position. On stopping the pumps, the head in the delivery pipe would tend to drop below the suction head, in which case water would be drawn through the bypass valve. The pressure would therefore only drop to the suction pressure less any friction loss in the bypass. The return wave over pressure would be reduced correspondingly. Fig. 6.4. gives the maximum and minimum head at pump after power failure.

This method of water hammer protection cannot be used in all cases, as the delivery pressure will often never drop blow the suction pressure. In other cases there may still be an appreciable water hammer overpressure (equal in value to the initial drop in pressure). This method is used only when the pumping head is considerably less than cv_0/g. In addition, the initial drop in pressure along the entire pipeline length should be tolerable. The suction reservoir level should also be relatively high or there may still be column separation in the delivery line.

Normally the intake pipes draw directly from a constant head reservoir. However, there may be cases where the intake pipe is fairly long and water hammer could be a problem in it too. In these cases a bypass reflux valve would, in a similar way to that described above, prevent the suction pressure exceeding the delivery pressure.

Water may also be drawn through the pump during the period that the delivery head is below the suction head, especially if the machine was designed for high specific speeds, as is the case with through flow pumps. In some cases the bypass reflux valve could even be omitted, although there is normally a fairly high head loss through a stationary pump. A constant bleeder line led off to the suction reservoir with a smaller diameter pipeline can also be connected to the pump outlet after the sluice valve to reduce the water hammer effects. This may result in wastage of energy.

(ii) Surge Tanks

The water surface in a surge tank is exposed to atmospheric pressure, while, the bottom of the tank is open to the pipeline. The tank acts as a balancing tank for the flow variations that may occur, discharging in case of a head drop in the pipe, or filling in case of a head rise. Surge tanks are used principally at the head of turbine penstocks, although there are cases where they can be applied in pumping systems. It is seldom that the hydraulic grade line of a pumping line is low enough to enable an open tank to be used. It may be possible to construct a surge tank at a peak in the pipeline profile and protect the pipeline between the pumps and the tank against water hammer by some other means. If the surge tank is relatively large, it could be treated as the discharge end of the intermediate pipeline length and this section could be treated as an independent pipeline shorter in length than the original pipeline.

The fluctuations of the water surface level in a surge tank following power failure may be studied analytically. The fluctuations in tank level may be dampened with a throttling orifice. In this case the pressure variations in the line may be more extreme than for the unrestricted orifice. The maximum heads at pump after power failure is presented in Fig. 6.4. A differential surge tank includes a small diameter riser in the middle of the tank. The tank may have a varying cross section or multiple shafts. Such variations are more applicable to hydropower plant than pumping systems as they are useful for dampening the surges in cases of rapid load variation on turbines.

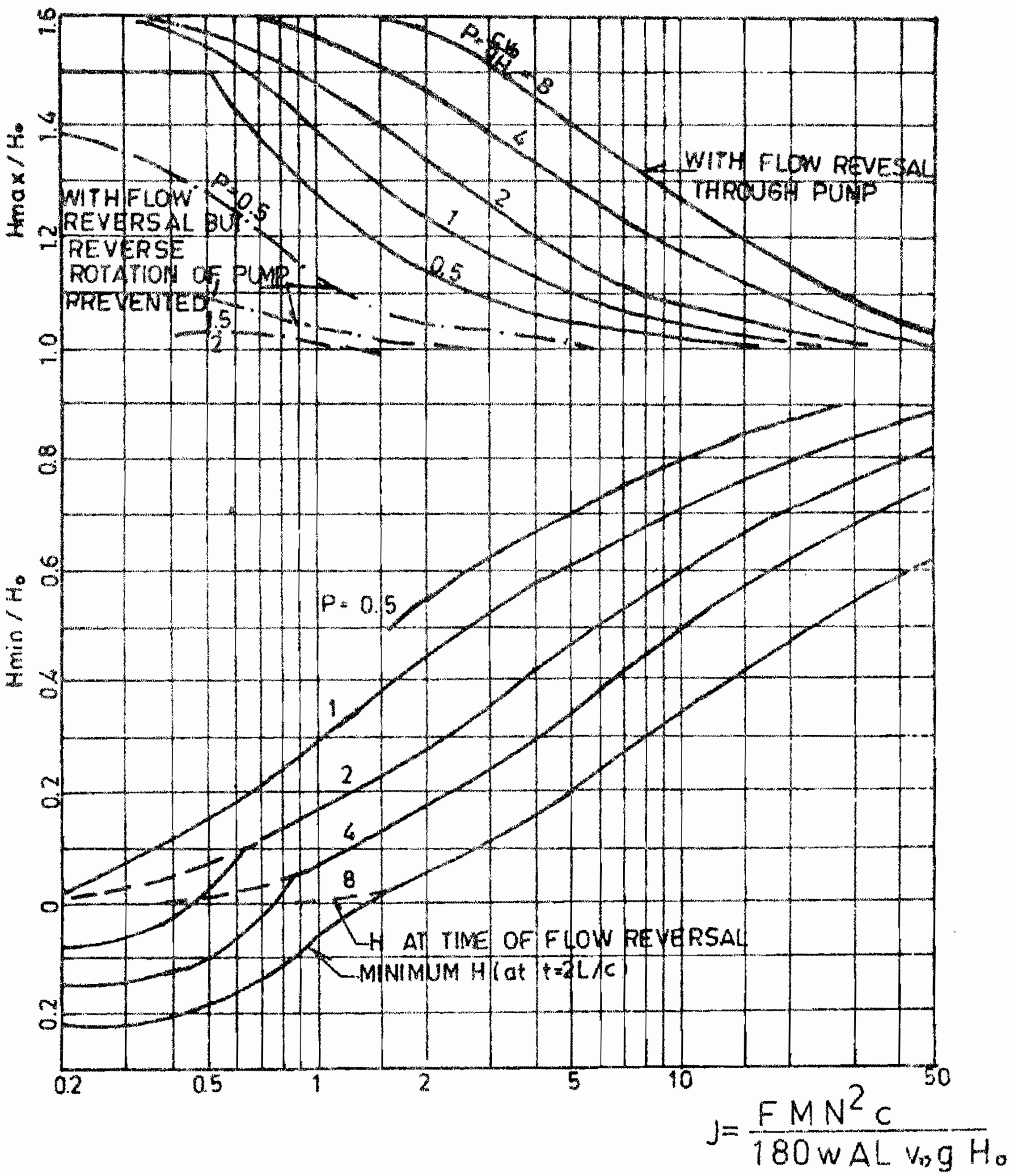

FIG. : 6.4 MAXIMUM AND MINIMUM HEADS AT PUMPS AFTER POWER FAILURE

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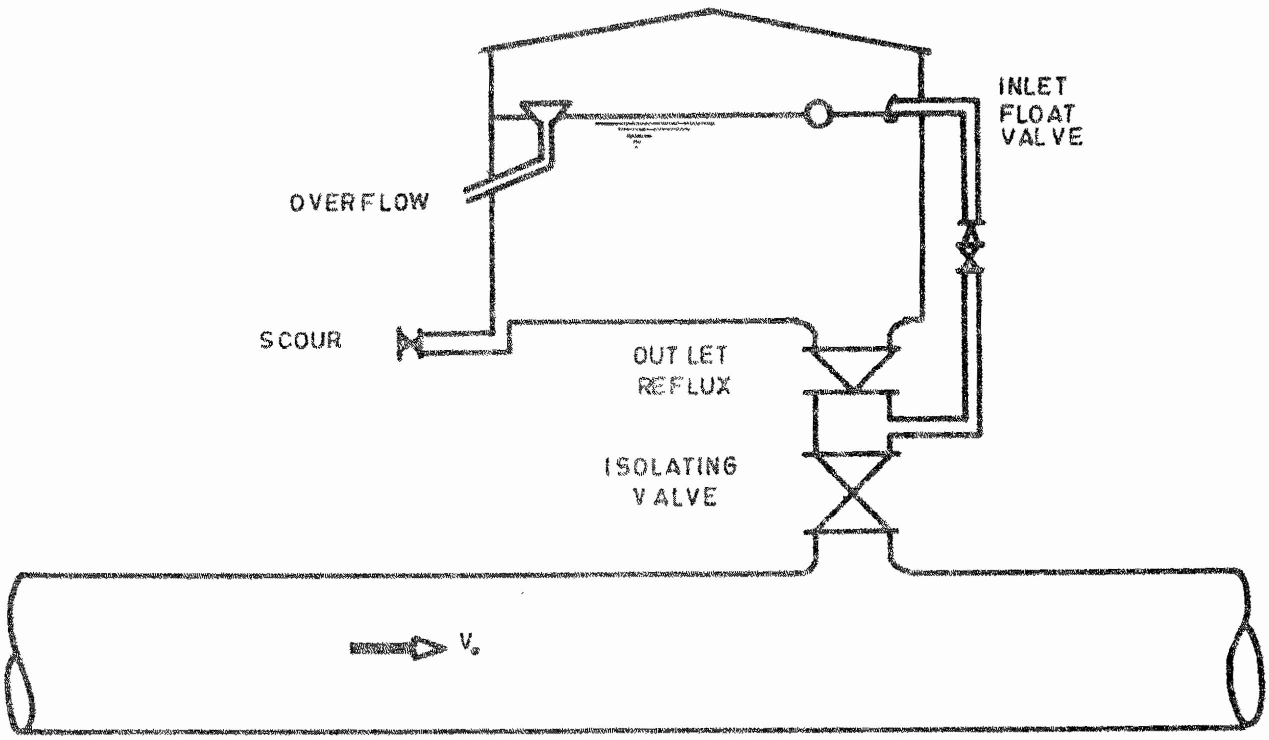

FIG. 6.5 : DISCHARGE TANK

(iii) Discharge Tanks

In situations where the pipeline profile is considerably lower than the hydraulic grade line it may still be possible to use a tank, but one which under normal operating conditions is isolated from the pipeline. The tank water surface would be subjected to atmospheric pressure but would be below the hydraulic grade line, as opposed to that of a surge tank.

A discharge tank would normally be situated on the first rise along the pipeline and possibly on subsequent and successively higher rises. The tank will be more efficient in reducing pressure variations, the nearer the level in the tanks is to the hydraulic grade line. It should be connected to the pipeline via a reflux valve installed to discharge from the tank into the pipeline if the pipeline head drops below the water surface elevation in the tank. Normally the reflux valve would be held shut by the pressure in the pumping line. A small bore bypass to the reflux valve, connected to a float valve in the tank, should be installed to fill the tank slowly after it has discharged. Fig. 6.5 depicts a typical discharge tank arrangement.

The function of a discharge tank is to fill any low pressure zone caused by pump stoppage, thus preventing water column separation. The water column between the tank and the discharge end of the pipeline (or a subsequent tank) will gradually decelerate under the action of the head differences between the two ends. It may be necessary to prevent reverse motion of the water column which could cause water hammer over pressures by installing a reflux valve in the line.

A discharge tank will only operate if the water surface is above the lowest level to which the head in the pipeline would otherwise drop following pump stoppage. For very long pipelines with a number of successively higher peaks, more than one discharge tank may be installed along the line. The tanks should be installed at the peaks where water column separation is most likely. The lowest head which will occur at any point beyond a tank as the down surge travels along the line is that of the water surface elevation of the preceding tank.

The best position for discharge tanks and inline reflux valves is selected by trial and error and experience. In a case with many peaks or major pipelines with large friction heads, a complete analysis should be carried out, either graphically or by computer. In particular, a final check should be done for flows less than the maximum design capacity of the pipeline.

Even though a number of tanks may be installed along a pipeline, vaporization is always possible along rising sections between the tanks. Provided there are no local peaks, and the line rises fairly steeply between tanks, this limited vaporization should not lead to water hammer overpressures.

6.17.4 AIR VESSELS

If the profile of a pipeline is not high enough to use a surge tank or discharge tank to protect the line, it may be possible to force water into the pipe behind the low-pressure wave by means of compressed air in a vessel. The pressure in the vessel will gradually decrease as water is released until the pressure in the vessel equals that in the adjacent line. At this stage the decelerating water column will tend to reverse. However, whereas the outlet of the air vessel should be unrestricted, the inlet should be throttled. A suitable arrangement is to have the water discharge out through a reflux valve that shuts when the water column reverses. A small orifice open bypass would allow the vessel to refill slowly. (Fig 6.6)

A rational design of air vessel involves calculation of the dimensionless parameters, as follows:

$$\text{Pipeline parameter} = \rho = CV_0 / 2gH_0 \qquad (6.22)$$

$$\text{Air vessel parameter} = \rho \frac{2C_0\, C}{Q_0\, L} \qquad (6.23)$$

Kc = Coefficient of Head Loss such that $K_c H_0$ is the total head loss for a flow of Q_0 into air vessel. (Ref. to Fig. 6.7 to 6. 10) C is water hammer wave velocity, V_0 is initial velocity and H_0 is absolute head (including atmospheric head), C_0 is the volume of Air, L is the length of pipeline.

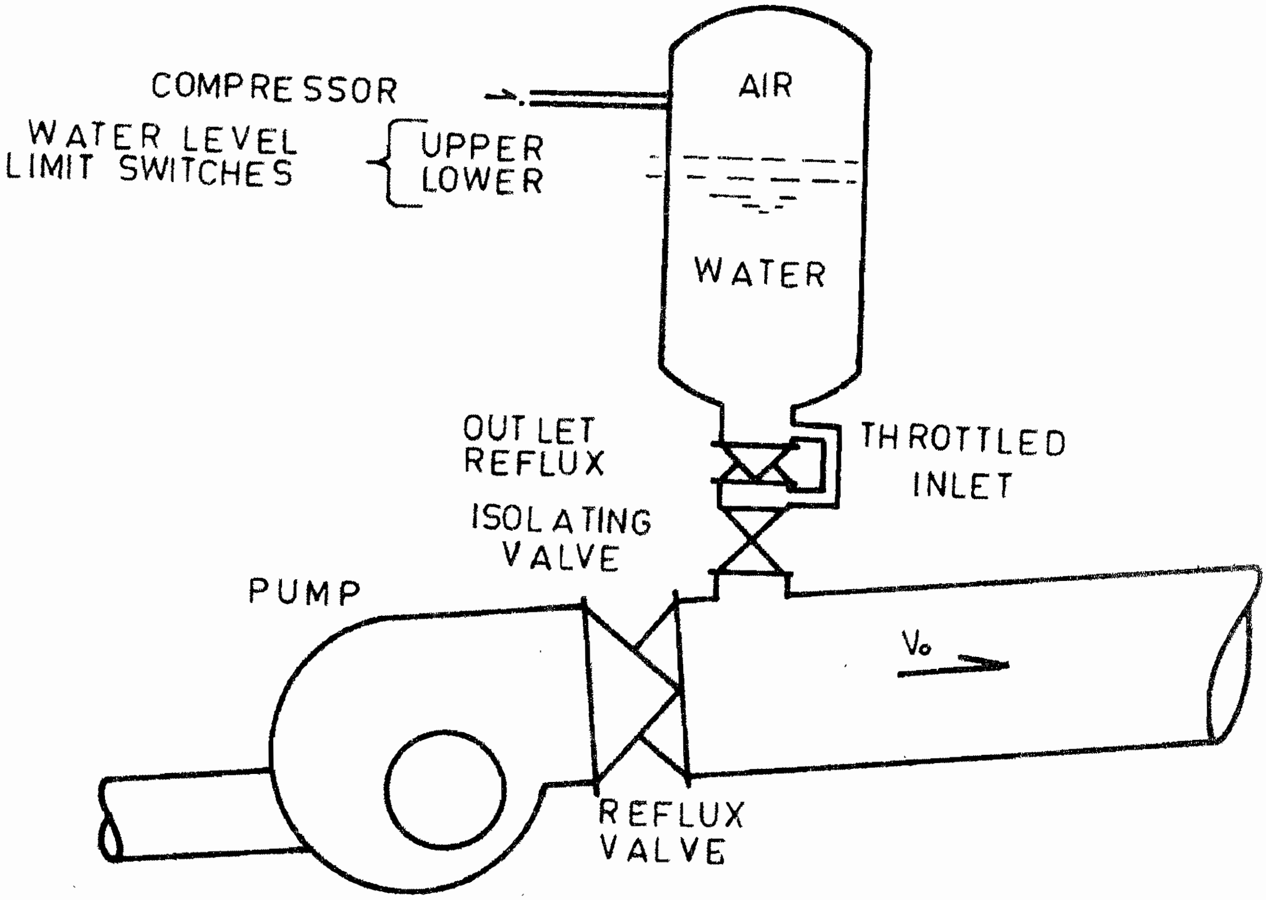

FIGURE 6.6 : AIR VESSEL

6.17.4.1 Design Of Air Vessel

The pipeline parameter, C is calculated from the maximum likely line velocity and pumping head, and the corresponding chart selected from Figs. 6.7 to 6. 10 for an assumed K_C value i. e. 0.0, 0.3, 0.5 or 0.7. The value of Air Vessel parameter corresponding to the selected line is used to read off the maximum head envelope along the pipeline from the same chart.

The volume of air, Co, is calculated once the air vessel parameter is known. The vessel capacity should be sufficient to ensure no air Where escapes into the pipeline, and should exceed the maximum air volume. This is the volume during minimum pressure conditions and is $S(H_0/H_{mm})^{1/1.12}$.

The outlet diameter is usually designed to be about one-half the main pipe diameter. The outlet should be designed with a bellmouth to suppress vortices and entertainment. The air in the vessel will dissolve in the water to some extent and will have to be replenished by means of a compressor.

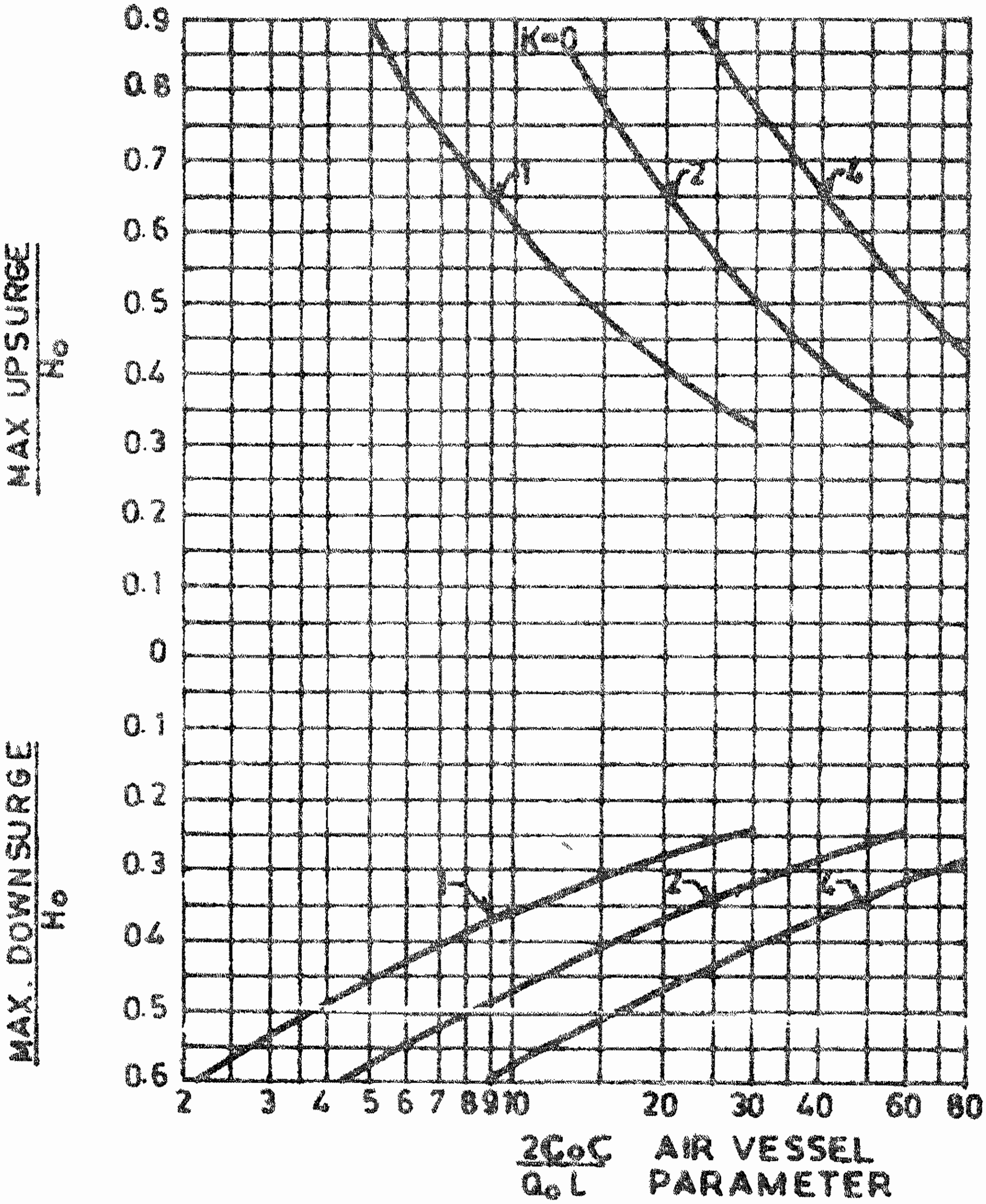

FIGURE 6.7 SURGES IN PUMP DISCHARGE LINE, $K_c = 0$

The requisite effective capacity of the vessel is also calculated from the expression.

$$V_0 = 1200 \text{ to } 1500\ Q/Z_P \quad (6.24)$$

Where,

V_0 = effective volume in litres.

Q = discharge of pumps in lps and

Zp = permissible number of switching operation per hour for three-phase motors:

(10-15 for squirrel-cage motors direct in line,

6-10 for squirrel-cage motors with star delta starter,

6-10 for motors with rotor starter,

Permissible number of starts for motors as per IS 325 is 3.

A worked out example is at Appendix 6.7)

6.17.4.2 In-Line Reflux Valves

Inline reflux valves would normally be used in conjunction with surge tanks, discharge tanks or Air vessels. Following pumps shutdown, the tank or vessel would discharge water into the pipe either side of the reflux valve. This would alleviate the violent pressure drop and convert the phenomenon into a slow motion effect. The reflux valve would then arrest the water column at the time of reversal, which coincides, with the point of minimum kinetic energy and maximum potential energy of the water column. There would therefore be little momentum change in the water column when the reflux valve is shut and consequently negligible water hammer pressure rise.

There are situations where water column separation and the formation of vapour pockets in the pipeline following pump stoppage would be tolerable, provided the vapour pockets did not collapse resulting in water hammer pressures. Reversal of the water column beyond the vapour pocket could in fact be prevented with an in-line reflux valve at the downstream extremity of the vapour pocket. The water column would be arrested at its point of minimum momentum, so there would be little head rise.

Vaporization would occur at peaks in the pipeline where the water hammer pressure drops to the vapour pressure of the water. If the first rise along the pipeline was higher than subsequent peaks, the vaporization would be confined to the first peak.

In locating the reflux valve, allowance should be made for some lateral dispersion of the vapour pocket. The valve should be installed at a suitable dip in the pipeline in order to trap the vapour pocket and to ensure proper functioning of the valve doors when the water column returns.

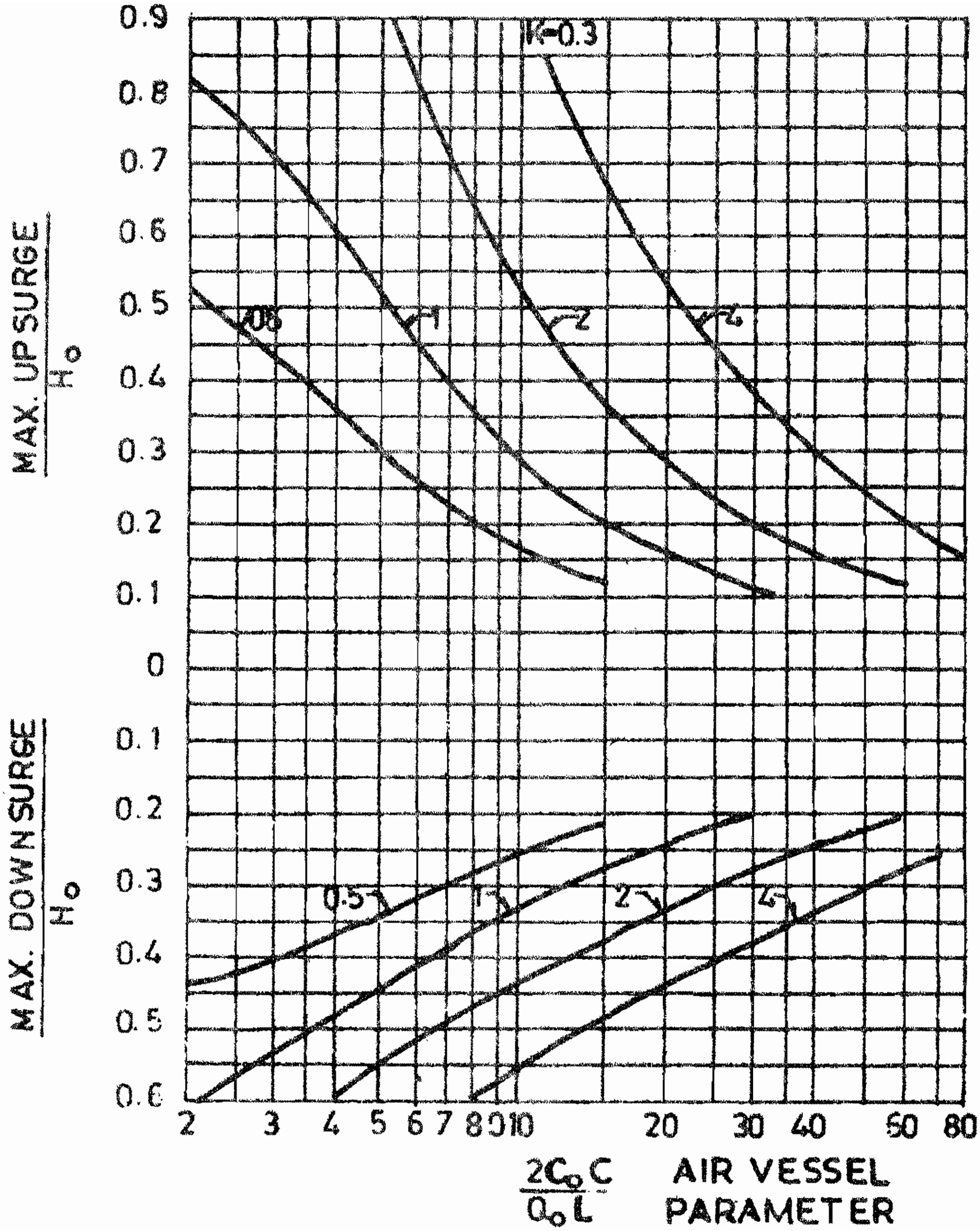

FIGURE 6.8 : SURGES IN PUMP DISCHARGE LINE, K_C = 0.3

A small diameter bypass to the reflux valve should be installed to permit slow refilling of the vapour pocket otherwise over pressures may occur on restarting the pumps. The diameter of the bypass should be of the order of one-tenth of the pipeline diameter. An air release valve should be installed in the pipeline at the peak to release air which would come out of solution during the period of low pressure.

It is common practice to install reflux valves immediately downstream of the pumps. Such reflux valves would not prevent water hammer pressures in the pipeline. They merely prevent return flow through the pump and prevent water hammer pressure reaching the pumps.

Normally a reflux valve installed on its own in pipe-line will not reduce water hammer pressures, although it may limit the lateral extent of the shock. In fact, in some situations indiscriminate positioning of reflux valves in a line could be detrimental to water hammer pressures. For instance if a pressure relief valve was installed upstream of the reflux valve the reflux valve would counteract the effect of the other valve. It may also amplify reflections from branch pipes or collapse of vapour pockets.

In some pumps installations, automatically closing control valves, instead of reflux valves, are installed on the pump delivery side.

6.17.4.3 Release Valves

There are a number of sophisticated water hammer release valves (often referred to as surge relief valve or surger suppressors) available commercially. These valves have hydraulic actuators which automatically open, then gradually close after pumps tripping. The valves are normally the needle type, which discharge into a pipe leading to the suction reservoir, or else sleeve valves, mounted in the suction reservoir. The valves must have a gradual throttling effect over the complete range of closure. Needle and sleeve valves are suitably designed to minimize cavitation and corrosion associated with the high discharge velocities which occur during the throttling process.

The valves are usually installed on the delivery side of the pump reflux valves and discharge directly to the suction reservoir. They should not discharge into the suction pipe as they invariably draw air through the throat, and this could reach the pumps.

The valves may be actuated by an electrical fault or by a pressure sensor. The valve should open fully before the negative pressure wave returns to the pumps as a positive pressure wave. As the pressure on the top of the piston increases again the valve gradually closes, maintaining the pressure within desired limits. The closing rate may be adjusted by a pilot valve in the hydraulic circuit.

If no over pressure higher than the operating head is tolerable, the valve would be sized to discharge the full flow at a head equal to the operating head, where reliability is of importance, and if water hammer is likely to be a problem during a partial shutdown of the pumps, two or more release valves may be installed in parallel. They could be set to operate at successively lower delivery heads.

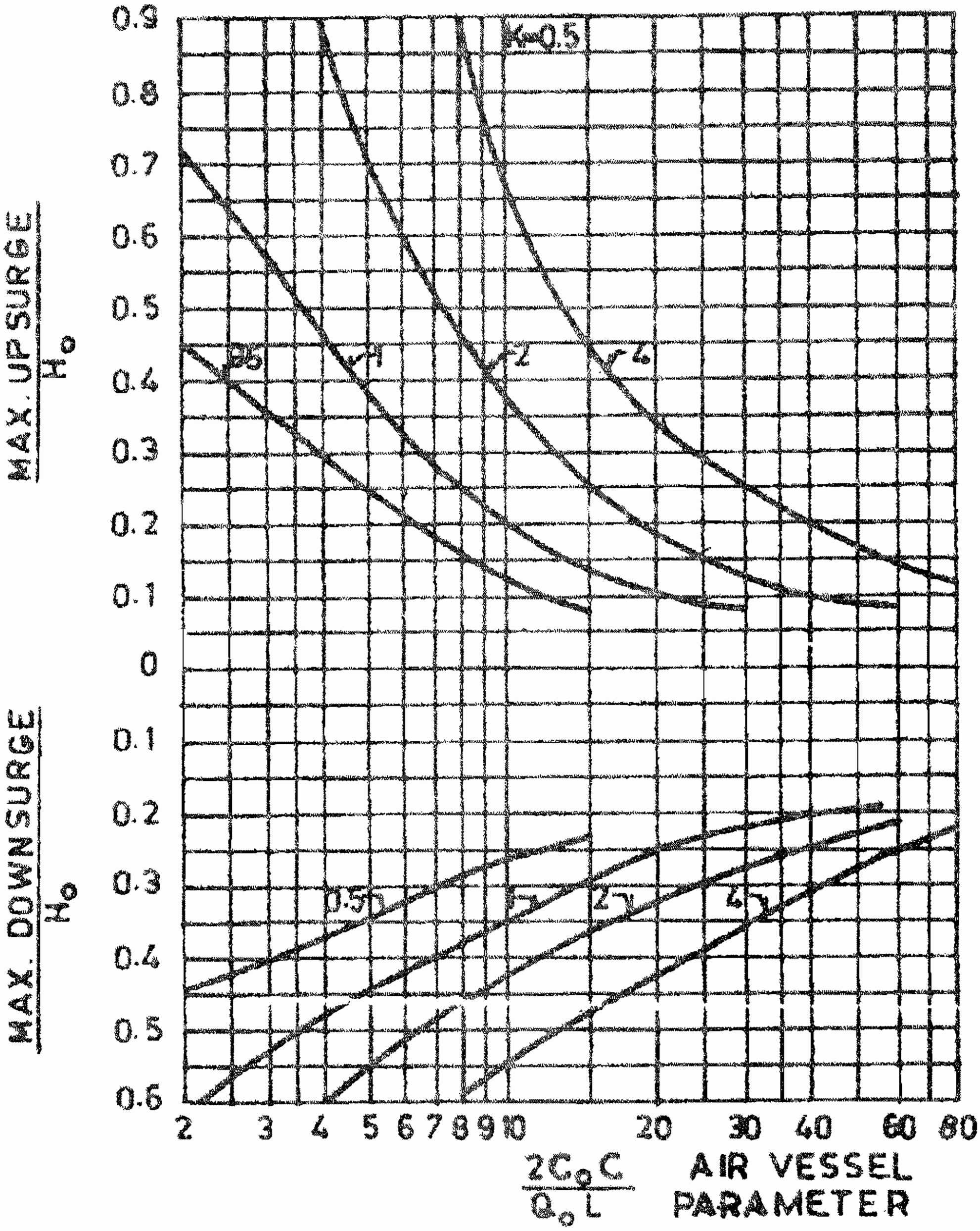

FIGURE 6.9 : SURGES IN PUMP DISCHARGE LINE, K_C = 0.5

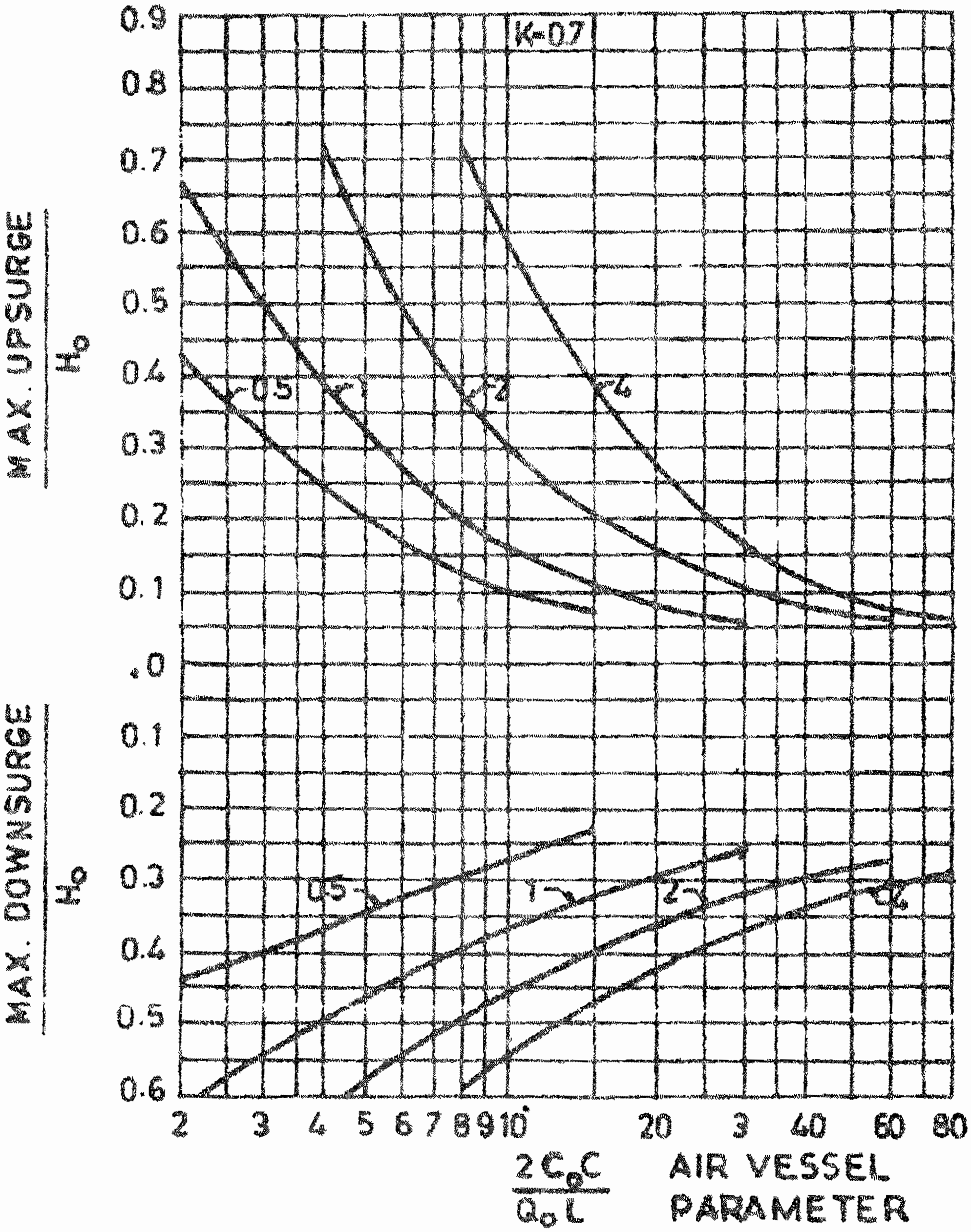

FIGURE 6.10 : SURGES IN PUMP DISCHARGE LINE, $K_C = 0.7$

could be disengaged to prevent their operation.

The types of control valves available as release valves for pumping lines normally cannot open in less than about five seconds. Their use is therefore limited to pipelines over two kilometers in length. This method of water hammer protection is normally most economical for cases when the pumping head greatly exceeds cv_0/g, since the larger the pumping head, the smaller the valve needed.

A less sophisticated valve than the control valves described above, which has been used on small pumps installation, is the spring-loaded release valve. The valve is set to open when the pressure reaches a prefixed maximum. Some over pressure is necessary to open the valve and to force water out.

Where a relief valve is power operated and actuated by a relay so as to open before reversal of flow takes place, over pressures can be held down so as not to rise more than 10 to 20% above normal operating pressure, although there may be an initial drop in pressure at the pump down to atmosphere or below. With the surge relief valve open, however, all succeeding reversals are dissipated through the open valve. The pipeline then assumes a penstock condition and the surge relief valve must be closed very slowly to prevent penstock surge. With large diameter lines for low pressure water service the economic justification for rather elaborate protective devices is obvious since without them the lines would have to be designed for shock pressures considerably in excess of the normal working pressure. This is particularly true in the case of concrete pipes, or of thin-walled steel pipes. With thin-walled steel pipes where the pressure may fall below atmospheric under shock conditions, it may be necessary to provide vacuum breakers to prevent collapsing of the pipe.

6.17.4.4. Shut-Off Effects On Suction Line

The effect of power interruption on the pump suction line depends on the arrangement of the suction piping. Nothing of much consequence will occur where the suction line is short and considerable suction lift has to be developed by the pump in order to get water flow to it. In the case of a booster pump, however, where water flows to the pumps suction through a long line under pressure, the result of a power interruption is much like what takes place in a discharge line and the measures taken to cushion shock are similar. To be most effective, a suppressor in such a booster pump suction should be placed close to the pumps. The booster pumps problem is frequently encountered in connection with the intermittent filling of standpipes such as overhead sprinkler tanks, pressure tanks in tall buildings and locomotive filling tanks in railroad yards. In cases where frequent fillings are required, the shock-pressure problem may be most annoying and corrective measures are clearly called for, especially if the pumps suction is taken from a branch line off a distribution network which may be adversely affected for large distances back from the pump. As an alternative to installing suppressors in such cases, consideration should be given to providing automatic means for slowly closing a valve in the pump discharge before power is cut off.

Another pump-suction problem involving surge on a large scale is encountered in water works intakes where the pumps may be fed through a conduit extending for several kilometers from some lake or reservoir in the mountains. In order to look after surge in the case of a sudden power interruption, it may be necessary to provide ample relief valves of gravity overflow, discharging to a receiving basin of generous proportions.

6.17.4.5 Reciprocating Pumps Or Hydraulic Rams

Reciprocating pumps cause pulsation problem not encountered with the continuous action of centrifugal pumps. Owing to the irregularity of flow through a reciprocating pump, more or less water hammer develops in the suction and discharge lines and cannot be suppressed entirely with vacuum or air chambers. For this reason it is advisable to design the suction and discharge lines of reciprocating pumps for something like 50% in excess of the normal working pressure and to provide ample air chambers at the pumps. Shock conditions obtaining with hydraulic rams are decidedly worse than with reciprocating pumps and generous provision should be made in the design of their piping. An allowance of at least 21 kg/cm^2 extra beyond the working pressure is called for with rams.

6.18 SPECIAL DEVICES FOR CONTROL OF WATER HAMMER

The philosophy is (i) to minimize the length of the returning water column causing water hammer (ii) to dissipate energy of the water column length by air cushion valve and (iii) to provide a quick opening pressure relief valve to relieve any rise in pressures in critical zones. These objectives are achieved by the following three valves.

6.18.1 Zero Velocity Valve

The principle behind the design of this valve is to arrest the forward moving water column at zero momentum i.e. when its velocity is zero and before any return velocity is established.

The valve fitted in the pipeline consists of an outer shell and an inner fixed dome leaving a streamlined annular passage for water. A closing disc is mounted on central and peripheral guide rods and is held in the closed position by one or more springs when there is no flow of water. A bypass connects the upstream and downstream sides of the disc. The springs are so designed that the disc remains in fully open position for velocity of water equal to 25% of the designed maximum velocity in the pipeline.

With sudden stoppage of pumps the forward velocity of water column goes on decreasing due to friction and gravity. When the forward velocity becomes less than 25% of the maximum, the flap starts closing at the same rate as the velocity of water. The flap comes to the fully closed position when forward velocity approaches zero magnitude, water column on the upstream side of the valve is thus prevented from acquiring a reversed velocity and taking part in creating surge pressures. The bypass valve maintains balanced pressures on the disc and also avoids vacuum on the downstream side of valve if that column experiences certain reversal.

The main advantages of zero velocity valves are :

(i) Controlled closing characteristics, and

(ii) Low loss of head due to streamlined design.

6.18.2 Air Cushion Valve

The principle of this valve is to allow large quantities of air in the pumping main during separation, entrap the air, compress it with the returning air column and expel the air under controlled pressure so as to dissipate the energy of the returning water column. An effective air cushion is thus provided.

The valve is mounted on TEE-joint on the rising main at locations where water column separation is likely. The valve has a spring loaded air inlet port, an outlet normally closed by a float, a spring loaded outlet poppet valve and an adjustable needle valve control orifice.

When there is sudden stoppage of pump due to power failure, partial vacuum is created in the main. With differential pressure, the spring loaded port opens and admits outside air into the main. When the pressure in the main becomes near atmospheric, the inlet valve closes under spring pressure. The entrapped air is then compressed by the returning water column till the poppet valve opens. With float in dropped position, the air is expelled through poppet valve and controlled orifice under predetermined pressure thus dissipating the energy of the returning water column.

6.18.3 Opposed Poppet Valve

As the name implies, the valve has two poppets of slightly different areas mounted on the same stem. The actual load on the stem is thus the difference in loads on the two poppets and is thus light. A weak spring is therefore, able to keep the valve closed under normal working pressure. If pressure in the water main increases beyond a certain limit, the increase in differential pressure overcomes the holding pressure of the spring, opens the valve and allows water to discharge through both the poppets.

On account of the light spring, the valve is able to open quickly and thus reduce the peak surge pressure to the desired limit.

6.19 WORKING OF THE SPECIAL DEVICES AS A SYSTEM

Every valve has a different function to perform for limiting water surge after power failure. Locations of the valves have therefore to be based on the results of the analysis of water column separation. Air cushion valves are located where separation of water column is indicated. Zero velocity valves are so placed that the entire length of water column is suitably divided in spite of differing gradients and undulations. More than one valve may be required in such cases.

Opposed Poppet pressure relief valves are generally placed near the air cushion valves or on the upstream side of the Zero Velocity Valves, if further limiting of peak surge pressure is required for the safety of the pipeline.

6.19.1 Choice Of Protective Device

The best method of water hammer protection for a pumping line will depend on the hydraulic and physical characteristics of the system. The accompanying Table 6.8 summarizes the ranges over which various devices are suitable. The most influential parameter in selecting the method of protection is the pipeline parameter $\rho = cv_0/gH_0$. When the pipeline parameter is much greater than 1, a reflux valve by passing the pumps may suffice. For successively smaller values of ρ it becomes necessary to use a surge tank, a discharge tank in combination with an inline reflux valve, an air vessel, or a release valve. The protective devices listed in Table 6.8 are arranged in approximate order of increasing cost. Thus, to select the most suitable device, one checks down the Table until the variables are within the required range.

It may be possible to use two or more protective devices on the same line. This possibility should not be ignored as the most economical arrangement often involves more than one method of protection. In particular the rotational inertia of the pump often has a slight effect in reducing the required capacity of a tank or air vessel. A comprehensive water hammer analysis would be necessary if a series of protection devices in combination is envisaged.

TABLE 6.8

SUMMURY OF METHODS OF WATER HAMMER PROTECTION

Method of protection (In approximate order of increasing cost)	Required range of Variables	Remarks
Inertia of pump	$(MN^2 / WALH_0)2 > 0.01$	Approximate
Pump bypass reflux valve	$(cv_0 / gH_0)2 \gg 1$	Some water may also be drawn through pump
In-line reflux valve	$(cv_0 / gH_0)2 > 1$	Normally used in conjunction with some other method of protection. Water column separation possible

Method of protection (In approximate order of increasing cost)	Required range of Variables		Remarks
Surge tank	H small		Pipeline should be near hydraulic grade line so height of tank is practical
Automatic release valve	$(Cv_0\ /\ gH_0)2$ $(L/C)2$	$<< 1$ > 5secs	Pipeline profile should be convex downwards. Water column separation likely.
Discharge tanks	$(Cv_0/gh)2$	> 1	h = pressure head at tank./ Pipeline profile should be convex upwards
Air vessel	$(Cv_0/gH_0)2$	< 1	Pipeline profile preferably convex downwards.

The example in App. 6.7 gives the methods of analysis and calculations for water column separation and computation of Air Vessel size.

M = Moment of Inertia of rotating parts of pump, motor and entrained water (mass x radius of gyration^2)

N = Pump speed in rpm

W = Wt. of water per unit volume

A = Pipe cross section Area

L = Pipeline length

H_0 = Pumping head

h = Pressure head

c = Water hammer wave velocity

v_0 = Initial velocity

J = Pump parameter,

f = Pump rated efficiency (expressed as a fraction).

CHAPTER 7
WATER TREATMENT

7.1 METHODS OF TREATMENT AND FLOW SHEETS

The aim of water treatment is to produce and maintain water that is hygienically safe, aesthetically attractive and palatable, in an economical manner. Though the treatment of water would achieve the desired quality, the evaluation of its quality should not be confined to the end of the treatment facilities but should be extended to the point of consumer use.

The method of treatment to be employed depends on the nature of raw water constituents and the desired standards of water quality. The unit operations in water treatment include aeration, flocculation (rapid and slow mixing) and clarification, filtration, disinfection, softening, deferrization, defluoridation and water conditioning and many different combinations of these to suit these requirements. Sketches of flow sheets are presented in Fig. 7. 1. The choice of any particular sequence of treatment units will depend not only on the qualities of the raw water available and treated water desired but also on the comparative economics of alternative treatment steps applicable.

In the case of ground waters and surface waters with storage which are well protected, where the water has turbidity below 10 NTU and they are free from odour and colour, plain disinfection by chlorination is adopted before supply as shown in Fig: 7.1 (a) and (b).

Where ground water contains excessive iron, dissolved carbon dioxide and odorous gases, aeration followed by flocculation (rapid and slow mixing) and sedimentation, rapid gravity or pressure filtration and disinfection may be necessary as in Fig. 7.1 (c). In case it contains only carbon dioxide or odorous gases, aeration followed by disinfection may be sufficient. In surface waters with turbidities not exceeding 50 NTU and where sufficient area is available, plain sedimentation followed by slow sand filtration and disinfection are practised.

Conventional treatment including prechlorination, aeration, flocculation (rapid and slow mixing) and sedimentation, rapid gravity filtration and post chlorination are adopted for highly polluted surface waters laden with algae or other microorganisms.

Sometimes, unconventional flow sheets may be adopted for waters of low turbidity (below 10 to 15 NTU) and containing low concentration of suspended matter (less than50 mg/l) as in Fig 7. 1. (f). Such raw waters are applied to the rapid sand filters with alum addition which may or may not be accompanied by slow mixing for a short period (10 minutes).

Slow sand filters can also be used to polish the filtrate from rapid sand filtration plant. Water with excessive hardness needs softening as in Fig. 7.1 (g). For removal of dissolved

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solids, demineralisation by ion-exchange may form a part of the domestic or industrial water treatment units as in Fig. 7.1 (h).

7.2 AERATION

Aeration is necessary to promote the exchange of gases between the water and the atmosphere. In water treatment, aeration is practised for three purposes:

a) To add oxygen to water for imparting freshness e.g:- water from underground sources devoid of or deficient in oxygen.

b) Expulsion of carbon dioxide, hydrogen sulphide and other volatile substances causing taste and odour e.g. water from deeper layers of an impounding reservoir; and

c) To precipitate impurities like iron and manganese in certain forms e.g. water from some underground sources.

7.2.1 LIMITATIONS OF AERATION

The unit operation of aeration requires significant head of water. The water is rendered more corrosive after aeration when the dissolved oxygen content is increased though in certain circumstances it may be otherwise due to removal of aggressive carbon dioxide. The designer should carefully consider the merits of other alternatives because of the additional cost of lifting which may be involved in aeration. For taste and odour removal, aeration is not highly effective but can be used in combination with chlorine or activated carbon to reduce their doses.

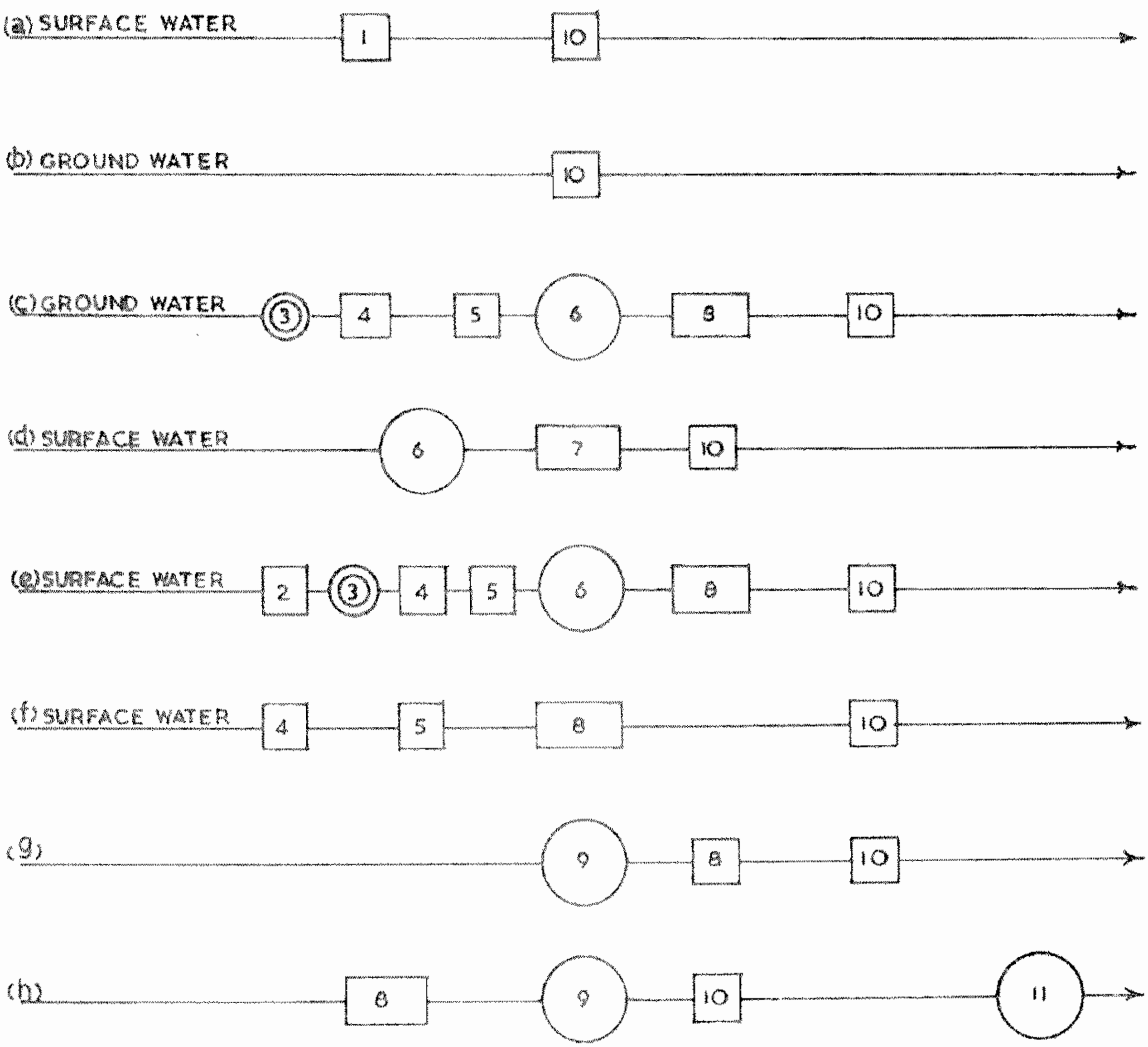
7.2.2 AERATION PROCESS

Gases are dissolved in or liberated from water until the concentration of the gas in the water has reached its saturation value. The concentration of gases in a liquid generally obeys Henry's law which states that the concentration of each gas in water is directly proportional to the partial pressure (product of the volume per cent of the gas and the total pressure of the atmosphere) or concentration of gas in the atmosphere in contact with water. The saturation concentration of a gas decreases with temperature and dissolved salts in water. Aeration tends to accelerate the gas exchange.

The rate of exchange of a gas is governed by the area of interface between the gas and the liquid, the thickness of the interlayers, time of contact, the partial pressure of the gas in the overlaying atmosphere and the degree of under-saturation or oversaturation of the gas in the liquid.

To ensure proper aeration, it is necessary:

a) To increase the area of water in contact with the air i.e. if the water is sprayed, the smaller the droplets produced, the greater will be the area available. Similarly, if the water is being made to fall as a film over packing material in a tower, the smaller the size of the packing material, the greater will be the area available.



1. STORAGE
2. CHLORINATION (PRE)
3. AERATION
4. RAPID MIXING
5. FLOCCULATION — SLOW MIXING
6. SEDIMENTATION
7. SLOW SAND FILTRATION
8. RAPID SAND FILTRATION
9. SOFTENING
10. CHLORINATION (POST)
11. DEMINERALISATION

FIG.7.1 : UNIT OPERATIONS IN WATER TREATMENT

b) To keep the surface of the liquid constantly agitated so as to reduce the thickness of the liquid film which would govern the resistance offered to the rate of exchange of the gas; and

c) To increase the time of contact of water droplets with air or to increase the time of flow which can be achieved by increasing the height of jet in spray aerator and increasing the height of bed in the case of packed media.

Where oxygen is to be dissolved in water the concentration or partial pressure of the oxygen may be increased by increasing the total pressure of the gases in contact with water. For this reason air injected into a main under pressure is a reasonably efficient method of increasing the amount of dissolved oxygen.

The exchange of gases from water to air or from air to water which takes place at the air water interface can be described by the following formulae:

$$C_t = C_s - (C_s - C_o)\exp\left(-(k\frac{A}{V}t)\right) \qquad (7.1)$$

(Gas absorption)

and

$$C_t = C_s + (C_s - C_o)\exp\left(-(k\frac{A}{V}t)\right) \qquad (7.2)$$

(Gas release)

Where,

C_t = actual concentration of the gas in the water after a given period 't';

A/V = ratio of exposed area to the volume of water;

C_s = gas saturation concentration;

k = gas transfer coefficient (having dimension of velocity);

C_o = concentration of gas initially present in the water; and

t = aeration period.

The gas saturation values of H_2S and CO_2 are generally 0 and 0.5 mg/l when exposed to normal atmosphere having partial pressures of the gases of 0 and 0.03 percent respectively. Because of the low saturation values, removal of H_2S and CO_2 by aeration is practicable.

If the initial concentration of the gas to be removed from water is much above the saturation limit, sizeable reduction in the concentration of the gas by aeration is possible.

7.2.3 TYPES OF AERATORS

There are two main types of aerators depending upon the mechanics of aeration:

a) those forming drops or thin sheets of water exposed to the atmosphere i.e. water is exposed to come in contact with the ambient air; and

b) those forming small bubbles of air which rise in the water i.e. air is brought in contact with the water.

Spray, water-fall or multiple tray, cascade and mechanical aerators can be considered under type (a), while diffusion aerators fall under type (b).

7.2.3.1 Spray Aerators

Water is sprayed through nozzles upward into the atmosphere and broken up into either a mist or droplets. Water is directed vertically or at a slight inclination to the vertical. The installation consists of trays and fixed nozzles on a pipe grid with necessary outlet arrangements.

Nozzles usually have diameters varying from 10 to 40 mm spaced in the pipe at intervals of 0.5 to 1 m or more. Special (patented) types of corrosion resistant nozzles and sometimes plain openings in pipes, serving as orifices, are used. The pressure required at the nozzle head is usually 7 m of water but practice varies from 2 to 9 m and the discharge ratings per nozzle vary from 18 to 36 m^3/hr. Usually aerator area of 0.03 to 0.09 $m^2/m^3/hr$ of design flow is provided.

The time of exposure of the droplets, the head required and the flow from each nozzle can be calculated from the following formulae:

$$\upsilon = C_v\sqrt{2gh} \tag{7.3}$$

$$q = C_d a\sqrt{2gh} \tag{7.4}$$

$$t = 2C_v \frac{\sqrt{2h}}{g} \quad Sinx \tag{7.5}$$

where,

h = Total head of water at the nozzle;

g = Acceleration due to gravity;

v = Initial velocity of drop emerging from the nozzle;

C_v = Coefficient of velocity,

C_d = Coefficient of discharge;

q = Discharge rate from each nozzle;

a = Area of cross-section of nozzle opening; and

t = Time of travel or exposure and x is the angle of inclination of the spray from the horizontal.

The vertical jet gives the longest exposure time for a given value of h (2 seconds for a head of 6 m), while the inclined jets can have less interference between falling drops. Wind can influence the path of the trajectory of each drop and allowance must be made for its effect. The dimensions of the tray must take into account the velocity and direction of the wind to ensure that no water is lost by carry-away. The size, number and spacing of nozzles, aeration time and interference between adjacent sprays, as already explained are also factors governing the aeration efficiency. Spray aerators are usually quite efficient with respect to gas transfer and can be expected to remove 70 to 90% of CO_2 and 90 to 99% H_2S and add to the appearance of a water treatment plant. They require large area and consequently difficult to be housed readily and pose operating problems due to corrosion and choking of the nozzles particularly during freezing weather.

The diameters of the pipe grid and orifices should be so designed as to ensure a uniform discharge (with a maximum variation of 5 percent) through all the nozzles in the grid. The loss of head in the pipe is kept low compared to the loss of head in the nozzle. Theoretically numerous small nozzles capable of producing atomised water could be used. Practically., however extremely small nozzles are to be avoided because of clogging and consequent excessive maintenance needed. Common friction formulae are used in the estimation of loss of head, excepting that the pipe with nozzles has to be considered to be carrying uniformly decreasing flow.

7.2.3.2 Waterfall Or Multiple Tray Aerators

Water is discharged through a riser pipe and distributed on to a series of trays or steps from which the water falls either through small openings to the bottom or over the edges of the trays. Water is caused to fall into a collection basin at the base. In most aerators, coarse media such as coke, stone or ceramic balls, ranging from 50 to 150 mm in diameter are placed in the trays to increase the efficiency. For iron removal (see 9.5.2) this may be beneficial. The trays about 4 to 9 in number (with a spacing of 300 mm to 750 mm) are arranged in a structure 1 m to 3 m high. With the media, good turbulance is created and large water surface is exposed to the atmosphere. By the addition of more trays, the time of contact can be increased. The space requirements vary from 0.013 to 0.042m^2 per m^3/hr of flow. Natural ventilation or forced draft is provided. Removal efficiencies varying from 65 to 90 percent for $C0_2$ and 60 to 70 percent for H_2S have been reported.

7.2.3.3 Cascade Aerators

In cascade aerators water is allowed to flow downwards after spreading over inclined surface in thin sheets and the turbulence is secured by allowing the water to pass through a series of steps or baffles. The number of steps is usually 4 to 6. Exposure time can be increased by increasing the number of steps and the area to volume ratio improved by adding baffles to produce turbulence. Head requirements vary from 0.5 to 3.0 metres and the space requirements vary from 0.015 to 0.045 m^2/m^3/hr. In cold climates, these aerators must be housed with adequate provision for ventilation. Corrosion and slime problems may be encountered. The gas transfer efficiency is less compared to the spray type. Removal of gas varies from 20 to 45 percent for CO_2 and upto 35 percent for H_2S.

7.2.3.4 Diffused Air Aerators

This is an obverse of waterfall type aerator. This type of aerator consists of a basin in which perforated pipes, porous tubes or plates are used for release of fine bubbles of compressed air which then rise through the water being aerated. As the rising bubbles of air have a lower average velocity than the falling drops, a diffused air type provides a longer aeration time than the water fall type for the same power consumed. These have higher initial costs and require greater recurring expenditure. Tanks are commonly 3 to 4.5 m deep and 3 to 9 m wide. Compressed air is injected through the system to produce fine bubbles which on rising through the water produce turbulence resulting in a continual change of exposed surface. Ratios of width to depth should not exceed 2:1 for effective mixing and the desired detention period varies from 10 to 30 minutes. The amount of air required ranges from 0.06 to 1m^3 of air per m^3 of water treated. The air diffusers are located on one side of the tank. The power requirements of blower vary from 3 to 13 w/m^3/hr.

The air should be filtered before passing through porous diffusers. Oil trap is also provided before diffusers. Diffused aerators require less space than spray aerators but more than tray aerators. Cold weather operating problems are not encountered. The aerators can also be used for mixing of chemicals.

Compressor power requirements may be estimated from the air flow, discharge and inlet pressures and air temperatures, using the following equation, which is based upon the assumption of adiabatic conditions:

$$P = \frac{wRT_1}{(8.41)e}\left[\left(\frac{p_2}{p_1}\right)^{0.283} - 1\right] \qquad (7.6)$$

where,

P	=	Power required in KW;
p_1	=	Absolute inlet pressure in atm. (normally 1 atm);
p_2	=	Absolute outlet pressure in atm.;
R	=	Gas constant (8.314 J/mole, ^{0}K);
w	=	Air mass flow in Kg/s;
e	=	Efficiency of the machine, (usually 0.7 to 0.8); and
T_1	=	Inlet temperature in degrees ^{0}K

7.2.3.5 Mechanical Aerators

These are not normally used in water treatment because of the availability of more economical alternatives but find application in waste water treatment.

7.3 CHEMICALS HANDLING HANDLING AND FEEDING

The chemicals are introduced into the water for the purposes of coagulation and flocculation, disinfection, softening, corrosion control, algae control and fluoridation. In general, chemicals are added as solutions or dilute suspensions. As the treatment is a continuous process, the flow of chemicals is regulated and measured continuously through chemical feeders which can be either solution feed type or the dry feed type. The installation of chemical feeders obviously promotes the uniform distribution of chemicals and eliminates wastage. Every chemical feeder should be arranged and positioned in such a way that checking of dosing rate can be made at regular intervals to verify the discharge rate.

7.3.1 SOLUTION FEED

Preparation of the solution of the chemical in water in desired strength is the first step and is done in the solution tanks. This solution is fed to the raw water through controlled feeders which are of gravity or pressure type. The selection of the proper type of feders and the point of application are important. For example, when mixing is done in a channel, it should be at a number of points in the cross-section of maximum turbulence. Also as different chemicals are to be fed at different points, the location at which the chemicals are fed is important to derive maximum efficiency.

7.3.1.1 Solution Tanks

There should be at least two tanks for each chemical feed. The capacity of each tank should generally be such as to hold 8 hours requirement at the maximum demand of chemical at the design flow. A minimum free board of 0.3 m is necessary. Dissolving trays or boxes and also adequate facilities for draining the solution tanks should be provided.

The solution tank may be constructed either of masonry, plain or reinforced cement concrete. Coating with bituminous paint may be adequate for alum tanks while for tanks for handling other corrosive chemicals, suitable lining of rubber, PVC or Epoxy resin may be necessary to resist corrosion.

The chemical solution tanks should be located in or as near the chemical storage godown as possible to avoid unnecessary lifting and handling of chemicals. These tanks should preferably be located at a suitable elevation, to facilitate gravity feed of the chemical solution.

A lifting tackle for lifting the chemicals to the elevated tanks should be provided. Each tank should have a platform which should be at least 0.75 m wide to allow the workers sufficient space for handling the chemicals and preparing the solution; wherever necessary, the platforms should have railings upto a minimum height of 0.75 m. The platforms should be located at an elevation to have clear headroom of 2.0 m from the ceiling. The top of the solution tank should not be higher than 1.0 m from the floor of the platforms.

7.3.1.2 Dissolving Trays Or Boxes

The chemicals after being carefully weighed, are placed into the dissolving trays which vary in size to suit the capacity of the treatment plant. The trays or boxes may be constructed of wood, cast iron or cement or cement concrete with slots or perforations both at the sides and at the bottom. These may be placed either inside or just above the solution tanks.

For small tanks, a pipe perforated with small holes to provide a spray of water to help dissolve the chemicals, may be placed above these trays. For plants of medium and large size, dissolving boxes should preferably be constructed of concrete with a pipe manifold having holes either at bottom or at sides for dissolving chemicals.

7.3.1.3 Preparation Of Solutions

It is essential to ensure that all the chemicals are dissolved before the solution is put into operation and the homogeneity of the prepared chemical solution is maintained. This can be achieved by proper mixing either by compressed air or recirculating the solution or by mechanical agitation. For plants having capacities not exceeding 2500 m^3/d manual mixing may be adopted ensuring proper mixing.

A knowledge of the solubility characteristics of the chemical as well as the solution strength that are used in normal practice will facilitate the choice of feed equipment. The solution strength of alum which is the most widely used coagulant shall not be more than 5% for manual operations and 10% for other operations with efficient mixing. It may be desirable to dilute down to 1% prior to addition. For other chemicals, reference may be made to Appendix 7.10 which gives the strengths to be used with mechanical mixing. With manual operation, lower strengths are recommended.

The chemical solution is conveyed from the solution tanks to the point of application by means of chemical feed lines. These should be as short and straight as possible.

Liquid Alum

Liquid alum contains 5.8 to 8.5% water soluble alumina as against 17% for crystalline alum, but is lower priced. Since its use also avoids construction of solution tanks, it may be economical in large plants especially if the waterworks are within a reasonable trucking distance of alum producing works. Acid-proof equipment such as rubber-lined or stainless steel tanks and piping is necessary for transport, handling and storage.

7.3.1.4 Solution Feed Devices

Solution feed devices are used to regulate the doses of chemical fed into water. The rate of flow of the chemical solution of known strength prepared in the solution tank is measured by means of either an orifice rotameter, positive displacement pump or by weirs. The solution feed equipment should be simple in operation and corrosion resistant.

The constant head orifice is the most common device used for measuring the rate of flow of solution. It is usually contained in a unit consisting of corrosion resistant, constant level box with a float valve and an orifice. The orifice can be of either variable size or constant size, the adjustment in the latter being made by using the required size to give the desired rate of flow. The unit should also be capable of adjustment to allow setting for various depths of solution in the box.

In large systems, automatic control of chemical feed could be practiced which assures that the quantity of chemical measured is not prone to human errors. The principle must be based upon the measurement of some attributes of the water such as the rate of flow, pH, colour, conductivity, chlorine residual.

Since the flow of water can fluctuate, it is necessary to maintain the flow of chemical in a fixed proportion to the flow of water for which a proportional feed device is necessary. Measurement of the water can be done in a number of ways, the simplest possibly being the tipping bucket or a pump with positive meter which provides a positive method of measurement but is applicable to the smaller installations only. The more common measuring device is a weir, venturi tube or orifice plate described in Chapter 4.

Another method is based on the actuation of a flow regulator directly or through a relay from the primary measuring unit. This usually involves the empirical calibration of some link in the system and care must always be taken to see that such arrangements are properly adjusted for they do not depend on a state of equilibrium.

The most satisfactory method of control is one that depends upon the matching of two factors, one of which is associated with the primary measuring unit (control) and the other with the flow of chemical. For example, a venturi tube will produce a differential pressure bearing known relationship to the flow of water through it. If it is desired to control the flow of a chemical solution, then some similar measurement associated with the flow of a chemical solution must be compared with the differential pressure and means provided for adjusting the flow of chemical so that the two factors so compared are mutually in equilibrium. Such a system is basically stable.

7.3.1.5 Solution Feeders

There are several types of solution feeders, some of which are discussed below:

(a) Pot Type chemical Feeders

The pot type chemical feeder is a simple type of equipment for feeding alum or alkali into water. The chemical, in large crystal or lump form, is charged into the feeding pot. A special orifice fitting, placed in the raw water line, contains an orifice plate which creates a pressure differential in pipes which connect the chemical pot into the orifice fitting.

This pressure differential causes a small stream of water to flow from the high pressure side of the orifice plate through a pipe and a regulating valve, into the bottom of the chemical feeding pot and this forms an equivalent stream of the chemical solution, formed in the pot, to flow out of the top of the pot into the raw water line on the low pressure side of the orifice plate.

Since the same pressure differential acts across the regulating valve as across the orifice, the flow through the regulating valve, at any setting, is a definite fraction of the flow through the orifice. Consequently, the rates of flow of the small stream of chemical fed to the raw water are directly proportional to the rates of flow of the raw water. These find use in small plants because they do not permit a uniform feed rate and the feed rate cannot be also checked. Sediment tanks are usually employed with these feeding lines.

(b) Pressure Solution chemical Feeders

Pressure solution chemical feeders are much more accurate than the pot type chemical feeders. In these a chemical solution of a definite strength is made by dissolving a weighed amount of chemical in a specified volume of water in the chemical solution tank. This batch of chemical solution, when required, is charged into the displacement tank through the bottom. As the specific gravity of the chemical solution is higher than that of water, the water in the displacement tank is displaced upwardly to waste through a valve.

A sight glass at the side of the feed tank has in it a glass float, which is so constructed that it floats in the heavy chemical solution but sinks in water. This float indicates, at all times, the level of the chemical solution thus notifying the operator when recharging is necessary.

A special orifice fitting, placed in the raw water line, contains an orifice plate which creates a pressure differential in the pipes connecting the displacement feed tank to the orifice fitting. This pressure differential causes a small stream of water to flow from one side of the orifice plate. The greater part of this stream flows through a secondary orifice and the smaller through an adjustable needle valve into the top of the displacement feed tank, where it displaces downwardly an equivalent stream of the heavier chemical solution.

This small stream of chemical solution is diluted when it discharges on the other side of the secondary orifice into the water flowing through this orifice and this diluted chemical solution is fed into the raw water line on the other side of primary orifice. This dilution serves to make the density of the effluent column approach the density of the influent column thus assuring a greater degree of accuracy, at varying flow rates, than is possible with a single orifice control.

Since the same pressure differential acts across the primary orifice as across the needle valve, the flow through the needle valve at each setting, is a constant fraction of the flow through the primary orifice. As the rates of flow of the chemical solution are directly proportional to the rates of flow of the raw water, this type of feed is applicable to water supplies of varying flow rates and pressure. Sediment tanks are usually employed with pressure sloution chemical feeders to keep sediment out of the feeding line. In cases, where corrosive chemicals are handled, special pressure solution chemical feeders are employed.

(c) Electro-chemical Feeders

The water flows through an integrating raw water meter causing an electrical circuit to start the feed control unit through a time switch. The feed control unit is a mechanism designed to lower the swing drawoff pipe at a rate which is proportional to the rate of flow of raw water. It consists of a motor, a speed reducing mechanism, two drums on which separate tapes are wound, a manual rewinding mechanism, a switch for operating an alarm for stopping the feed at low level in the solution tank and a dial for indicating directly the depth of solution removed from the tank.

(d) Gravity Orifice chemical feeders

The gravity orifice chemical feeder is limited in application to those cases where the flow rate of the water being treated is constant. The solution from the chemical solution tank flows by gravity, through a strainer and through a float valve, into the orifice box.

The float valve keeps the chemical solution in the orifice box always at the same level so that the adjustable orifice operates under constant head. By gravity, the chemical solution flows from the orifice box through the adjustable orifice to the point of application.

To stop and start the chemical and water simultaneously, a float switch may be used in the settling basin to operate a solenoid-operated valve on the orifice box discharge and an electrically controlled valve on the raw water line. Thus the flows of raw water and chemical solution are stopped whenever the level of the water in the basin has reached a certain height. When the level has fallen a certain distance, the float switch closes an electric circuit thus starting simultaneously the flows of raw water and chemical solution.

Instead of being connected to an electrically controlled valve in the raw water line, the float switch may be connected so as to start or stop a raw water pump simultaneously with the starting or stopping of the chemical feeder.

The amount of chemical solution fed to the raw water may be varied over a wide range by means of the adjustable orifice located in the orifice box.

Instead of the chemical solution flowing by gravity to the point of application, it may be discharged in to a pump suction box from which it is pumped to the point of application.

(e) Reciprocating Pump chemical Feeders

This method of feeding chemical employs a motor-driven reciprocating chemical pump. The pump withdraws a chemical solution, or suspension of suitable strength, from a tank and discharges the solution or suspension to the point of application under any desired pressure. The feeding pump may be designed to treat either a variable or a constant flow of water.

The chemicals to be fed are prepared in solution tanks. If the chemical to be fed is relaively insoluble, a high speed motor-driven agitater maintains uniform suspension through out the full depth of the tank. If the chemical forms a clear solution, a dissolving basket is furnished and the mechanical agitator is omitted.

(1) Variable rate proportional feeders

If the rate of flow of water being treated varies, proportional feeding of chemicals is necessary. This is carried out by accurately measuring the amount of chemical fed by the pump. This pump is a proportioning and metering device which delivers a definite volume of chemical with each stroke. A water meter with an electrical contactor is placed in the raw water line. The contactor closes a circuit every time a given volume of water flows through the meter. The closing of the circuit energizes the motor of the reciprocating pump, which then operates to deliver a given volume of chemical until an electric time switch breaks the circuit, thereby stopping the pump. The cycle repeats itself approximately every thirty seconds, at maximum flow, with the pump operating for approximately twenty seconds after each contact. The amount of chemical fed is thus accurately proportioned to the flow of water regardless of variations in the rate of flow, because both the volume of water treated between meter contacts and the volume of chemical added to treat the water are accurately measured. However this suffers from the disadvantage that, particularly when used with alum solutions, the water is subject to an overdose and no-dose sequence. It is better to have the chemical pump run continously and to modulate the stroke of the pump either manually or with a mechanical device.

For a number of chemicals fed simulataneously, one meter control serves to operate any number of pumps.

(2) Constant rate feeding for uniform flow

If the flow of water being treated is constant, the chemical pump operates continuously at the set dosage. When the flow of water ceases, the chemical pump is stopped automatically so as to shut off the flow of chemicals. When the flow of water begins again, the chemical feeding is automatically resumed.

(3) Adjustment of feeding rates

Two methods are available for adjusting the rate of chemical feeding. Firstly, the length of the pump stroke can be changed to vary the rate of feeding of a given strength of solution over a wide range. Secondly, the strength of the chemical solution or suspension in the chemical tank can be changed when a new chemical charge is made up so as to provide a different chemical dosage for the same setting of the chemical pump.

The method of adjustment of the chemical feeding rate varies with the type of proportioning pump used. The single feed pump varies the feeding rate by a simple screw adjustment, which changes the length of the plunger stroke. The duplex pump varies its feeding rate by screwing the adjusting coupling toward the liquid end of the pump to increase the capacity or away from the end to decrease the capacity.

The reciprocating chemical pumps can be provided with ball check valves on both suction and discharge, thus assuring maximum efficiency of displacement, non-clogging and selfcleaning features, elimination of air binding and the minimising of wire drawing of valve seats. The check valves are readily opened, to inspect the ball checks and seats, without disconnecting either suction or discharge piping.

7.3.2 Dry Feed

Dry chemical feeders incorporate a feed hopper which sometimes serves as a storage hopper also mounted above the feeding device. This device may consist of a rotating table and scraper, a vibrating trough or an oscillating displacer or some equivalent method of moving the chemical from the point where it leaves the feed hopper to the point of discharge. The rate of movement of the chemical determine the quantity to be discharged on a volumetric basis. Gravimetric feeders are also available in which the quantity discharged in a unit of time is continuously weighed and the speed of operation automatically controlled to maintain a constant weight. The feeder may be designed for constant rate operation or for feeding chemicals in proportion to the rate of flow of water. The dry feeders with a completely enclosed feeding mechanism have many advantages over the solution feeder like accuracy of feeding, reproducibility of feeding rate for any feeder setting with a stepless adjustment of dosage in a wide feeding range. A single feeder serves as a spare for a group of feeders handling different materials and the height of chemical in standard or extension hopper has no effect on feeding rate. When small rates of chemical feeding are desired, one hopperful of chemical will allow the feeder to operate for several days unattended.

Chemicals stored in a steep-sided hopper feed downward to a discharge opening at the bottom of the hopper. Chemicals which have a tendency to arch or stick, such as lime and soda ash, are made free-flowing by a vibrator mounted on the side of the hopper at a point where it produces the most effective vibration. Exact volumes of chemicals are sliced off and displaced from the bottom of the discharge opening by an endless belt with integral lugs in the form of equally spaced partitions.

Machined guides on both sides and above the lugs insure that each pocket is filled with an exact volume of chemical. As the belt moves forward it passes over a pulley where each pocket is stretched open as the belt goes over and then under this pulley so that all the chemical is dropped into a mixing or dissolving chamber. A jet of water admitted tangentially or power driven paddles in the mixing chamber provides the agitation needed for mixing or dissolving the chemical. This chemical solution, or suspension then overflows or is pumped to service.

Where the quantity to be handled is large a storage hopper is usually constructed above the relatively small fed hopper. The capacity of the storage hopper is usually arranged for recharging once a day or once a shift. Because of the height of such hoppers, it is almost inevitable that storage of chemicals has to be at an elevated place to obviate the need of lifting of the chemicals every time.

7.3.3 CHEMICALS

7.3.3.1 Chemicals Used And Their Properties

Appendix -7.10 gives the list of chemicals commonly used in water treatment and their properties.

7.3.3.2 Chemical Storage

The chemical store should be of damp proof construction, properly drained. Special precautions against flooding should also be taken.

For chemicals purchased in bags, storage by piling on the floor of the store room may be arranged. A height of stack not exceeding 2 m is recommended. Hygroscopic chemicals should be obtained in moisture-proof bags and stored in air-tight containers.

All plants, particularly small ones, should keep on hand at all times, a supply of chemicals sufficient to provide a safety factor. A storage of 3 months is advisable but this again depends upon the location of the plant as well as the source of supply, transport facilities and the arrangement made with the suppliers for the supply of chemicals.

In cases where the major storage is provided at a place away from the feed equipment, a week's storage space should be provided near the plant.

Dampness may cause severe caking even in chemicals such as aluminium sulphate which usually are free from such troubles. Quick lime gradually expands on prolonged storage and may even burst the containers if kept too long.

Chemicals such as powdered activated carbon which are likely to cause dust problems should preferably be stored in separate rooms.

Storage of acid materials near alkalis is undesirable as their contact generates considerable heat resulting in combustion. This is also true of oxidising chemicals such as chloride of lime mixed with activated carbon. Hence they should be isolated. It is advisable to store chlorine cylinders separately as gaseous chlorine in contact with activated carbon leads to severe fire hazards.

7.3.3.3 Handling Of Chemicals

Oridinarily a 50 kg container can be handled by a single person when aided by small hand carts. Heavy containers should be handled with the aid of mechanical contrivances such as trucks, monoraill pulley, cranes and other special equipments.

Chemicals such as chlorine , ferric chloride, sodium hydroxide, sulphuric acid, ammonium chloride, ammonia, suplhur dioxide and sodium bisulphite should be handled by equipment, specially designed to reduce the hazards in their handling to a minimum. Care should be taken to prevent the dropping or bumping of the containers of these chemicals. For safe lifting, cranes should be preferred to ropes.

Sufficient space with access should be provided for handling bulk storages allowing for negotiating of vehicles and cranes likely to be used.

Rolling of cylinders, barrels and drums on the floor should be avoided.

Chlorine, ammonia and sulphur dioxide are toxic gases when present even in small concentrations in the air. Hence special care must be exercised in their handling. Sodium bisulphate may give off sulphur dioxide and may cause corrosion when spilled. Ferrous sulphate mixed with lime is likely to generate enough heat to start combustion. When such chemicals are used, special care needs to be given to ventilation arrangements (IS: 3103-1965). In the case of chlorination rooms, ventilation is specially necessary at the bottom and should be provided by exhaust fans.

7.4 COAGULATION AND FLOCCULATION

The terms 'Coagulation' and 'Flocculation' are often used indiscriminately to describe the process of removal of turbidity caused by fine suspensions, colloids and organic colour.

'Coagulation' describes the effect produced by the addition of a chemical to a colloidal dispersion, resulting in particle destabilization. Operationally, this is achieved by the addition of appropriate chemical and rapid intense mixing for obtaining uniform dispersion of the chemical.

'Flocculation' is the second stage of the formation of settleable particles (or flocs) from destabilised colloidal sized particles and is achieved by gentle and prolonged mixing.

In modern terminology, this combination of mixing (rapid) and stirring or agitation(slow mixing) that produces aggregation of particles is designated by the single term 'flocculation'. It is a common practice to provide an initial rapid or flash mixing for dispersal of the coagulant or other chemicals into the water followed by slow mixing where growth of floc takes place.

7.4.1 INFLUENCING FACTORS

Both these states in flocculation are greatly influenced by physical and chemical forces such as electrical charges on particles, exchange capacity, particle size and concentration, pH water temperature, electrolyte concentrations and mixing.

7.4.1.1 Coagulant Dosage

Although there is some relation between turbidity of the raw water and the proper coagulant dosage, the exact quantity can be determined only by trial. Even thus determined, the amount will vary with other factors such as time of mixing and water temperature. The use of the minimum quantity of coagulant determined to be effective in producing good flocculation in any given water, will usually require a fairly long stirring periods varying from 15 to 30 minutes in summer and 30 to 60 minutes in the colder months, as water temperatures approach the freezing point.

Addition of coagulants in excess of the determined minimum quantity may increase bactericidal efficiency. It is, however, usually more economical to use the minimum quantity of coagulant and to depend on disinfectant for bacterial safety.

Very finely divided suspended matter is more difficult to coagualte than coarse particles, necessitating a larger quantity of coagulant for a given turbidity. The cation-exchange capacity of the particles of turbidity bears a significant relationship to the success of flocculation.

7.4.1.2 Characteristics of Water

The characteristics of water especially pH have considerable influence on the satisfactory formation of flocs. Some natural waters need certain adjustments in acidity or alkalinity of water.

7.4.1.2.1 Optimum pH Zone

There is at least one pH zone for any given water in which good flocculation occurs in the shortest time with a given dose of coagulant, or in a given time with the required minimum dose of coagulant. Coagulation should be carried out within this optimum zone using alkalis and acids for correction of pH wherever necessary. For many waters, usually those which are low in colours and well buffered and having pH in the optimum zone, no adjustment of pH is necessary. However, in waters of low mineral content, or in the presence of interfering organic matter, constant attention is needed for pH adjustment. Failure to operate within the optimum zone, may be a waste of chemicals and may be reflected in the lowered quality of the plant effluent. As a result of studies of the effect of pH on coagulation, it has been found that "the more dilute the water in total dissolved solid and the less the alum added, the narrower becomes the pH zone".

In the case of coagulation with alum, the control over the alkalinity is very important. Not only should the water contain sufficient alkalinity to completely react with the aluminium sulphate but there should be a sufficient residual to ensure that the treated water is not corrosive. A consideration of the reaction involved shows that one molecule of "filter alum" (molecular weight of Al_2 $(SO_4)_3$. 18 H_2O = 666 requires three molecules of calcium bicarbonate [Ca $(HCO_3)_2$] x 3 = 486 for complete reaction.

If the alkalinity is expressed in terms of calcium carbonate, the theoretical requirement of 666 parts of "filter alum" works out to 300 parts of alkalinity, i.e. approximately in the ratio of 2:1. This reduction of alkalinity should be taken into consideration and sufficient alkalinity should be added to the water, if necessary. For this purpose, hydrated lime $Ca(OH)_2$ is usually added, or "soda ash" (Na_2CO_3) may be used when the increase of hardness is to be avoided.

When ferrous sulphate is used as a coagulant, the pH should be maintained above 9.5 to ensure complete precipitation of the iron. This is done by the addition of hydrated lime. For this reason, the process is sometimes known as "iron and lime process".

7.4.1.3 Coagulant Aids

Coagulant aid is a chemical, which when used along with main coagulant, improves or accelerates the process of coagulation and flocculation by producing quick-forming, dense and rapid-settling flocs.

Finely divided clay, fuller's earth, bentonites and activated carbon arc the most commonly used materials as nuclei to floc formation. The particles may become negatively charged making them subject to attraction by the positively charged aluminium ion.

Activated silica, i.e. sodium silicate activated with aluminium sulphate, sulphuric acid, carbon dioxide or chlorine, when applied to water, produces a stable solution having a high negative charge which unites with the positively charged alum or other floc to make it denser and tougher. It is especially useful for clear water that do not coagulate well with the usual processes. It has a wider range of use in water softening.

Polyelectrolytes which are polymers containing ionisable units have been used successfully as both coagulant aids and coagulants but care should be taken to guard against their toxicity. They are soluble in water, conduct electricity and are affected by the electrostatic forces between their charges. Cationic, anionic and ampholytic polyelectrolytes have been used; the cationic being able to serve as both a coagulant and coagulant aid while the other two as coagulant aids primarily. Polyelectrolytes create extraordinarily slippery surfaces when spilled on floor and are difficult to clean up.

Toxicity of any polyelectrolyte has to be checked before it can be used as coagulant or coagulant aid.

7.4.1.4 Choice of Coagulant

In selecting the best coagulant for any specific treatment problem, a choice has to be made from among various chemicals, each of which may offer specified advantages under different conditions. The common coagulants used in water works practice are salts of aluminium viz. filter alum, sodium aluminate and liquid alum and iron salts like ferrous sulphate (Copperas), ferric sulpnate, ferric chloride and chlorinated copperas which is an equimolecular mixure of ferrous sulphate and ferric chloride being obtained by chlorinating ferrous sulphate. Some coagulants derived from natural products such as Nirmali seeds have also been used.

Selection of aluminium or iron coagulants is largely decided by the suitability of either type and its easy availability. Both filter alum and ferric sulphate have certain specific advantages. Alum does not cause the unsightly reddish brown staining of floors, walls and equipment which may result when iron salts are used; nor is its solution as corrosive as the ferric form of iron salts. The dissolving of ferric sulphate also offers difficulties not encountered with alum. The trivalent aluminium ion is not reduced to a more soluble bivalent iron, as may be the case when ferric salts are used with waters high in organic matter. On the other hand, ferric floc is denser than alum floc and is more completely precipitated over a wider pH range. Also good flocculation with alum is not possible in some waters.

The choice of the coagulant to be used for any particular water should preferably be based upon a series of jar tests, so planned that it will permit accurate comparison of the materials being studied under identical experimental conditions. The coagulant dose in the field should be judiciously controlled in the light of the jar test values.

A few of the many substances used in coagulation of water are listed in Appendix 7.10

7.4.2 Rapid Mixing

Rapid mixing is an operation by which the coagulant is rapidly and uniformly dispersed throughout the volume of water, to create a more or less homogeneous single or multiphase system. This helps in the formation of microflocs (Perikinetic flocculation) and results in proper utilisation of chemical coagulant preventing localisation of concentration and premature formation of hydroxides which lead to less effective utilisation of the coagulant. The chemical coagulant is normally introduced at some point of high turbulence in the water. The source of power for rapid mixing to create the desired intense turbulence are gravitational, mechanical and pneumatic.

The intensity of mixing is dependent upon the temporal mean velocity gradient, 'G'. This is defined as the rate of change of velocity per unit distance normal to a section (or relative velocity of two flow lines divided by the perpendicular distance between them) and has the dimensions of and generally expressed as s^{-1}. The turbulence and resultant intensity of mixing is based on the rate of power input to the water and G can be measured or calculated in terms of power input by the following expression:

$$G = \sqrt{P / \mu(vol)} \qquad (7.7)$$

Where,

G = Temporal mean Velocity gradient, s^{-1};

P = Total input of power in water, watts;

μ = Absolute viscosity of water, $N.s/m^2$; and

Vol = Volume of water to which power is applied, m^3.

Where head loss through the plant is to be conserved as much as possible and where the flow exceeds 300 m^3/hr, mechanical mixing also known as flash mixing, is desirable. Multiple units may be provided for large plants. Normally a detention time of 30 to 60 seconds is adopted in the flash mixer. Head loss of 0.2 to 0.6 m of water, which is approximately equivalent to 1 to 3 watts per m^3 of flow per hour is usually required for efficient flash mixing. Gravitational or hydraulic devices are simple but not flexible, while mechanical or pneumatic devices are flexible, but require external power.

7.4.2.1 Gravitational Or Hydraulic Devices

In these devices, the required turbulence is obtained from the flow of water under gravity or pressure. Some of the more common devices are described below.

(a) Hydraulic jump Mixing

This is achieved by a combination of a chute followed by a channel with or without a sill. The chute creates super critical flow (velocity 3 to 4 m/s), the sill defining the location of the hydraulic jump and the gently sloping channel induces the jump. Standing wave flumes specially constructed for measurement of flow can also be used in which the hydraulic jump takes place at the throat of the flume. In the hydraulic jump mixing, loss of head is appreciable (0.3 m or more) and the detention time is brief. This device though relatively inflexible, is simple and free from moving parts. This can be used as a standby in large plants to the mechanical mixers while for small plants, this can serve directly as the main unit. Typical residence time of 2 seconds and G value of 800 s^{-1} have been reported. overflow weirs have also been used for rapid mixing A head loss of 0.3 to 0.6 m across the weir has been reported.

(b) Baffled Channel Mixing

In this method, the channel section (neglecting the baffle) is normally designed for a velocity of 0.6 m/s.

The angle subtended by the baffle in the channel is between 40° to 90° with the channel wall. This angle should ensure a minimum velocity of 1.5 m/s while negotiating the baffle.

The main walls of the channels are constructed of brick masonry, stone masonry or reinforced cement concrete finished smooth to avoid growing of weed etc. The baffles are made of concrete or brick, finished in the same manner as the channel. A minimum free board of 150 mm is normally provided.

(c) Other Type of Hydraulic Mixing

Sudden drop in hydraulic level of water over a weir can cause turbulence and chemicals can be added at this "plunge" point with the aid of diffusers. Similarly in pressure conduits, the chemicals can be added at the throat of a venturi or just upstream of orifice located within the pipe. In this system, no effective control is possible even though mixing takes place. Rapid mixing can also be obtained by injection of chemicals preferably, in the suction end or delivery end of low lift pumps where the turbulence is maximum. In this system also, the detention time is brief while the cost is low.

7.4.2.2 Mechanical Devices

There are two devices, the usual one being the rapid rotation of impellers or blades in water and the other mixing with the aid of a jet or impingement over a plate. Propeller type impellers are commonly employed in flash mixers, with high revolving speeds ranging from 400 to 1400 rpm or more. The blades are mounted on vertical or inclined shaft and generate strong axial currents. Turbine types and paddle types are also used. In the design of a mechanical flash mixer unit, a detention time of 30 to 60,sec. is provided. The relatively high powered mixing devices should be capable of creating velocity gradients of 300 s^{-1} or more. Power requirements are ordinarily 1 to 3 watts per m^3/hr of flow. Usually, the flash mixers are deep, circular or square tanks. The usual ratio of impeller diameter to tank diameter is 0.2 to 0.4 and the shaft speed of propeller greater than 100 rpm imparting a tangential velocity greater than 3 m/s at the tip of the blade. The ratio of tank height to diameter of 1: 1 to 3: 1 is preferred for proper dispersal.

Vertical strips or baffles, projecting 1/10 to 1/12 tank diameter, at minimum of four places, along the walls of the tank should be provided to reduce vortex formation or rotational movement of water about the impeller shaft. The mixing chamber can be placed below the chemical feed floor ensuring short chemical feedlines. The usual mechanical agitator drive is an electric motor with continuous duty, operating through a reduction gear. Good results are achieved by adding the chemical just near the tip of the blade or the propeller in the tank. Mechanical type consumes very little head of water and permits flexibility of operation. When there is possibility of short circuiting in the tank, one more compartment may be provided. This requires more external power input and needs constant attention and maintenance.

In the impingment type, water is forced as jet through a nozzle, impinging on a plate where the chemical is added. An auxiliary pump is used to create the jet action. The rapid mixing takes place at the point of impingement where turbulence occurs. The power requirement of the auxiliary pump should be workd out in accordance with Table 7.1. Rapid mixing channels may be obviated by pipe mixing preferably with orifice (instead of venturi) or with a mechanical impeller through a stuffing box into a pipe.

7.4.2.3 Pneumatic Devices

When air is injected or diffused into water after suitable compression, it normally expands isothermally and the resultant work done by the air can be used for necessary agitation. They are not common in water works practice. The typical range of velocity gradients and contact times are in the range of 3000 to 5000 s and 0. 5 to 0. 4 sec. respectively.

Taking into account the various types of rapid mixing devices velocity gradients and the detention times, the following equation is proposed:

$$G = 2790\ t^{-0.35} \qquad (7.8)$$

where,

G = Velocity gradient, s-1

t = Detention time, S

In the field, it has been observed that the detention time reduces much faster with increase in the value of G. Hence the G.t value instead of remaining constant reduces with increase in G value. Equation 7.8 is based on this field experience. Variation in the value of G could be from 300 s^{-1} to 5000 s^{-1}.

7.4.3 SLOW MIXING OR STIRRING

Slow mixing is the hydrodynamic process which results in the formation of large and readily settleable flocs (orthokinetic flocculation) by bringing the finely divided matter into contact with the microflocs formed during rapid mixing. These can be subsequently removed in settling tanks and filters.

TABLE 7.1

RECOMMENDED DETENTION TIME AND NET POWER REQUERED

Detention Time	Velocity gradient	Net Power input per unit volume	Net Power input per unit discharge
S	s^{-1}	watts/m^3 of volume	watts/m^3 of flow/hr
60	300	72	1.2
50	360	104	1.4
40	450	162	1.8
30	600	288	2.4
25	720	415	2.9
20	900	648	3.6

Note: Power calculations are based on water temperature of 30°C (μ =0.8x10^{-3} N.S./m^2)

7.4.3.1 Design Parameters

The rate at which flocculation proceeds depends on physical and chemical parameters such as charges on particles, exchange capacity, particle size and concentration, pH, water temperature, electrolyte concentration, time of flocculation, size of mixing basin and nature of mixing device. The influence of these and other unknown factors which vary widely for different waters, is not yet fully understood. Information on the behaviour of the water to be treated can be had by examination of nearby plants treating similar water and by laboratory testing using Jar Test.

The physical forces of slow mixing of the coagulant fed water and adhesion, controlled by chemical and electrical forces are responsible to a large extent in influencing the flocculation processess.

Slow mixing is meant to bring the particles to collide and then agglomerate. The rate of collision among the particles is dependent upon the number and size of particles in suspension and the intensity of mixing in the mixing chamber.

Since flocculation is a time-rate process, the time provided for flocculation to occur is also significant factor in addition to the intensity of agitation and the total number of particles. The number of collisions is proportional to G.t where t is the detention time of the flocculation basin. The product G.t is non-dimensional and is a useful parameter for the design and operation of flocculation.

The desirable values of G in a flocculator vary from 20 to 75 s^{-1} and G.t from 2 to $6x10^4$ for aluminium coagulants and 1 to $1.5x10^5$ for ferric coagulants. The usual detention time, provided, varies from 10 to 30 minutes. Very high G values tend to shear flocs and prevent them from building to size that will settle rapidly. Too low G values may not be able to provide sufficient agitation to ensure complete flocculation.

Another useful parameter is the product of G.t and the floc volume concentration 'C' (Volume of floc per unit volume of water). This parameter G.C.t reflects to a certain extent the contact opportunity of the particles but the usefulness of this parameter is not yet fully established. The values are of the order of 100.

To ensure maximum economy in the input of power and to reduce possible shearing of particles floc formation, tapered flocculation is sometimes practised. The value of G in a tank is made to vary from 100 in the first stage to 50 or 60 in the second stage and then brought down to 20 s^{-1} in the third stage in the direction of flow.

7.4.3.2 Types Of Slow Mixers

Similar to rapid mixing units, these can be categorised under gravitational or hydraulic, mechanical and pneumatic. The hydraulic type uses the kinetic energy of water flowing through the plant created usually by means of baffles, while mechanical type uses the external energy which produces agitation of water.

(1) Gravitational or Hydraulic Type Flocculators

Several types of gravitational or hydraulic flocculators are used in practice.

(a) Horizontal Flow Baffled Flocculator

Fig. 7.2 shows the plan of a typical horizontal flow baffled flocculator. This flocculator consists of several around-the-end baffles with in between spacing of not less than 0.45 m to permit cleaning. Clear distance between the end of each baffle and the wall is about 1.5 times the distance between the baffles, but never less than 0.6m. Water depth is not less than 1.0 m and the water velocity is in the range of 0.10 to 0.30 m/s. The detention time is between 15 and 20 minutes. The flocculator is well suited for very small treatment plants. It is easier to drain and clean. The head loss can be changed as per requirement by altering the number of baffles. The velocity gradient can be achieved in the range 10-100s^{-1}.

(b) Vertical Flow Baffled Flocculator

Fig. 7.3 shows the cross section of typical vertical flow baffled flocculator. The distance between the baffles is not less than 0.45 m. Clear space between the upper edge of the baffles and the water surface or the lower edge of the baffles and the basin bottom is about 1.5 times the distance between the baffles. Water depth varies between 1.5 to 3 times the distance between the baffles and the water velocity is in the range 0.1-0.2m/s. The detention time is between 10-20 minutes. This flocculator is mostly used for medium and large size treatment plants.

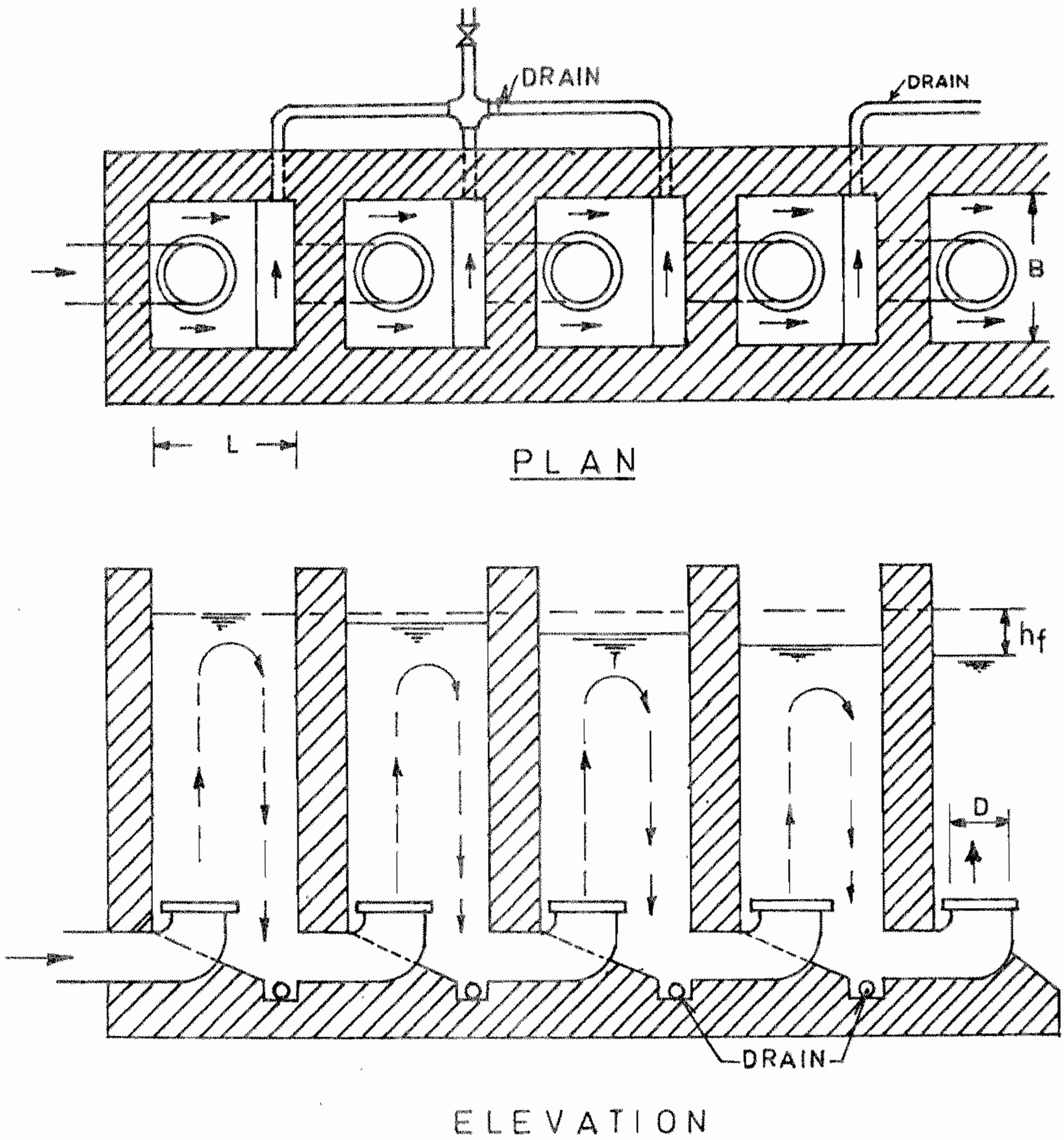
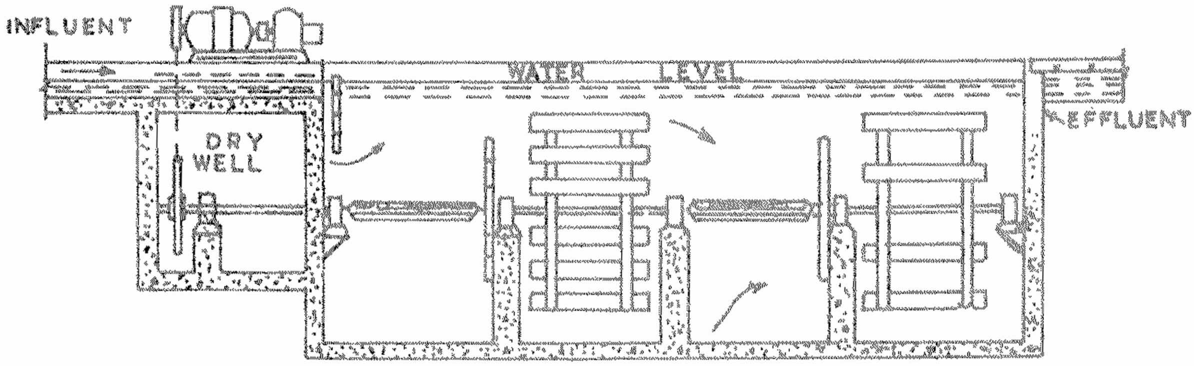
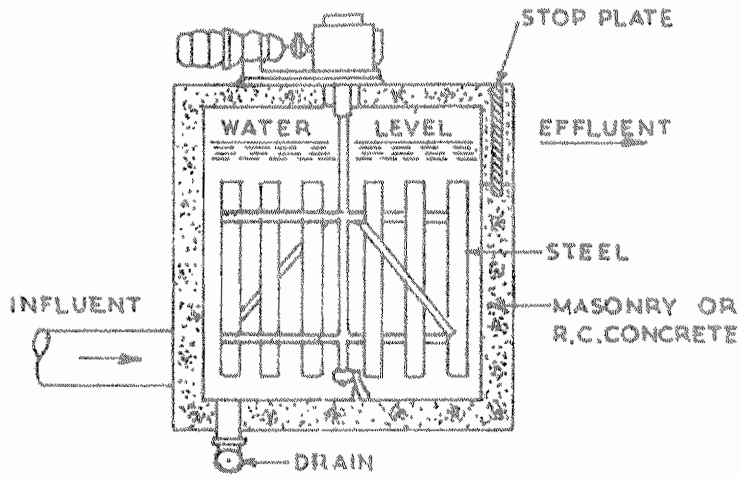


FIGURE 7.5 : ALABAMA TYPE FLOCCULATOR

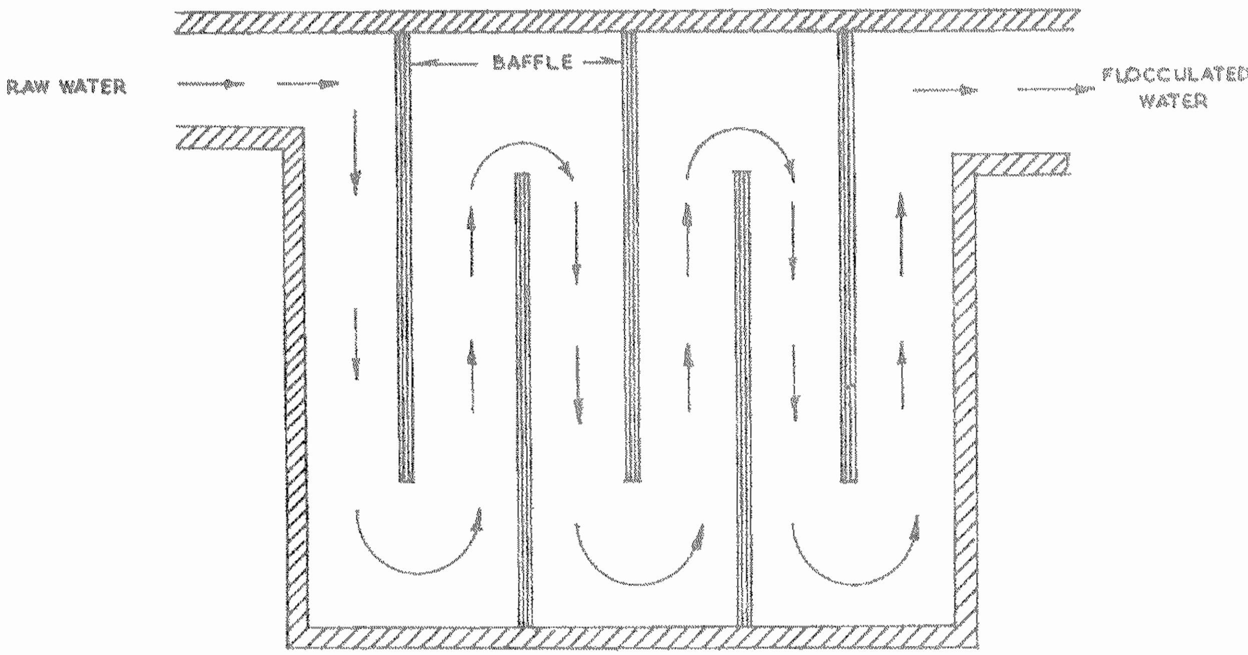


(a) FLOCCULATOR LONGITUDINAL FLOW



(b) VERTICAL FLOCCULATOR

FIGURE 7.6 : MECHANICAL TYPE FLOCCULATOR WITH PADDLES



PLAN

FIGURE 7.2 : HORIZONTAL FLOW BAFFLED FLOCCULATOR

(c) *Hydraulic jet Action Flocculator*

This is a less known type of hydraulic flocculator and is suitable for small treatment plants.

In one type of jet flocculators, shown in Fig. 7.4, the coagulant (alum) is injected in the raw water using a special orfice device at the inlet bottom of the tank. Water is then let into this hoppered tank. Heliccoidal-flow (also called tangential-flow or spiral-flow) type as well as staircase type flocculators can also be used.

(d) Alabama-Type Flocculator

An Alabama-type flocculator shown in Fig. 7.5. is a hydraulic flocculator having separate chamber in series, through which the water flows in two directions. Water flows from one chamber to another, entering each adjacent partition at the bottom and through outlets facing upwards. This flocculator was initially developed in Alabama State, U.S.A. (hence the name) and was later introduced in Latin America.

For effective flocculation in each chamber, the outlets are placed at depth of about 2.50 m below the water level. The loss of head is normally about 0.35 to 0.50 m for the entire unit. The range for the velocity gradient is 40 to 50 s^{-1}. The common design criteria are: Rate capacity per unit chamber = 90 to 180 $m^3/m/hr$; velocity at turns = 0.40 to 0.60 m/s; length

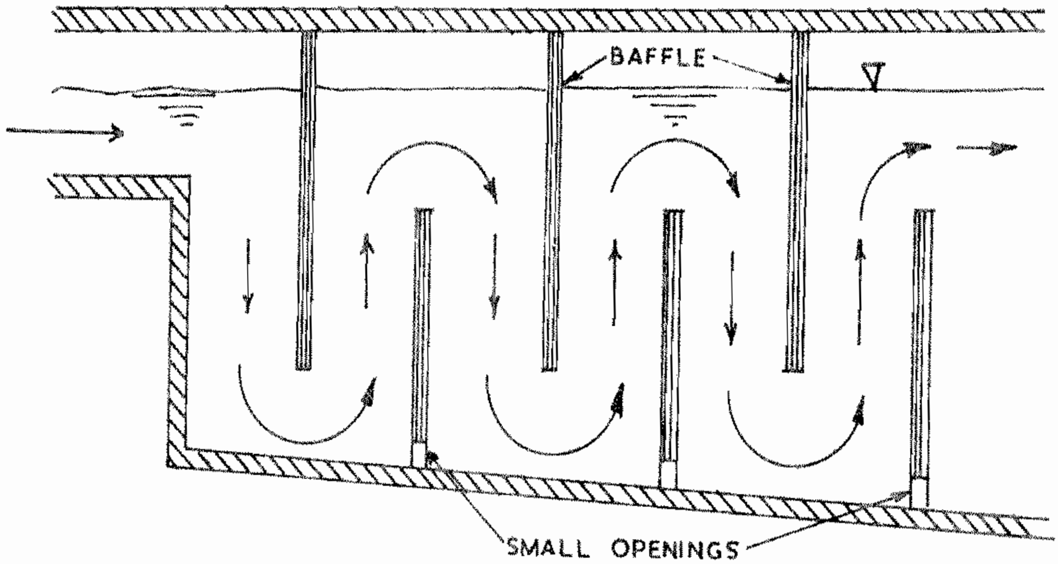


FIG. 7.3 VERTICAL FLOW BAFFLED FLOCCULATOR

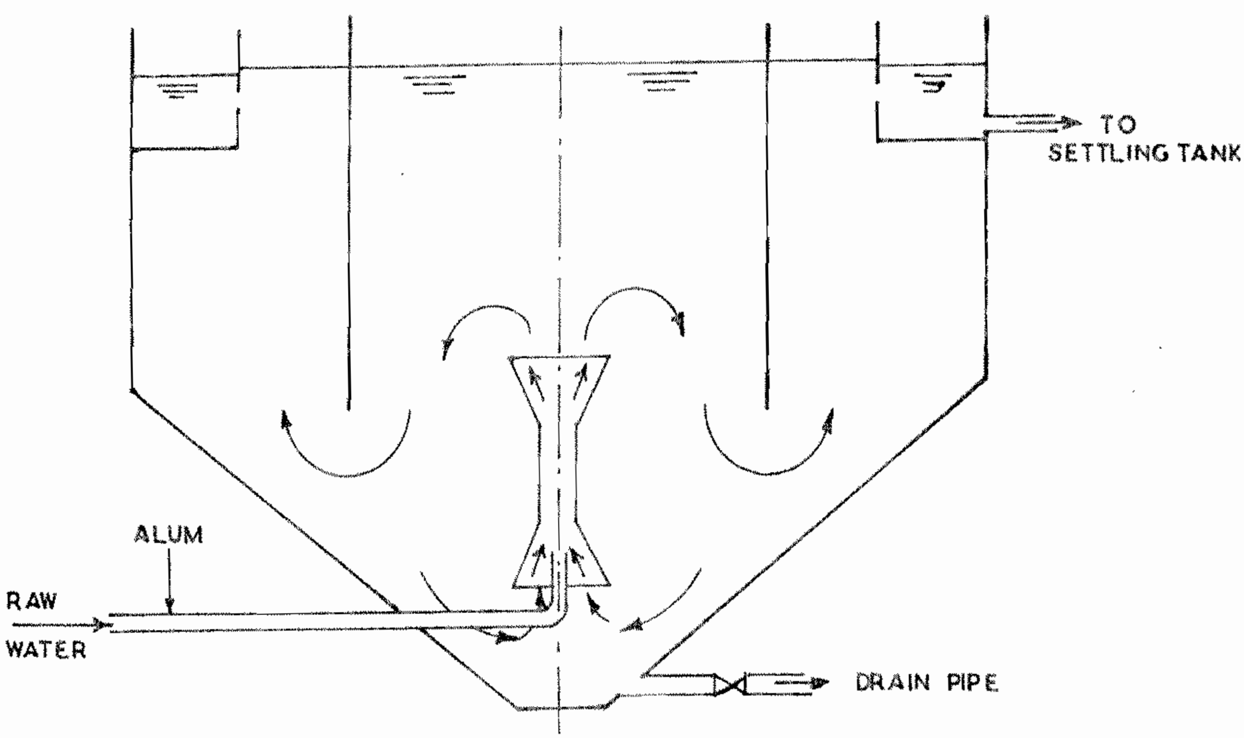


FIGURE 7.4 : JET FLOCCULATOR

of unit chamber = 0.75 to 1.50 m; width = 0.50 to 1.25 m; depth = 2.50 to 3.50 m; and detention time = 15 to 25 min.

(e) Tangential Flow Type

Water is introduced tangentially at an inclination in a square tank with chamfered corners to induce a circulatory motion, thus resulting in turbulence and mixing. Chances of short circuiting are high and intimate mixing may not be obtainable.

(f) Pipe Flocculators

The turbulence during the flow through a pipe can create velocity gradients leading to flocculation. The mean velocity gradient is calculated from

$$G = \left[\frac{\rho g Q h_f}{Vol(\mu)}\right]^{\frac{1}{2}} \tag{7.9}$$

in which Q = flow rate, m^3/s; Vol = Volume of pipe of length L in m^3, and h_f = headloss in pipe of length l ; $h_f = \dfrac{flv^2}{2gd}$

Where v = Velocity, m/s; f = friction factor for the pipe; d = diameter of pipe, in m.

(2) Mechanical Type flocculator

Paddle flocculators are widely used in practice. Fig. 7.6 shows two types of mechanical type flocculator with paddles. The design criteria are: depth of tank = 3 to 4.5 m; detention time, t = 10 to 40 min. normally 30 min; velocity of flow = 0.2-0.8 m/s normally 0.4 m/s; total area of paddles = 10 to 25% of the cross-sectional area of the tank; range of peripheral velocity of blades = 0.2-0.6 m/s; 0.3-0.4 m/s is recommended; range of velocity gradient, G = 10 to 75 s^{-1}, range of dimensionless factor Gt = 10^4 -10^5 and power consumption; 10.0 to 36.0 kw/mld, outlet velocity to settling tank where water has to flow through pipe or channel = 0.15 to 0.25 m/s to prevent settling or breaking of flocs. For paddle flocculator, the velocity gradient is given by

$$G = \left[\frac{1}{2}\cdot\frac{C_D A_P \rho (V_P - V_W)^3}{\mu(vol)}\right]^{\frac{1}{2}} \tag{7.10}$$

In which C_D = coefficient of drag (0.8 to 1.9), A_P = area of paddle (m^2), Vol = volume of water in the flocculator (m^3), V_P = velocity of the tip of paddle (m/s), Vw = Velocity of the water adjacent to the tip of paddle (m/s).

The optimum value of G can be calculated

$$G_{opt}^{2.8}\, t.c = 44 \times 10^5 \tag{7.11}$$

In which G = optimum velocity gradient, s^{-1}, t = time of flocculation, min.; and c = alum concentration (mg/1).

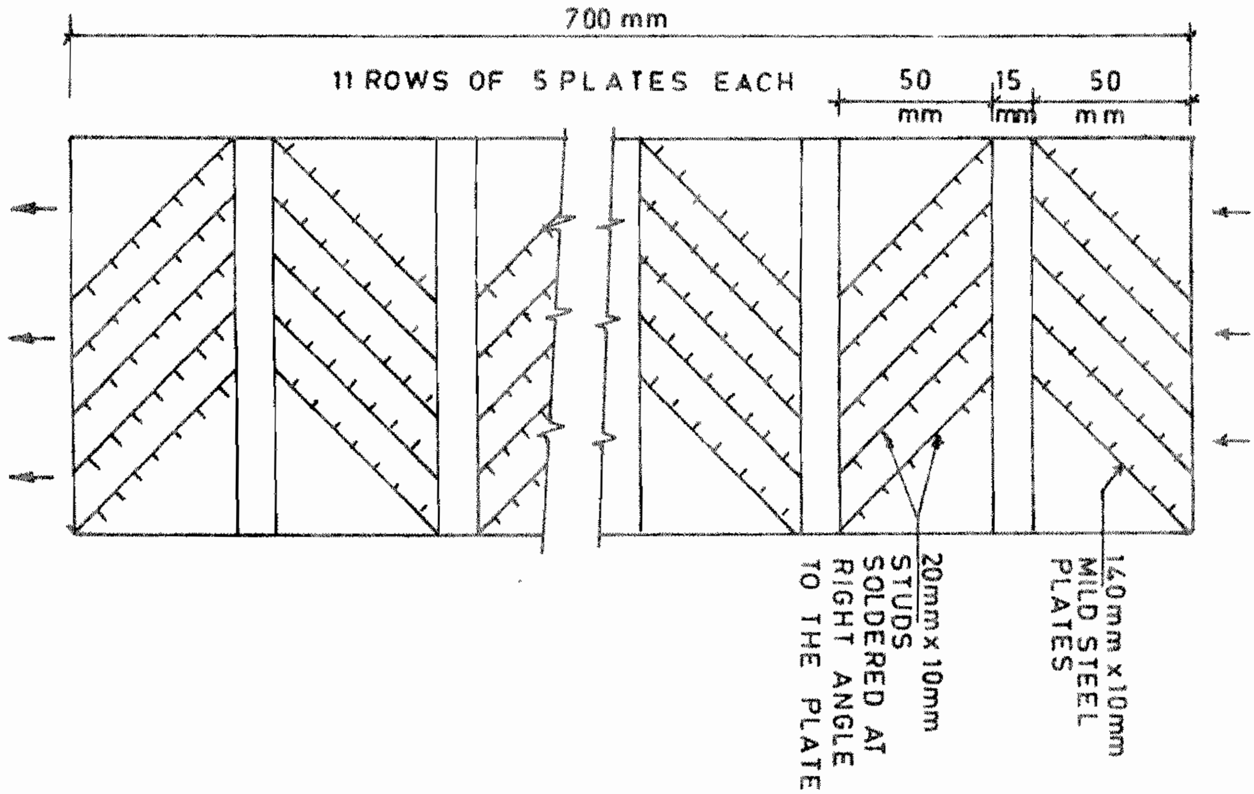


FIGURE 7.7 : SURFACE CONTACT FLOCCULATOR

In large plants, it is desirable to provide more than one compartment in series to lessen the effect of short circuiting. While translating laboratory jar test data to plant scale, it must be borne in mind that the good mixing conditions available in the laboratory cannot be simulated in the plant.

The paddles can be driven by electric motors or by turbines rotated by water fall when sufficient head is available. The direction of flow is usually horizontal moving parallel or at right angles to the paddle shafts. The shape of the container also affects the process of flocculation. For the same volume and height of water in the containers of several shapes such as circular, triangular, square, pentagonal and hexagonal, it was observed that the pentagonal shape gave the best performance.

Introduction of stators in the flocculator helps to improve the performance of flocculation.

(3) Pebble Bed Flocculator

The pebble bed flocculator contains pebbles of size ranging from 1 mm to 50 mm. Smaller the size of the pebbles, better is the efficiency, but faster is the build up of the headloss and vice-versa. The depth of the flocculator is between 0.3 to 1.0 m.

The velocity gradient is given by

$$G = \left[\frac{\rho g Q h_f}{\alpha \mu A}\right]^{\frac{1}{2}} \tag{7.12}$$

In which

h_f = Head loss across the bed (m);

α = Porosity of bed;

A = Area of flocculator (m^2); and

L = Length of the bed (m)

The main advantage of the pebble bed flocculator is that it requires no mechanical devices and electrical power. The operation and maintenance cost is also low. The drawback of this flocculator is that there is gradual build up of the head loss across the pebble bed and therefore needs periodical cleaning by simultaneous draining and hosing.

(4) Fluidized Bed Flocculator

In a fluidized flocculator the sand bed is in the fluidized form. Even a 10% expansion of the sand bed is enough to create the required turbulence without chocking the media. The sand size is between 0. 2 to 0. 6 mm and depth of sand bed is between 0.3 to 0.6 m. The flow of water is upwards. This flocculator also does not require any mechanical equipment or electrical power. Further, there is no build up of the head loss across the bed.

(5) Pneumatic Flocculator

In a pneumatic flocculator, air bubbles are allowed to rise through a suspension. This creates velocity gradient useful for flocculation. The velocity gradient can be calculated from

$$G = 0.236\frac{gD\rho}{\mu}\left[\frac{Vol_A}{Vol}\right]^{\frac{1}{2}} \tag{7.13}$$

In which

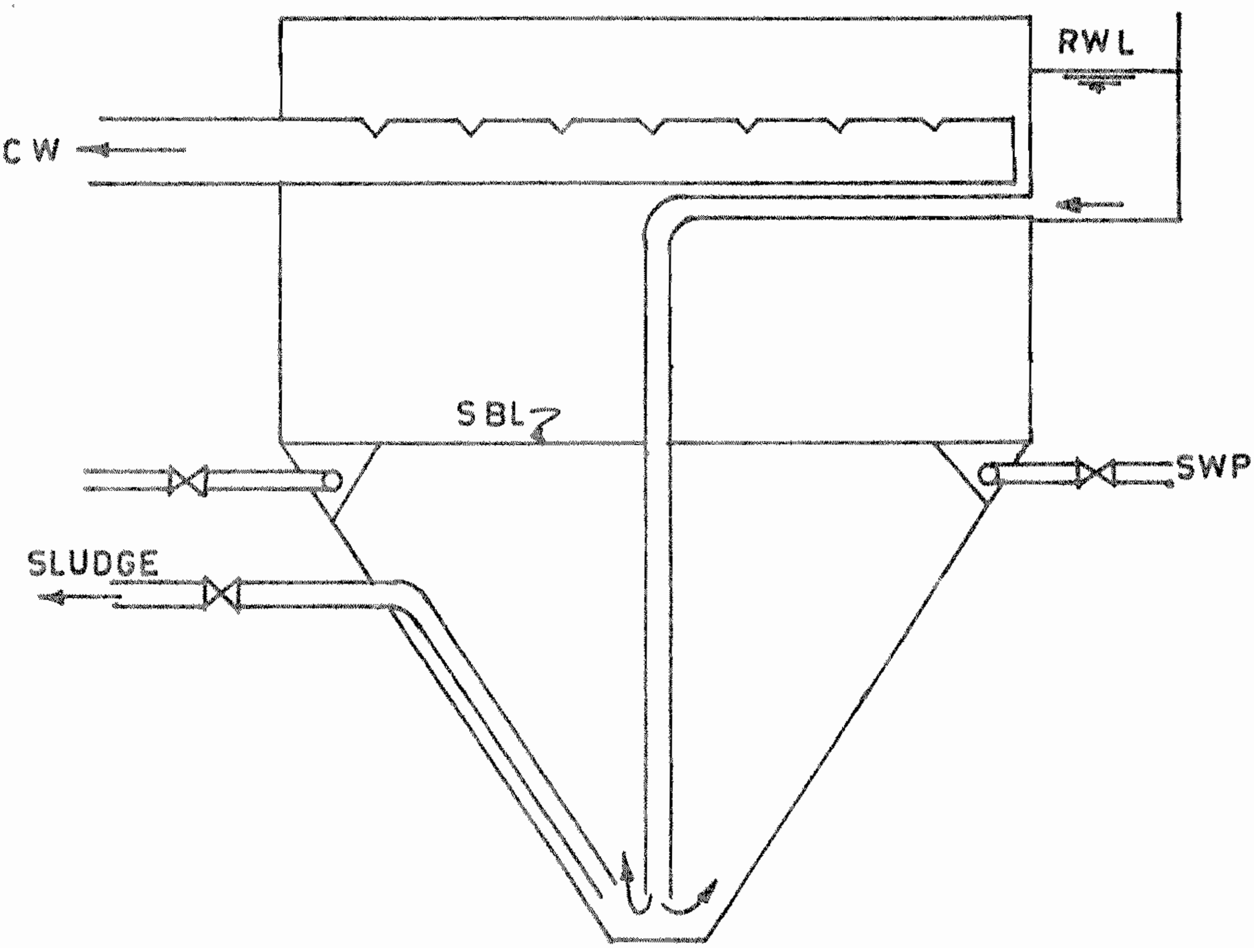
D = diameter of air bubbles (m); and (VoI_A/Vol)= volume of air supplied per unit water volume.

The flocculator needs air compressor and the problem of clogging of diffuser is quite common. It is less efficient than the paddle -flocculator and therefore not commonly used.

(6) Surface Contact Flocculator

The surface contact flocculator was studied experimentally in India to overcome the inherent problem of choking, which increases the head loss over a period of time in pebble bed flocculators.

The surface contact flocculators consist of studded plates, placed in a zigzag form along the direction of flow. An experimental flocculator, shown in Fig. 7.7, comprised of 55 mild steel plates, 140 mm x 60 mm in size, arranged in 11 rows of 5 plates each. These plates were fixed at 45° to a base plate in zigzag fashion. The flocculator was tested in a continuous



LEGEND

RWL	RAW WATER LEVEL
CW	CLEAR WATER
SBL	SLUDGE BLANKET LEVEL
SWP	SLUDGE WASTE PIPE

FIGURE 7.8(A) : SLUDGE BLANKET CLARIFIER

downflow system with velocity of flow ranging from 5 m/h to 25 m/h and turbidity ranging from 50 to 1600 NTU. Further work in this direction is necessary.

Another type of surface contact flocculator is made of PVC Pipes. These pipes either square or circular in cross-section are cut longitudinally in two equal pieces, length of each piece being 1m. These pieces are then tied with copper wire in perpendicular directions in alternate layers. Suitable gaps are provided between the pieces. The whole assembly of the pipes can be called as floc module. The depth of the module can be between 2.0 to 2.5 m to provide about 15 to 20 minutes of detention time. The modules are housed in a square or rectangular chamber with hopper bottoms. The top of the module is about 200 to 300 mm below the water level in the chamber. The modules can be supported by mild steel grating fabricated from 50 mm x 50 mm mild steel flats welded in vertical positions to mild steel angle frame of 55 mm x 50 mm x 5 mm size angles. The grating can be placed 200 mm above the top of hopper level. The settled sludge can be periodically removed from the hopper.

(7) *Inline Flocculator*

An Inline static flocculator, or an inline static mixer is a relatively recent device. it is housed in a gravity main and is static. The head loss in an inline flocculator is comparatively less and the maintenance cost is also almost negligible. Only occassional flushing is necessary since deposition of some flocs takes place. The capitalized cost of typical inline flocculator is one-third of the capitalized cost of conventional mixing impellers. Laboratory experiments show that twisted aluminium plates as static mixer in the pipeline give better performance compared to the semicircular plates or the inclined plane plates.

(8) *Sludge Blanket Clarifier*

A sludge Blanket clarifier includes both flocculation and clarification. Flocculators are generally independent of the settling tanks that follow. They can also be installed such that both the functions are performed in a single unit, though in different zones. In the case of rectangular tanks, the bottom portion can be used for flocculation and the top for sedimentation. In circular tanks, flocculators are at the centre and the flow is vertical.

The more common form of the combination unit is the up-flow clarifier which combines one of flocculation or solids contact unit of either a sludge blanket filtration (also called contact filtration) or a slurry recirculation type with sedimentation. Though there are differences amongst them, all of them seek to take advantage of the mass action effect of floc formation in the presence of previously formed masses of floc. In such basins which are usually deep, vertical flow is induced from the bottom and the decanted water is skimmed out from the top. For floc build-up, inlet and sludge zones are in close contact and the flocculation zone is occupied in part or as a whole by a blanket of flocs. Rising flocs come into contact with both the settling flocs and the stationary blanket of flocs which is in equilibrium with the hydraulic environment. Agglomeration of flocs thus takes place by direct contact.

The bottom section is devoted to mixing of incoming water with chemicals. In the sludge blanket type, chemicals are directly fed into the blanket. All chemical reactions occur in the blanket so that the newly formed insoluble salts precipitate directly on the sludge particles already present. In this manner a completely flocculated system is constantly maintained and a type of sludge is produced which settles very rapidly and results in completely "cracked" water. At the same time, the filtering action of the blanket traps the finer particles.

The clarfication zone extends from the top of the sludge blanket to the surface of the liquid. Upon emergence from the sludge blanket, the water passes through this clarification zone and is collected for use.

From time to time the excess sludge is withdrawn either by gravity or by pumping. For larger tanks, it is advisable to provide mechanical scrapers for removal of the settled solids.

Several designs of the "Solids Contact Units" are available and they are fundamentally similar in design in that they combine solids contact mixing, flocculation, solids liquid separation and continuous removal of sludge in a single basin. The general design features are:

i) Rapid and complete mechanical mixing of chemicals, raw water and suspension of solids;

ii) Provision of mechanical means for constant circulation of large volumes of liquid containing the solids being used for contact. This is achieved either inside the tank by an impeller in the inner compartment or in the outer compartment used for settlement. In other types, the solids from the clarification zone are removed and mixed with the raw water in a chamber located outside. Rapid sludge recirculation ensures quick mixing with incoming water; and

iii) Operation at higher than conventional flow rates.

As the efficiency of this type depends on the formation of a sludge blanket, skilled and delicate operation for control is needed. The turbidity of raw water that can be applied to the Solids Contact Unit is limited to 700 to 1000 NTU. These are not advisable for the high algae laden water. A typical sketch of the plant is shown in Fig. 7.8 (a). The different problems involved in the conventional clarifier are in connection with the dosing and mixing, desludging and the stability of the blanket. An attempt was made in India to overcome these inherent defects, through a modified sludge blanket clarifier, shown in Fig. 7.8 (b).

The velocity gradient of the sludge blanket can be calculated from

$$G = \left[\frac{\rho g}{\mu}(S_s - 1)(1-\alpha)h \Big/ \frac{Vol}{Q}\right]^{\frac{1}{2}} \tag{7.14}$$

In which S_s = specific gravity of flocs; α =porosity of blanket; h = depth of blanket (m); Vol. = capacity of clarifier (m^3); and Q = rate of flow (m^3/s).

(9) Tapered Velocity Gradient Flocculator

In a tapered velocity gradient flocculator, the water is intially subjected to a high velocity gradient and finally to a low velocity gradient, thus generating dense, large size and tough flocs which in turn settle more quickly.

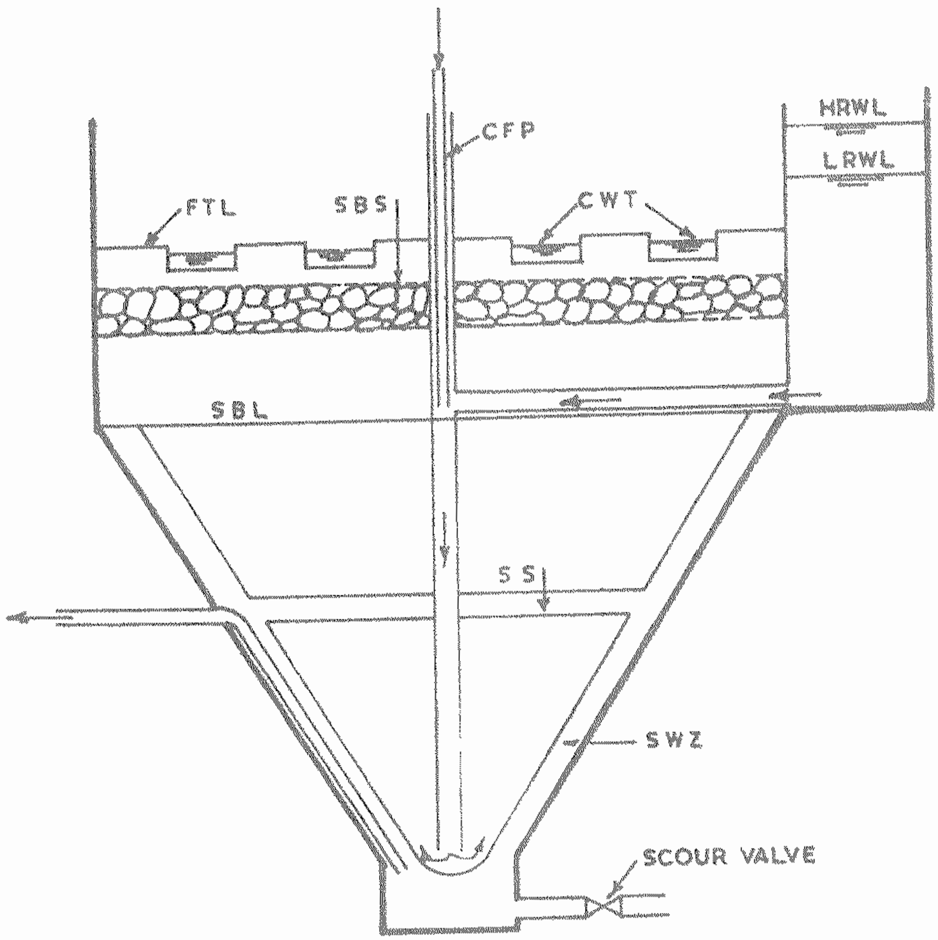


FIGURE 7.8(B) : MODIFIED SLUDGE BLANKET CLARIFIER

Recent studies indicated that the efficiency of a tapered velocity gradient flocculator increases when (a) there is increase in the range of the velocity gradient, the mean value of G remaining the same (b) there is gradual decrease in the velocity gradient and no sudden change of velocity gradient along the direction of flow, and (c) dual tapering, i.e. tapering in velocity gradient as well as time of flocculation is achieved, i.e. highest velocity gradient for the shortest time, followed by little lower value of G, velocity gradient, for a little more time and so on, so that in the end the value of the velocity gradient is the least with the maximum time of floccuation.

7.5 SEDIMENTATION

Sedimentation is the separation from water by gravitational settling of suspended particles that are heavier than water. It is one of the most commonly used unit operation in the flow sheet of conventional water treatment. Sedimentation (settling or clarification) is used to remove readily settling sediments such as sand and silt, coagulated impurities such as colour and turbidity and precipitated impurities such as hardness and iron. When suspended solids are separated from the water by the action of natural forces alone i.e. by gravitation with or without natural aggregation, the operation is called plain sedimentation. Plain sedimentation is usually employed as a preliminary process to reduce heavy sediment loads from highly turbid raw waters prior to subsequent treatment processes such as coagulation/filtrtation. Finely divided solids and colloidal particles, which cannot be removed by plain sedimentation within commonly used detention periods of few hours, are converted into settleable flocs by coagulation and flocculation and subsequently settled in sedimentation tanks.

The factors that influence sedimentation are :

a) Size, shape, density and nature (discrete or flocculent) of the particles ;

b) Viscosity, density and temperature of water;

c) Surface over flow rate;

d) Velocity of flow;

e) Inlet and outlet arrangements,

f) Detention period; and

g) Effective depth of settling zone.

7.5.1 Types of Suspended Solids

In water treatment practice, three main types of suspended particles are to be separated from water. The first type of suspended particles are finely divided silt, silica and clay having specific gravities ranging from 2.65 for sand and 1.03 for flocculated mud particles containing 95 percent water. The grain size may be 0.002 mm or more. Alum and iron flocs constitute the second type of suspended solids. These absorb and entrain water and specific gravities for alum and iron flocs may range from 1.18 and 1.34 respectively to as little as 1.002. Floc particles range in size from submicroscopic to 1 mm or more. The third type is the precipitated crystals of calcium carbonate obtained from limesoda softening operations. Their specific gravity is 2.7 with particle size of 15 to 20 µm. However, due to absorption of water upto 75 %, the specific gravity reduce to 1.2 and formation of cluster of crystals increases the size to a typical value of 0.1 mm

Suspended particles may settle either as discrete or flocculant particles. Discrete particles do not change their size, shape or weight during settling. The settling velocity of discrete particles can be computed by well-defined mathematical relationships as described in section 7.5.2. On the contrary, flocculent particles tend to agglomorate forming clusters of different size, shape and weight. Though the density of these floc clusters decreases due to entrapment and absorption of water, they settle faster due to increased size.

7.5.2 Settling Velocity of Discrete Particles

The following equations may be used in arriving at settling velocity of discrete spherical particles;

Law	Equation	Applicable for range of	
		Reynolds Number, N_R	Particle size in mm for specific gravity of 2.65 and temp. of 20°C
Stock's (Laminar)	$V_S = \frac{g}{18}\left(\frac{\rho_s - \rho}{\mu}\right)d^2$	1	Upto 0.1
Hazen's (Transition)	$V_S = \left[\frac{4}{3}\frac{g}{C_D}\left(\frac{\rho_s - \rho}{\rho}\right)\right]^{0.5}$	1-1000	0.1-1.0
Newton's (Turbulent)	$V_S = \left[3.3g\left(\frac{\rho_s - \rho}{\rho}\right)d\right]^{0.5}$	10^3-10^4	Greater than 1

Where,

V_s = Settling velocity of particle, (L/T)

ρ_s = Mass density of the particle, (M/L^3)

ρ = Mass density of water, (M/L^3)

g = Acceleration due to gravity, (L/T^2)

d = Diameter of the particle, (L)

C_D = Dimensionless drag coefficient defined by

$$C_D = \frac{24}{N_R} + \frac{3}{\sqrt{N_R}} + 0.34 \qquad (7.15)$$

N_R = Reynolds number = $\frac{V_S d.\rho}{\mu}$ dimensionless

μ = Absolute or dynamic visosity of water $\left(\frac{M}{L.T}\right)$

7.5.3 Removal Efficiencies of Discrete and Flocculent Suspensions

The removal efficiency of a unisize discrete suspension settling in an ideal settling tank is given by the ratio of settling velocity of the particles, v_s, and the surface overflow rate (SOR) which is numerically equal to flow divided by the plan area of the basin. For an ideal sedimentation tank, SOR represents the velocity of settling of those particles which covers the depth of the basin in time equal to the theoretical detention period or the settling velocity of the slowest settling particles which are 100 percent removed.

When water contains discrete particles of different sizes and densities, the overall removal, R, of suspended particles is given by

$$R = \left(1 - P_0\right) + \frac{1}{V_0}\int_0^{P_0} V_S dp \qquad (7.16)$$

Where P_0 in portion of particles with a settling velocity $\leq V_0$, the surface overflow rate.

Flocculent particles coalesce during settling increasing the mass of particles which settle faster. The degree of flocculation depends on the contact opportunities which in turn are affected by the surface overflow rate, the depth of the basin, the concentration of the particles, the range of particle sizes and the velocity gradients in the system. To determine the removal efficiency of a flocculent suspension, no adequate mathematical equation exists and settling column analyses are to be performed.

Settling analyses of flocculent suspensions are performed in column at least 300 mm in diameter and having depth equal to that of proposed basin. The column usually has ports at 0.6 m interval for withdrawal of samples. The flocculent suspension for which the settling characteristics are to be determined is introduced into the column in such a way that a uniform distribution of particle size occurs from top to bottom. The settling is allowed to occur under quiescent conditions and at constant temperature to eliminate convection currents. Samples are withdrawn at various selected time intervals from different depths and analysed to determine the suspended solids concentrations. The percentage removals of suspended solids are computed at different times and depths and the percentage removal is plotted as a number against time and depth. The iso-percent removal curves are drawn in a similar manner as contours are drawn from spot levels.

A generalized plot is given in Fig. 7.9. The percentage removal for a given time, t, can be computed from the relationship:

$$\text{Percentage removal} = \frac{\left(R_1 + R_2\right)}{2}.\frac{\Delta h_4}{h} + \frac{\left(R_2 + R_3\right)}{2}\frac{\Delta h_3}{h} + \frac{\left(R_3 + R_4\right)}{2}.\frac{\Delta h_2}{h} + \frac{\left(R_4 + R_5\right)}{2}.\frac{\Delta h_1}{h} \qquad (7.17)$$

where, R_1, R_2, R_3, R_4 and R_5 are percent removals and R_1 is the percent removal at time t and at 100% depth.

The curves can also be used to determine the detention period, depth and surface overflow rate required to obtain a given percentage removal of flocculent particles.

7.5.4 Types of Tanks

The tanks may be categorized into horizontal flow tanks or vertical flow tanks on the basis of direction of flow of water in the tank. The tanks may be rectangular, square or circular in plan.

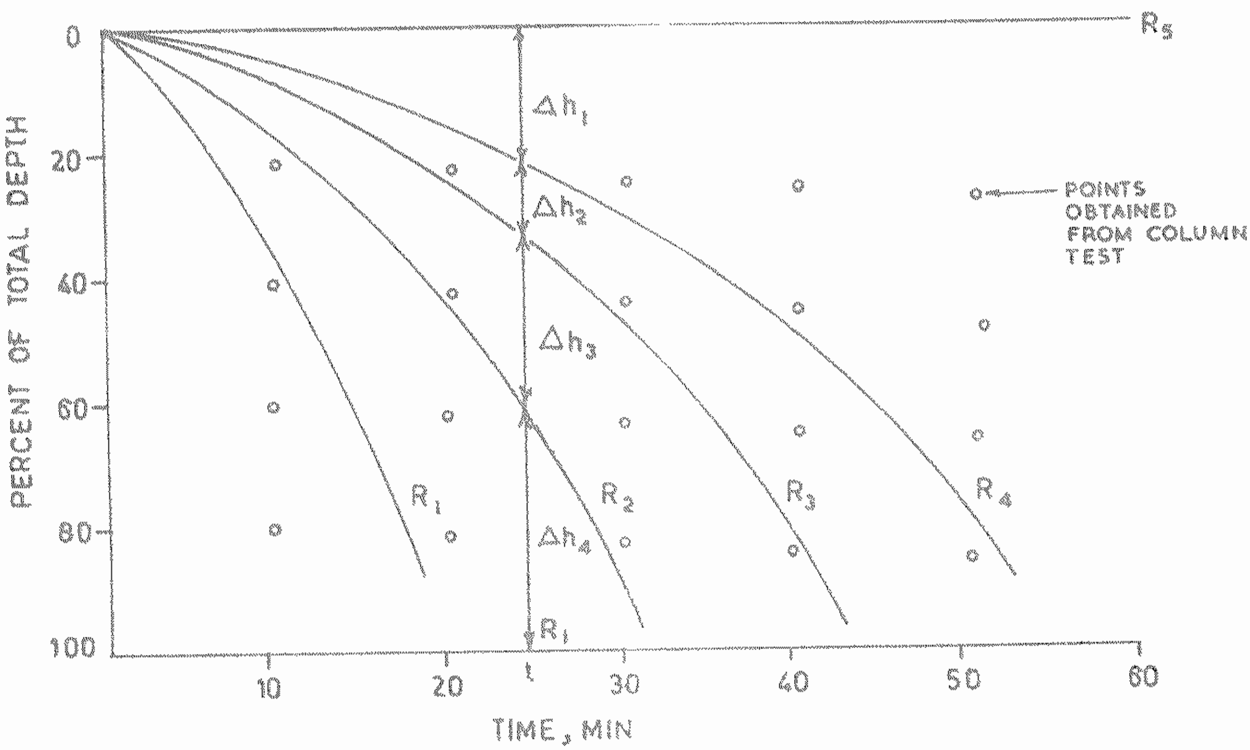


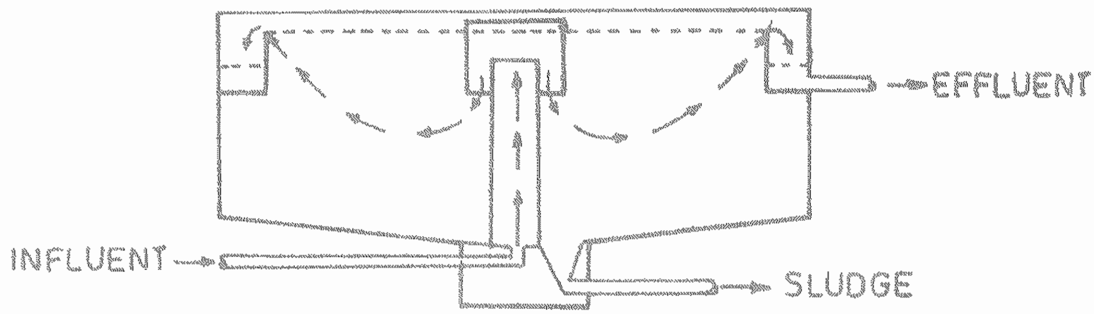
FIGURE 7.9 : SETTLING OF FLOCCULENT SUSPENSION

7.5.4.1 Horizontal Flow Tanks

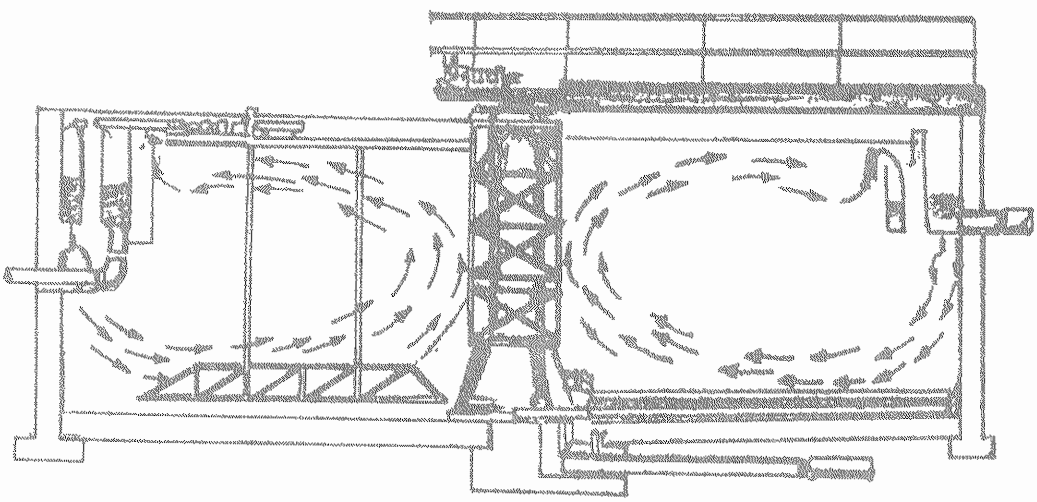
In the design of a horizontal flow tank, the aim is to achieve as nearly as possible the ideal conditions of equal velocity at all points lying on each vertical line in the settling zone. The direction of flow in the tanks is substantially horizontal. Among the representative designs of the horizontal flow settling tanks, the following may be mentioned:

a) Radial flow circular tank with central feed. The water enters at the center of the tank and emanates from multiple ports of circular well in the centre of tank to flow radially outwards in all directions equally. The aim is to achieve uniform radial flow with decreasing horizontal velocity as the water flows towards the periphery and is withdrawn from the tank through effluent structure. The sludge is plowed to central sump mechanically and continuously and is withdrawn during operation. The sludge removal mechanism consists of scraper blades mounted on two or four arms revolving slowly.

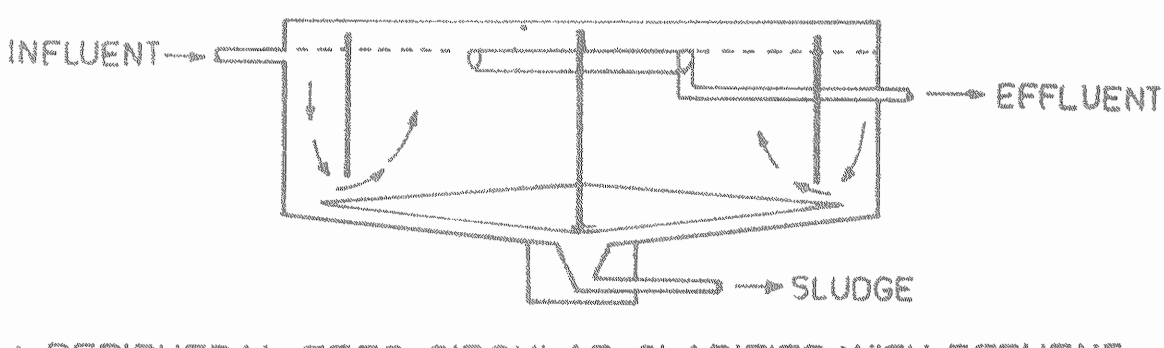
b) Radial flow circular tanks with peripheral feed. These tanks differ from the central feed circular tanks in that the water enters the tank from the periphery or the rim. It has been demonstrated that the average detention time is greater in peripheral feed basins leading to better performance.



(a) CIRCULAR CLARIFIER WITH CENTRE FEED



(b) PERIPHERAL FEED CIRCULAR CLARIFIER WITH EFFLUENT AND INFLUENT CHANNELS SEPARATED BY A SKIRT



(c) PERIPHERAL FEED CIRCULAR CLARIFIER WITH EFFUENT WEIRS NEAR THE CENTRE OF BASIN

FIGURE 7.10 : VARIOUS TYPES OF CIRCULAR CLARIFIERS

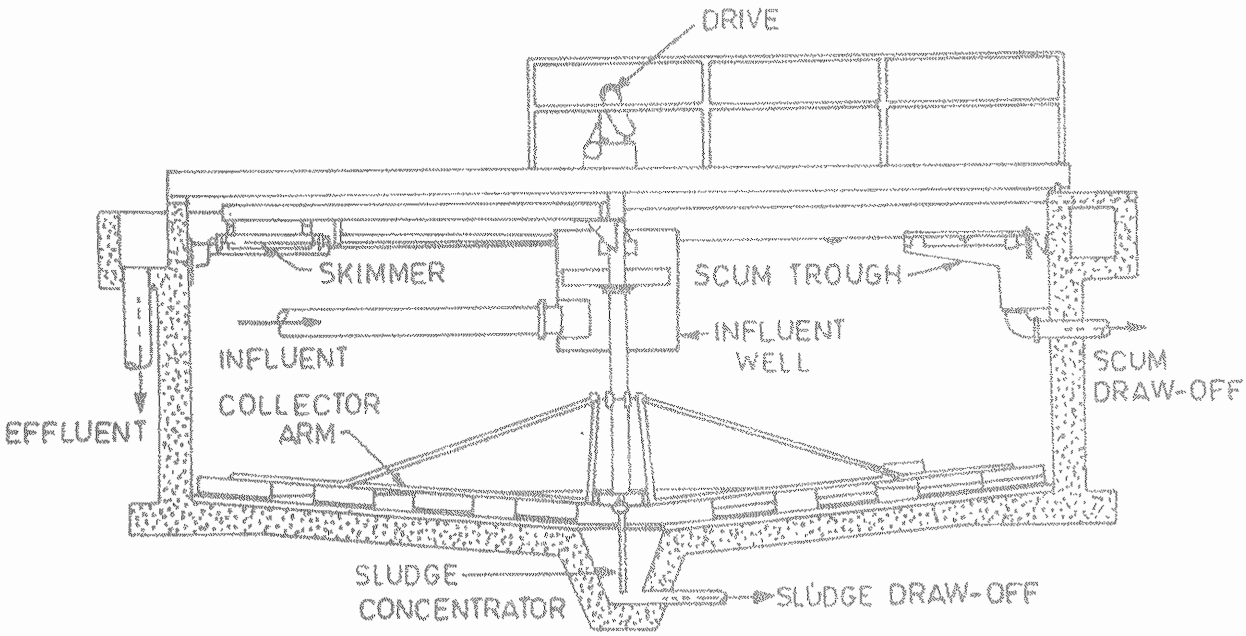


FIG. 7.11 CIRCULAR CLARIFIER DESIGN

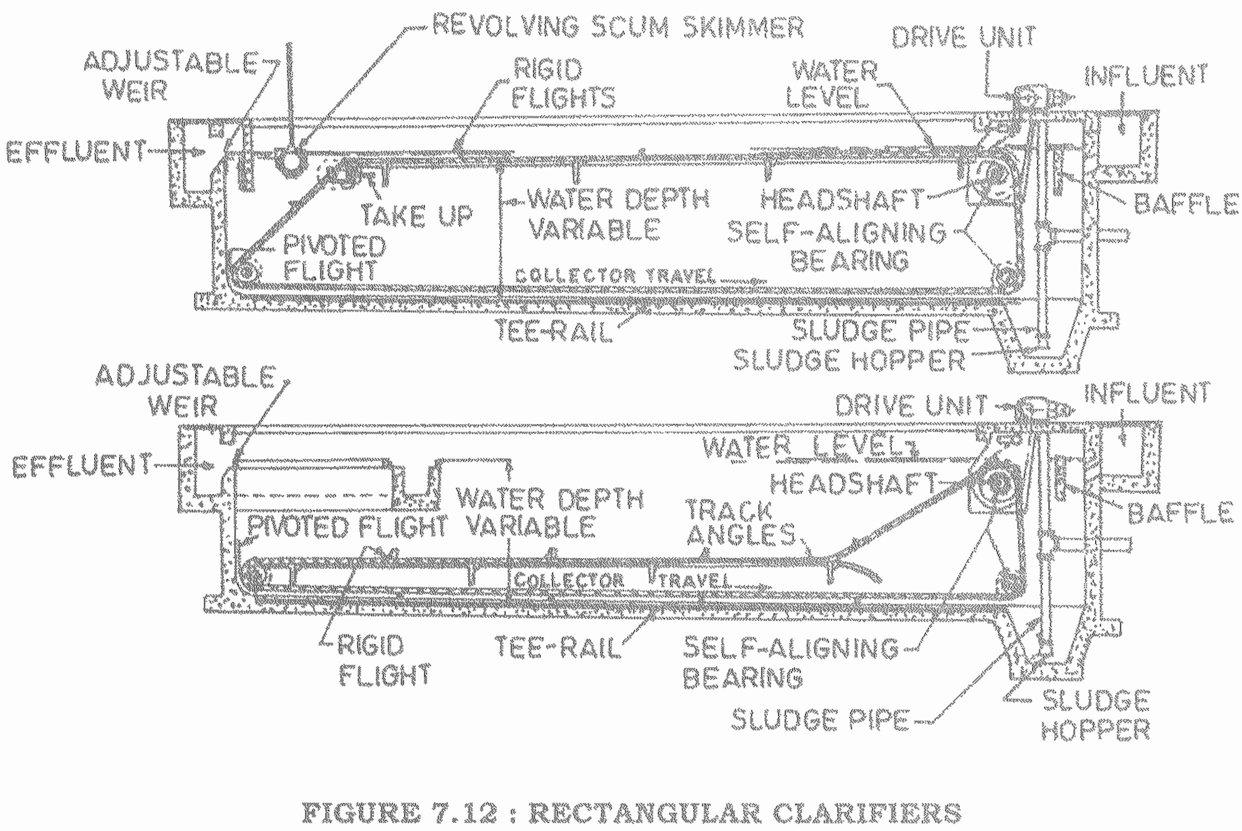


FIGURE 7.12 : RECTANGULAR CLARIFIERS

c) Rectangular tanks with longitudinal flow where the tanks are cut out of operation for cleaning. The solids are flushed to sump for removal from the dewatered tank.

d) Rectangular tanks with longitudinal flow where sludge is mechanically scraped to the sludge pit located usually towards the influent end and removed continuously or periodically without disrupting the operation of the tanks.

7.5.4.2 Vertical Flow Tanks

Vertical flow tanks normally combine sedimentation with flocculation. These tanks are square or circular in plan and may have hopper bottoms. The influent enters at the bottom of the unit where flocculation takes place as particles cojoin into aggregates. The upflow velocity decreases with increassed cross sectional area of the tank. There is a formation of blanket of floc through which the rising floc must pass. Because of this phenomenon, these tanks are also called as upflow sludge blanket clarifier. The clarified water is withdrawn through circumferential or central weir.

These tanks have no moving parts and except for a few valves, require no mechanical equipment. They are compact units requiring less land area.

7.5.4.3 Clariflocculators

Clariflocculators are widely used in the country in water and wastewater treatment. The coagulation and sedimentation processes are effectively incorporated in a single unit in the clariflocculator.

All these units consist of 2 or 4 flocculating paddles placed equidistantly. These paddles rotate on their vertical axis. The flocculating paddles may be of rotor-stator type rotating in opposite direction around this vertical axis. The clarification unit outside the flocculation compartment is served by inwardly raking rotating blades. The water mixed with chemicals is fed in the flocculation compartment fitted with paddles rotating at slow speeds.

The flocculated water passes out from the bottom of the flocculation tank to the clarifying zone through a wide opening, the area of the opening being large enough to maintain a very low velocity. Under quiescent conditions in the annular settling zone the floc embedding the suspended particles settle to the bottom and the clear effluent overflows into the peripheral launder.

7.5.5 Tank Dimensions

The settling basins may be long and narrow rectangular tanks, square or almost square tanks and circular tanks. The rectangular tanks have lengths commonly upto 30 m but larger lengths upto 100 m have also been adopted. The length to width ratio of rectangular tanks should preferably be from about 3: 1 to 5: 1. The narrower the tank, the less chance there is for setting up of cross currents and eddies due to wind action, temperature changes and other factors invloved. In very large size tanks where the depth is necessarily great, it may be advisable to provide longitudinal baffles to confine the flow to definite straight channels. These walls could be of thin sections since the pressure on both sides will be the same.

The diameter of the circular tank is governed by the structural requirement of the trusses that carry the scraping mechanism. Circular tanks upto 60 m in diameter are in use but are generally upto 30 m to reduce wind effects. Square tanks are generally smaller usually with sides upto 20 m. Square tanks with hopper bottoms having vertical flow have sides generally less than 10 m to avoid large depths.

The depth of the settling basin depends on the character of sludge handled, storage capacity required and cost. In warm climates and where the sludge is likely to contain considerable organic matter, it is not advisable to store sludge for long periods; otherwise, the decomposition of the sludge adversely affects the settling process. Depths commonly used in practice very from 2.5 to 5 m with 3.0 m being a preferred value. Bottom slopes may range from 1 % in rectangular tanks to about 8% in circular tanks. The slopes of sludge hoppers range from 1.2:1 to 2:1 (vertical: horizontal).

7.5.6 Common Surface Loadings and Detention Periods

The removal of particles of varying hydraulic subsidence values is solely a function of surface overflow rate also called "surface loading" and is independent of the depth of the basin for discrete particle and unhindered settling. However, contact opportunities among particles leading to aggregation increase with increasing depths for flocculent particles having tendency to agglomerate while settling, such as alum and iron flocs. The range of surface loadings and detention periods for average design flow for different types of sedimentation tanks are as follows:

Tank type	Surface loading $m^3/m^2/d$*		Detention period, hr*		Particles normally removed
	Range	Typical value for design	Range	Typical value for design	
Plain Sedimentation	upto 6000	15-30	0.01-15	3-4	Sand, silt and clay
Horizontal flow, Circular	25-75	30-40	2-8	2-2.5	Alum and iron floc
Vertical Flow (Upflow)Clarifiers	-	40-50	-	1-1.5	Flocculent

* at average design flow

7.5.7 Inlets and Outlets

Inlet structures must (i) uniformly distribute flow and suspended particles over the cross section at right angle to flow within individual tanks and into various tanks in parallel (ii) minimize large-scale turbulence and (iii) initiate longitudinal or radial flow, if high romoval efficiency is to be achieved. For uniform distribution of flow, the flow being divided must encounter equal head loss or the head loss between inlets on inlet openings must be small in comparison to the head available at the inlets. If hi and q_i are the head and discharge at the first inlet from the point of supply in a settling tank and h_n and q_n, being the head and discharge at the n^{th} inlet opening, farthest from the point of supply, the following relationship holds

$$h_n = k\, q_n^2 = k\,(mq_1)^2 = m^2 h_1 \tag{7.18}$$

If the discharge in n^{th}, inlet is held to mq_1 where $m < 1$, the head at the first inlet can also be expressed in terms of head lost between the first and n^{th} inlet, h_f

$h_n = h_1 - h_f = m^2 h_1$, and

$h_f = (1-m^2)h_1$

For $m = 0.99$, $h_f = 0.02\, h_1$, and $h_n = 0.98\, h_1$

Inlet or influent structures may have different arrangements as shown in Fig. 7.13. Each inlet opening must face a baffle so that most of the kinetic energy of incoming water will be destroyed and a more uniform lateral and vertical distribution of flow can occur. One of the satisfactory method of attaining uniform velocity of flow is to pass the water through a training or dispersion wall perforated by holes or slots. The velocity of flow through such slots should be about 0.2 to 0.3 m/s and head loss is estimated as 1.7 times the velocity head. The diameter of the hole should not be larger than the thickness of the diffuser wall.

Outlet or effluent structure comprises of weir, notches or orifices; effluent trough or launder and outlet pipe. V-notches attached to one or both sides of single or multiple troughs are normally preferred as they provide uniform distribution at low flows. The V-notches are generally placed 150-300 mm centre to centre. A baffle is provided in front of the weir to stop the floating matter from escaping into effluent.

Effluent troughs act as lateral spillway and can be designed on similar lines to those of wash water troughs in rapid gravity filters. The widely used equation for the design of effluent trough is

$$Y_1 = \sqrt{Y_2^2 + \frac{2\left((qL.N)^2\right)}{gb^2 Y_2}} \tag{7.19}$$

which was originally developed for flumes with level inverts and parallel sides; channel friction is neglected and the drawdown curve is assumed parabolic.

Y_1 = water depth at upstream end of launder, m

Y_2 = water depth in trough at a distance L from upstream end, m

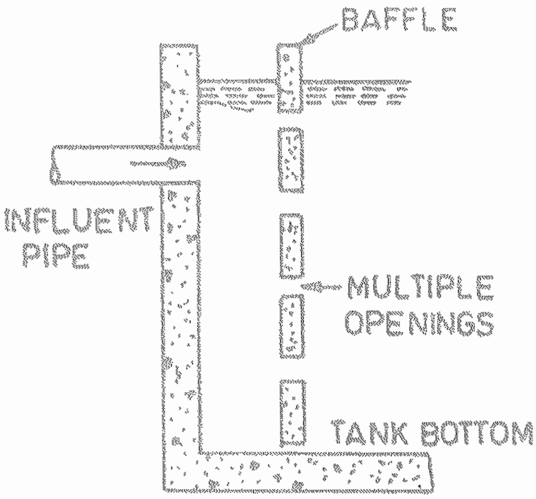
q = discharge per unit length of the weir, $m^3/s.m$

b = width of launder, m

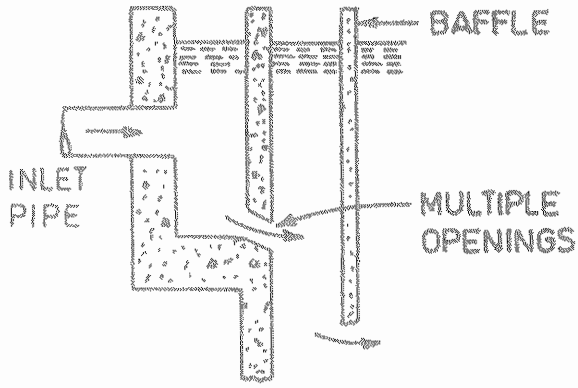
N = number of sides the weir receives the flow (one or two)

In the absence of any control device, it is reasonable and customary to assume critical flow at the lower end of the channel and hence Y_2 at lower end of channel of length L is,

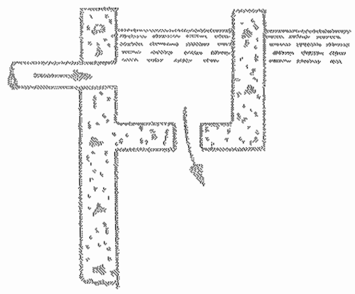
$$Y_2 = \left[\frac{(qL)^2}{b^2 g}\right]^{1/3} \tag{7.20}$$



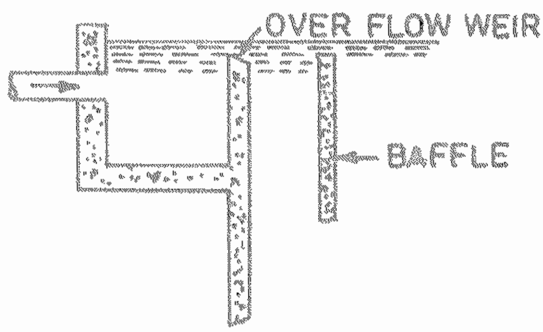
(a) DIFFUSER WALL WITH SLOTS OR PERFORATED BAFFLES



(b) INFLUENT CHANNEL WITH SUBMERGED ORIFICES IN THE INSIDE CHANNEL WALL

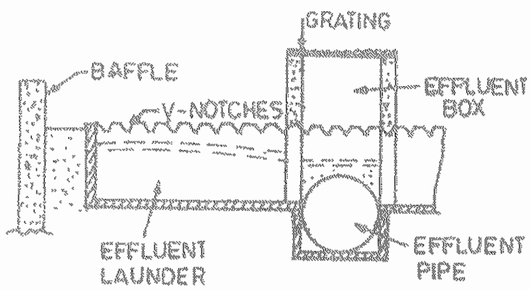


(c) INFLUENT CHANNEL WITH BOTTOM OPENINGS.

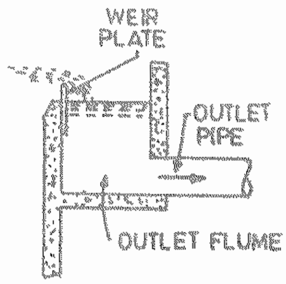


(d) OVERFLOW WEIR FOLLOWED BY A BAFFLE.

FIG. 7.13(A) FOUR TYPES OF INLETS FOR SETTLING BASINS



(a) OUTLET CONSISTING OF V-NOTCHES, EFFLUENT LAUNDER, EFFLUENT BOX AND AN EFFLUENT PIPE



(b) OUTLET WITH RECTANGULAR WEIR

FIGURE 7.13(B) : TYPICAL OUTLETS FOR SETTLING TANKS

There is a growing trend towards the use of effluent launders or troughs covering a good part of the surface of the settling basins. These are spaced at a distance of one tank depth between the troughs. The use of maximum feasible weir length in the tank from the outlet towards the inlets assists greatly in controlling density currents. Weirs, however, suffer from the difficulty in levelling which is not faced with perforated pipe launder. Perforated launders, with ports commonly submerged 30 to 600 mm below the surface are useful in varying the water level in the basin during operation and prevent floating matter passing to the filters.

7.5.8 WEIR LOADING

Weir length relative to surface area determines the strength of the outlet current. Normal weir loadings are upto 300 $m^3/d/m$. But when settling tanks are properly designed, well clarified waters can be obtained at weir loadings of even upto 1500 $m^3/d/m$.

7.5.9 SLUDGE REMOVAL

Sludge is normally removed under hydrostatic pressure through pipes. The size of the pipe will depend upon the flow and the quantity of suspended matter. It is advisable to provide telescopic sludge discharge arrangement for easy operation and for minimising the wastage of water. For non-mechanised units, pipe diameters of 200 mm or more are recommended. Pipe diameters of 100 to 200 mm are preferred for mechanised units with continuous removal of sludge with hydrostatic head. In circular tanks, where mechanical scrapers are provided, the floor slopes should not be flatter than 1 in 12, to ensure continuous and proper collection of sludge. For manual cleaning, the slope should be about 1 in 10.

The power required for driving the scraping mechanism in a circular tank depends upon the area to be scraped and the design of the scraper. The scraping mechanism is rotated slowly to complete one revolution in about 30 to 40 minutes or preferably the tip velocity of the scraper should be around 0.3 m/min or below. Power requirements are about 0.75 w/m^2, of tank area.

Sludge and wash water should be properly disposed of without causing any problems of pollution if discharged into water courses.

For sludge blanket type vertical flow settling tanks, the slopes of the hoppers should not be less than 55° to horizontal to ensure smooth sliding and removal of sludge. In such tanks special slurry weirs are provided with their crests in level with the top of sludge blanket for continuous bleeding of the excess sludge.

Special types of consolidation tanks with a capacity of 30 min are sometimes provided to consolidate the sludge and recover water from it.

In non-mechanised horizontal flow rectangular settling tanks, the basin floors should slope about 10% from the sides towards the longitudinal central line adopting a longitudinal slope of at least 5% from the shallow outlet end towards the deeper inlet area where the drain is normally located. Manual cleaning of basins is normally done hydraulically, using high pressure hoses. Admitting settled water through the basin outlet helps this function. If sludge is to be withdrawn continuously or nearly continuously from the bottom of the basin by gravity without mechanical equipment, hopper bottoms have to be used with slope of not less than 55° to the horizontal.

Reclamation of water from the sludge removed from the settling basin should be encouraged. The various methods include disposal of sludge on land or on sludge drying beds.

7.5.10 Settling Tank Efficiency

The efficiency of basins is reduced by currents induced by inertia of the incoming water, wind, turbulent flow, density and temperature gradients. Such currents short circuit the flow. The efficiency of real basin affected by current induced short circuiting may be mathematically expressed as

$$\frac{Y}{Y_0} = 1 - \left[1 + n\frac{V_0}{Q/A}\right]^{-\frac{1}{n}} \tag{7.21}$$

where,

$\frac{Y}{Y_0}$ = Efficiency of removal of suspended particles

n = Coefficient that identifies basin performance

V_0 = Surface over flow rate for ideal settling basin

Q/A = Required surface overflow rate for real basin to achieve an efficiency of Y/Y_o for given basin performance.

The values of n are assumed 0 for best possible performance, 1/8 for very good performance, 1/4 for good performance, 1/2 for average performance 1 for very poor performance. Mathematical analysis of longitudinal mixing in settling tanks indicates that the value of n can be approximated by the ratio of the differences between the mean and modal flow-through periods to the mean flow-through period.

The short-circuiting characteristics of tanks are usually measured by addition of a slug of dye, electrolyte or tracer and observing the emergence of this tracer substance with passage of time. A frequency distribution plot of the concentration with respect to time is plotted. Modal, median and mean flow-through periods identify the central tendency of the time-concentration distribution and percentiles reflect its variance. The ratio of the median time to the mean time or the ratio of the difference between the mean and the modal (or mean and median) to the mean indicate the stability or efficiency of the basin. The lower the first value is from unity or the higher the second value, the lesser the efficiency and the more the shortcircuiting. A well designed tank should be capable of having a volumetric efficiency of at least 70%.

To achieve better clarification, the flow regime in settling basin should be as close as possible to ideal plug flow. A narrow and long rectangular tank approximates plug flow conditions better than wide shallow rectangular tank, peripheral feed circular tank and centre feed radial flow tank.

Settling tanks should be capable of giving settled water having turbidity not exceeding 20 and preferably less than 10 NTU.

7.5.11 PRESEDIMENTATION AND STORAGE

The turbidity of raw water from rivers and streams may exhibit wide flucutations and values exceeding a few thousand NTU are not uncommon during high flow season. The sediment load of the river during floods chiefly derives from soil erosion and consists predominantly of coarse suspended solids Removal of large- sized and rapidly settleable silt and other materials can be accomplished by presedimentation and storage before the raw water reaches the treatment plant. Presedimentation and storage have been used for both highly turbid waters and waters of relatively low turbidity.

When removal of coarse and rapidly settling silt is aimed at in presedimentation, lower detention periods of 0.5 to 3 hours and higher surface loading of 20 to 80 $m^3/m^2/d$ have been recommended. These plain sedimentation tanks can be constructed with wooden sheet piles or dug out of the earth with sloping sides besides being or made of conventional materials like masonry or concrete. The storage basins or reservoirs, unlike presettling basins, are designed for very large detention periods ranging from about one week to a few months. While storage is best considered for waters of extremely high turbidity, big storage basins have also been constructed for waters of low initial turbidity.

7.5.12 TUBE SETTLERS

Settling efficiency of a basin is primarily dependent upon surface area and is independent of depth. Attempts have been made to use this concept to achieve better efficiency and economy in space as well as cost. Wide shallow trays inserted within conventional basins with a view to increase the surface area have not met with success. However, very small diameter tubes having a large wetted perimeter relative to wetted area providing laminar flow conditions and low surface loading rate have shown good promise. Such tube settling devices provide excellent clarification with detention times of equal to or less than 10 minutes. Tube configurations can be horizontal or steeply inclined. In inclined tubes (about 60°) continuous gravity drainage of the settleable material can be achieved. At angles greater than 40°, the units lose efficiency rapidly whereas with angles less than 60°, sludge will not slide down the floors. Under such situations, hosing down the sediments may have to be resorted to. With horizontal tubes (normally inclined at 5°) auxiliary scouring of settled solids is necessary. While tube-settlers have been used for improving the performance of existing basins, they have also been successfully used in a number of installations as a sole settling unit. It has been found that if one-fifth of the outlet end of a basin is covered with tube or plate settlers, the effective surface loading on the tank is nearly halved or the flow through the basin can be nearly doubled without impairement of effluent quality.

The tubes may be square, circular, hexagonal, diamond shaped, triangular, rectangular or chevron shaped. A widely used material for their construction is thin plastic sheet (1.5 mm) black in colour, though plastic and asbestos cement pipes have also been used. There are number of proprietory devices such as Lemella clarifier.

7.5.12.1 Analysis Of Tube Settlers

The performance of the tube settlers is normally evaluated by a parameter, S, defined as

$$S = \frac{V_s}{V_0}(\sin\theta + L\cos\theta) \qquad (7.22)$$

Where,

V_s = Settling velocity of the particle in a vertically downward direction (L/T)

V_o = Velocity of flow along the tube settler

θ = Angle of inclination of tube settlers, degrees.

L = Relative length of settler = l/d, dimensionless

l,d = Length and diameter (width) of the tube settlers, (L)

If the value S equals or exceeds a critical value, S_c for any particle, it is completely removed in the tube settlers under ideal conditions. For laminar flow regime in tube settlers, the value of S_c have been determined as 4/3, 11/8 and 1 for circular, square and parallel plates type of tube settlers assuming uniform flow.

It is found that the performance of tube settlers is improved significantly with L values of upto 20 and insignificantly beyond 20. Therefore, it is desirable to design tubesettlers with L values around 20 but less than 40. Increasing the angle of inclination of tube settlers beyond 40°, results in deterioration in their performance. Essentially horizontal tube settlers perform better than steeply inclined tube settlers. It is opined that from relative economics point of view, the order of preference for tube settlers is parallel plates followed by circular tubes and square conduits.

It is recommended to increase the dimensionless length L of tube settlers by an additional amount L' to account for transition zone near inlet to change to fully developed laminar flow.

$$L' = 0.058\,\frac{v_0.d}{\nu} \qquad (7.23)$$

Where ν is the kinematic viscocity of water.

7.6 FILTRATION

7.6.1. General

Filtration is a process for separating suspended and colloidal impurities from water by passage through a porous medium or porous media. Filtration, with or without pretreatment, has been employed for treatment of water to effectively remove turbidity (e.g., silt and clay), colour, microorganisms, precipitated hardness from chemically softened waters and precipitated iron and manganese from aerated waters. Removal of turbidity is essential not only from the requirement of aesthetic acceptability but also for efficient disinfection which is difficult in the presence of suspended and colloidal impurities that serve as hideouts for the microorganisms.

Filters can be classified according to (1) the direction of flow (2) types of filter media and beds (3) the driving force (4) the method of flow rate control and (5) the filtration rate. Depending upon the direction of flow through filters, these are designated as down flow, upflow, biflow, radial flow and horizontal flow filters. Based on filter media and beds, filters have been categorized into (a) granular medium filters and (b) fabric and mat filters and micro-strainers. The granular medium filters include single-medium, dual-media and multi media (usually tri-media) filters. Sand, coal, crushed coconut shell, diatomaceous earth and powdered or granular activated carbon have been used as filter media but sand filters have been most widely used as sand is widely available, cheap and effective in removing impurities. The deriving force to overcome the fractional resistance encountered by the flowing water can be either the force of gravity or applied pressure force. The filters are accordingly referred to as gravity filters and pressure filters. In the fourth category are constant rate and declining or variable rate filters. Lastly dependent upon the flow rates, the filters are classified as slow or rapid sand filters.

Filtration of municipal water supplies normally is accomplished using

(a) slow sand filters, or

(b) rapid sand filters

Both of these types of filters are downflow, granular-medium(Single-medium) gravity filters. The rapid sand filters have been conventionally operated at constant rate of filtration.

7.6.2 SLOW SAND FILTERS

7.6.2.1 General

Slow sand filters can provide a single step treatment for polluted surface waters of low turbidity (< 20 NTU) when land, labour and filter sand are readily available at low cost, chemicals and equipments are difficult to procure and skilled personnel to operate and maintain are not available locally.

When raw water turbidity is high, simple pre-treatment such as storage, sedimentation or primary filtration will be necessary to reduce it to within desirable limits. Chemical coagulation and flocculation have also been successfully tried to effectively pretreat turbid waters without adverse effect on filtrate quality by slow sand filtration.

7.6.2.2 Description

A slow sand filter consists of an open box about 3.0 m deep rectangular or circular in shape and made of concrete or masonry (Fig. 7.14). The box contains a supernatant water layer, a bed of filter medium, an underdrainage system and a set of control valves and appurtenances.

The supernatant provides the driving force for the water to flow through the sand bed and to overcome frictional resistance in other parts of the system. It can also provide a storage of several hours to the incoming water before it reaches the sand surface.

The filter bed consists of natural sand with an effective size (E.S.) of 0.25 mm to 0.35 mm and uniformity coefficient (U.C) of 3 to 5. For best efficiency, the thickness of filter bed should be not less than 0.4 - 0.5 m. As a layer of 10-20 mm sand will be removed every time the filter is cleaned, a new filter should be provided with an initial sand depth of about 1.0 m. Resanding will then become necessary only once in 2-3 years.

A — RAW WATER INLET VALVE
B — SUPERNATANT DRAINOUT VALVE
C — RECHARGE VALVE
D — FILTER SCOUR VALVE
E — FILTERED WATER OUTLET VALVE
F — FILTER TO WASTE VALVE
G — FILTERED WATER VALVE

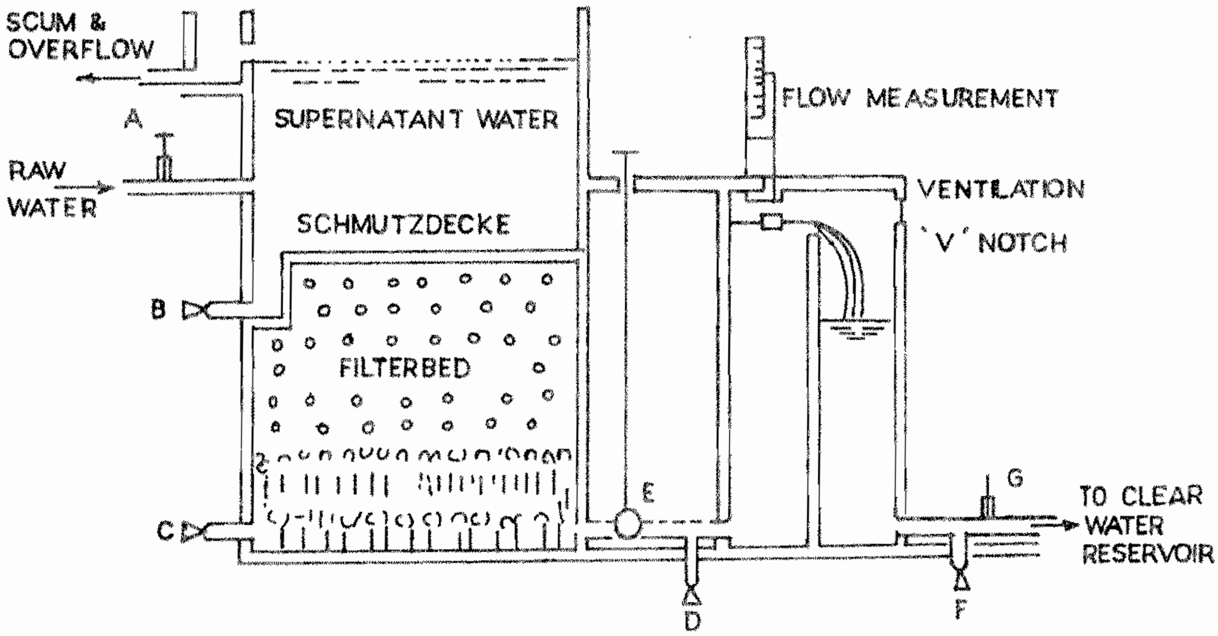


FIGURE 7.14 : BASIC ELEMENTS OF SLOW SAND FILTER (SCHEMATIC)

The underdrainage system supports the sand bed and provides unobstructed passage for filtered water to leave the underside of the filter. The underdrains may be made of unjointed bricks laid to form channels, perforated pipes or porous tiles laid over drains. Graded gravel to a depth of 0.2 -0.3 m is placed on the underdrains to prevent the sand from entering the underdrains and ensure uniform abstraction of filtered water from the entire filter bed.

A system of control valves facilitates regulation of filter rate and adjustment of water level in the filter at the time of cleaning and backfilling when the filter is put back into operation after cleaning.

7.6.2.3 Purification in a Slow Sand Filter

In a slow sand filter, water is subject to various purifying influences as it percolates through the sand bed. Impurities are removed by a combination of straining, sedimentation, bio-chemical and biological processes. Shortly after the start of filtration, a thin slimy layer called the 'schmutzdecke' is formed on the surface of sand bed. It consists of a great variety of biological organisms which feed on the organic matter and convert it into simple,

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harmless substances. Considerable portion of inert suspended particles is mechanically strained out in this layer. During its passage through 0.4 - 0.6 m of sand bed, the water becomes virtually free from suspended solids, colloids, pathogens and complex salts in solution. The result is a simultaneous improvement in the physical, chemical and bacteriological quality of water. Starting with an average quality of raw water, a properly designed and operated filter can produce a filtrate satisfying normal drinking water standards. Nevertheless, the filtrate should be disinfected to render it safe.

7.6.2.4 Design Considerations

(a) Design Period

In slow sand filter construction, there is no economic advantage in building large plants to serve long years into future. Therefore, the design period should be short, say 10 years. This will help to optimise the long-term investment in water supply and will allow the available money to finance more new projects immediately.

(b) Plant Capacity

The per capita water supply multiplied by the projected future population gives the design demand. It would be convenient to convert the daily required volume to a design flow 'Q', the quantity of water to be treated per hour rather than per day, This is because the daily requirement of water may be treated over a period of 24 hours just in a few hours as is done in small plants. Thus for a given daily output, the size of the plant depends on duration of filter operation.

(c) Filtration rate and number of filters

To provide for changes in raw water quality and uncertainties in operation and maintainance, it is desirable to design filters for a normal filtration rate of 0.1 m/hr. A minimum of two filter units should be provided. This will restrict the overload rate to 0.2 m/hr when one unit is taken out for cleaning and would ensure uninterrupted production. There is no need to provide for any standby unit. The number of filter units for a given area can be increased to gain higher flexibility and reliability. For a given area, the optimum number and size of filters which will be only 10 percent more expensive than the minimum 2 bed unit are given in Table 7.2

TABLE 7.2

RECOMMENDED NUMBERS OF SLOW SAND FILTERS FOR GIVEN PLAN AREAS

Area in m^2	Number of beds
Upto 20	2
20- 249	3
250-649	4
650- 1200	5
1201 - 2000	6

(d) Filter shape and layout

Rectangular filters offer the advantage of common wall construction and may be preferred except for very small installations where circular shape may be economical. Arranging filters in a row maximizes the number of common walls and facilitates construction, operation and maintenance. Filters can also be arranged symmetrically on either side of a central pipe gallery. The layout will be determined by local topography and the placement of pump houses, storage and other facilities.

(e) Depth of Filter Box

The elements that determine the depth of the filter box and their suggested depths are free board (0.2 m), supernatant water reservoir (1.0 m), filter sand (1.0 m), supporting gravel (0.3 m), and under-drainage system (0.2 m) with a total depth of 2.70 m. The use of proper depths for these elements can reduce the cost of the filter box considerably without adversely affecting efficiency.

(f) Filter Sand and Gravel

Undue care in the selection and grading of sand for slow sand filters is neither desirable nor necessary. Use of builder grade or locally available sand can keep the cost low. Similarly, rounded gravel, which is often quite expensive and difficult to obtain, can be replaced by hard, broken stones to reduce cost.

7.6.2.5 Construction Aspects

(a) General

The construction of slow sand filters should be based on sound engineering principles. Some of the important considerations that need attention are: (i) the type of soil and its bearing capacity; (ii) the ground water table and its fluctuation and (iii) the availability and cost of construction materials and labour. Water tight construction of the filter box should be guaranteed, especially when the ground water table is high. This will prevent loss of water through leakage and contamination of filtered water. The top of the filter should be atleast 0.5 m above the ground level in order to keep away dust, animals and children. The danger of short circuiting of raw water may be prevented by rough finishing of the inside of the walls upto maximum sand level. The drainage system should be carefully laid as it can not be inspected, cleaned or repaired without the complete removal of the filter bed material.

(b) Inlet

The inlet structure is an important component of a slow sand filter and should be so designed and constructed as to cause minimum disturbance to the filter bed, while admitting raw water and to facilitate routine operation and maintenance. A filter needs to be cleaned periodically and this is done by lowering the water level a few centimeters below the sand bed and scraping the top layer of 10-20 mm of sand. It is found in practice that draining the water through the filter bottom takes several hours, at times 1-2 days. In order to obviate this difficulty, a supernatant drain out chamber with its top just above the sand level, has to be provided. By a proper design, the filter inlet and the supernatant drain out could be suitably combined in a single chamber (Fig. 7.15).

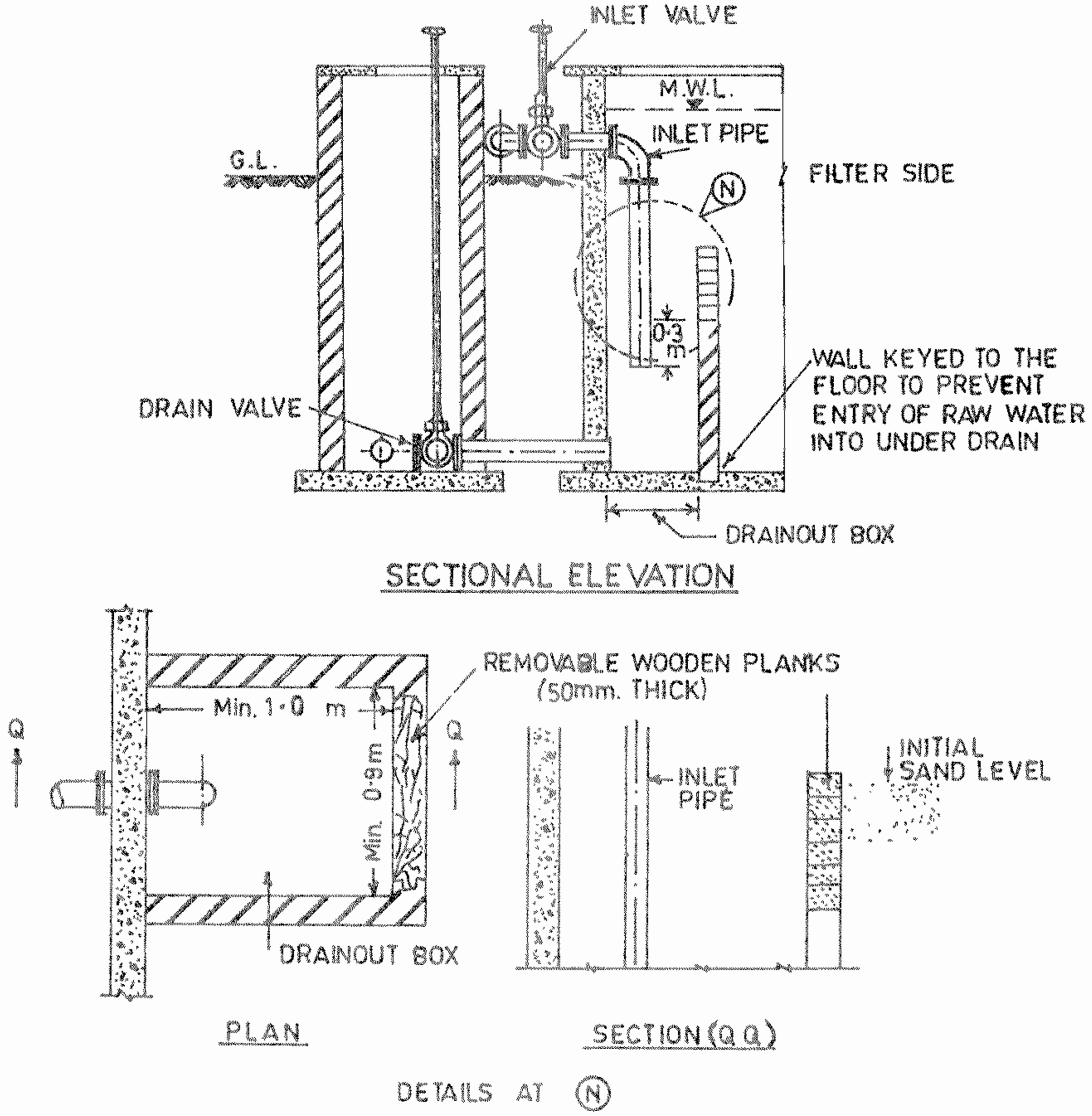


FIGURE 7.15 : INLET CUM SUPERNATANT DRAINOUT BOX

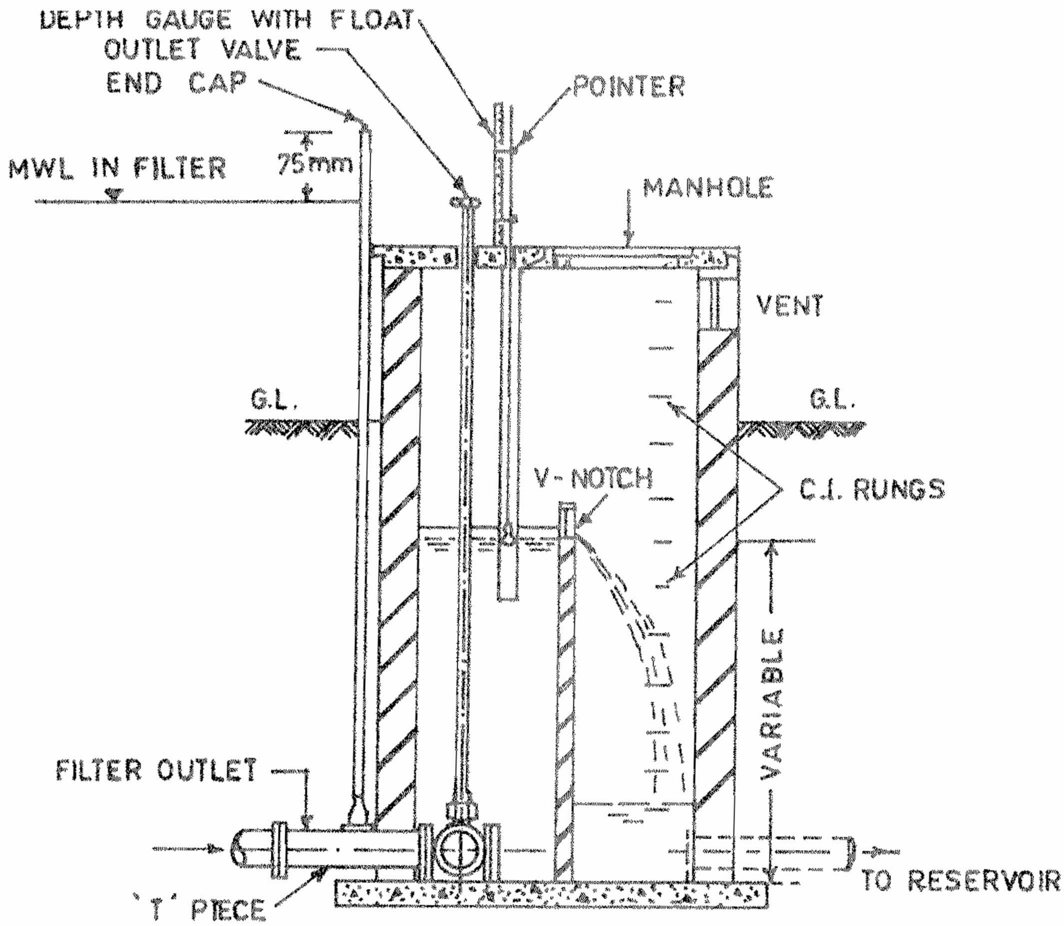


FIGURE 7.16 : OUTLET CHAMBER

(c) Outlet

The outlet structure incorporates means for measuring the filter flow and backfilling with clean water after sand scraping and recommissioning of the filter.

In small filters, the outlet chamber is usually constructed in two parts separated by a wall with a weir. The sill of the weir is fixed above the highest sand level in the filter bed. This makes filter operation independent of fluctuations in the clear water storage level and prevents occurrence of negative head in the filter. It also aerates the filtered water thus raising its oxygen content. To facilitate aeration, a ventilation opening properly screened is provided in the chamber (Fig. 7.16).

(d) Scum and overflow outlet

To facilitate drainage of surplus water entering the filter and scum that may accumulate on the supernatant water, an overflow pipe/ weir should be provided in the filter.

7.6.2.6 Operation And Maintenance

(a) Initial Commissioning

While commissioning, a newly constructed filter is charged with water from bottom through the underdrain until it rises 0.1-0.15 m above the sand bed. This ensures expulsion of entrapped air in the filter bed and the underdrainage system. The inlet valve is then gradually opened and water is admitted to the filter from top. The water is allowed to filter at approximately the normal filtration rate and the effluent is run to waste till the formation and 'ripening' of the filter skin is complete. The ripening period is complete when the bacteriological analysis indicates that the effluent quality is good and can be put into the distribution system.

(b) Flow Control

The rate of filtration can be controlled either at the inlet or at the outlet of the filters. Both methods have advantages.

In an inlet-controlled filter, the rate of filtration is set by the inlet valve. Once the desired rate is reached, no further manipulation of the valve is required. At first the water level over the filter will be low but gradually it will rise to compensate for the increasing resistance of the filter skin. Once the level has reached the overflow outlet, the filter has to be taken out for cleaning.

Inlet control reduces the amount of work which has to be done on the filter to just clean it. The rate of filtration will always be constant with this method and the build-up of resistance in the filter skin is directly visible. On the other hand, the water is not retained for very long at the beginning of the filter run, which may reduce the efficiency of treatment.

In an outlet-controlled filter, which is more common, the rate of filtration is set with the outlet valve. Daily or every two days this valve has to be opened a bit to compensate for the increase in resistance in the filter skin. The disadvantage of this method is that the outlet valve also has to be manipulated on a regular basis, causing a slight variation in the rate of filtration. Thus, the operator is forced to visit the plant atleast every day, otherwise the output will fall. The water is retained for five to ten times as long as in the inlet-controlled filter at the beginning of the filter run, which may make purification more efficient. Removal of scum will also be much simpler than with inlet-controlled filtration.

In many situations electricity and diesel fuel are not available all the time, so existing slow sand filter plants sometimes function only for part of the day. Research has shown that this leads to a serious deterioration in the quality of the out-flowing water and must be avoided. If electricity for working pumps is likely to be intermittent, either a raw-water storage reservoir which can feed water to the filters under gravity supply can be built, or 'declining rate filtration' should be used. That is, when the raw water is stopped, all valves remain in the same position and filtration continues at a declining rate as the water level in the plant falls. When the raw water supply is resorted, the water standing over the sand bed will rise to its earlier level. Where declining rate filtration is used, a larger filter is needed.

(c) Filter Cleaning

When the filter has attained the maximum permissible headloss, it is taken out of service for cleaning. The inlet is closed and the supernatant is drained out or allowed to filter though so as to expose the sand bed. Experience has shown that filtering through takes a long time, occasionally even one or two days. Hence, lowering the water level by opening the supernatant drain out valve should be preferred. When the supernatant is drained out, the water level is lowered 10- 15 cm below the top of the sand bed by opening the scour valve. Without allowing the bed to dry up, the filter is cleaned manually by removing the top layer of 2-3 cm of sand along with the filter skin. The filter is returned to service by admitting through bottom filtered water from the adjacent filter to a level of a few centimeters above the sand bed before allowing raw water from top. The removed sand is washed, dried and stored for future use.

(d) Resanding

Due to periodic cleaning, when the sand depth is reduced to a minimum of 0.4 m, it is necessary to make up the sand depth to the original level. This is done by replenishing with a fresh lot of sand, taking care to see that the remaining old sand is placed on top of the new sand. This avoids accumulation of dirt in the deeper layers of filter bed and helps in quick ripening after resanding.

7.6.2.7 Cost Aspects

(a) Minimum filter cost

The cost of a filter excluding pipes and valves is made up of two components: the total cost for floor, underdrains, sand and gravel; and the cost of walls of the filter box.

This cost in general is

$$C = K_A A + K_p P$$

Where, A is the total filter bed area in m^2 ; P the total wall length in m, K_A, the cost per unit area of filter bed, and K_p, the cost per unit length of wall. For rectangular filters with common walls, the problem is to minimize C subject to:

$$A = nlb \text{ and } P = 2nb + l(n+l) \qquad (7.25)$$

Where 'n' is the number of filters, b is breadth, and 'l' is the filter length.

The term $K_A A$ is constant for any value of 'n' and any filter shape. Hence, the minimum cost solution is the solution that minimises P, which is

$$l^2 = \frac{2A}{n+1} \qquad (7.26)$$

and

$$b = \frac{(n+1)l}{2n} \qquad (7.27)$$

The equation for b, when re-arranged, shows that 2 nb = (n + 1)l, or the condition for minimum filter cost is to have the sum of the length equal to the sum of the breadths. It can be shown that this is true whether filter units are arranged in a single row or as blocks on each side of a central gallery. The general expression for the minimum cost is found by substituting Eqn. 7.26 and 7.27 for Eqn. 7.24.

$$C = K_A A + 2K_P\left(\sqrt{2A(n+1)}\right) \tag{7.28}$$

The values of K_A and K_p, can be worked out for any place based on prevailing prices for construction materials. For Nagpur, India (1983),

$$C = 500A + 1660\left(\sqrt{2A(n+1)}\right) \tag{7.29}$$

(b) Economy of Scale

A general cost model for the filter beds be written as:

$$C = K(A)^a \tag{7.30}$$

Where 'A' is the total area of the filter beds, K is the cost per unit area of filter bed construction including walls, and 'a' is the exponent that represents the economy of scale factor.

The cost data obtained from Eq. 7.29 for various values of A can be used to determine the parameters K and 'a' of the function by the method of least squares. The resulting equation for Nagpur (1983) is given by:

$$C = 1617\ A^{0.869} \tag{7.31}$$

Large economics of scale are associated with small values of the exponent. Until the exponent decreases to about 0.6 or 0.7, there is no economic incentive to overdesign. Thus, very little saving is accomplished by increasing the size of the project in order to provide service over a long time into the future.

(c) Cost of Slow Versus Rapid Sand Filters

There is a general misconception that slow sand filters, because of their relatively larger area, are expensive. However, this is not always true. Comparative cost analysis for slow and rapid filters has shown that that slow sand filters are cost effective, especially for rural and small community water supplies. The economic capacities have to be determined for specific situations using local cost data before deciding on the choice between the two types of filters.

TABLE 7.3

SUMMARY GUIDELINES FOR DESIGN OF SLOW SAND FILTERS

Description	Recommended Design value	Description	Recommended Design value
Design period	10 years	Depth of Supernatant water	1.0m
Filteration rate		Free Board	0.2m
Normal Operation	0.1m/hr	Depth of filter sand Initial	1.0m
Max. overload rate	0.2m/hr	Final (Minimum)	0.4m
Number of filter beds Minimum	2	Size of sand Effective size	0.2-0.3mm
Areas upto 20 m^2	2	Uniformity coefficient (U.C)	5
Areas upto 20-249 m^2	3	Gravel (3-4 layers) depth	0.3m
Areas upto 250-649 m^2	4	Underdrains (made of bricks or perforated pipes)	0.2m
Areas upto 650-1200 m^2	5	Depth of filter box	2.7m
Areas upto 1201-2000 m^2	6	Effluent weir level above sand bed	20-30mm

7.6.3 Rapid Sand Filters

7.6.3.1 Filtration Process

The rapid sand filter comprises of a bed of sand serving as a single medium granular matrix supported on gravel overlying an underdrainage system. The distinctive features of rapid sand filtration as compared to slow sand filtration include careful pretreatment of raw water to effectively flocculate the colloidal particles, use of higher filtration rates and coarser but more uniform filter media to utilise greater depths of filter media to trap influent solids without excessive head loss and backwashing of filter bed by reversing the flow direction to clean the entire depth of filter. Pretreatment of filter influents should be adequate to achieve efficient removal of colloidal and suspended solids despite fluctuations in raw water quality. A typical sketch for granular medium gravity flow filter is shown in figure 7.17.

When water containing suspended matter is applied to the top of filter bed, suspended and colloidal solids are left behind in the granular medium matrix. Accumulation of suspended particles in the pores and on the surface of filter medium leads to build up of head loss as pore volume is reduced and greater resistance is offered to the flow of water simultaneously with the build up of head loss to a predetermined terminal value, the suspended solids removal efficiency of successive layers of filter medium is reduced as solids accumulate in the pore space and reach an ultimate value of solids concentration as defined by operating conditions. This results eventually in break through of suspended solids and the filtrate quality deteriorates. Ideally, a filter run should be terminated when the head loss reaches a predetermined value simultaneously with the suspended solids in filtrate attaining the preselected level of acceptable quality.

7.6.3.2 Principal Mechanisms of Partiicle Removal

The removal of particles within a deep granular-medium filter, such as rapid sand filter, occurs primarily within the filter bed and is referred to as depth filtration. Several mechanisms either singly or in combination, act to achieve overall removal of suspended and colloidal matter in depth filtration. Conceptually the removal of particles takes place in two distinct steps, a transport and an attachment step. In the first step, the impurity particle must be brought from the bulk of the liquid within the pores close to the surfaces of the medium or the previously deposited solids on the medium. Once the particles come closer to the surface, an attachment step is required to retain it on the surface instead of letting it flow down the filter.

The transport step may be accomplished by straining, gravity settling, impaction, interception, hydrodynamics and diffusion and it may be aided by flocculation in the interstices of the filter. The particle transport is a physical process principally affected by those parameters which govern mass transfer. These physical variables include size of filter medium, dm; filtration rate, v; density ρ_s and size of the susupended particles, d_p; and water temperature.

The particle attachment step is a physicochemical process involving electrostatic interactions, Van der Waal's forces of molecular attraction, chemical bridging or specific adsorption. Attachment is affected by chemical characteristics of the water and filter medium. Pretreatment of filter influents by coagulants and pH of water affect the efficiency of attachment step and consequently of solid removal in a filter. The need, therefore, of adequate pretreatment before filtration to achieve efficient removal of suspended solids is evident.

Dimensionless parameters have been defined for various transport and attachment mechanisms and mathematical equations are proposed to predict the removal efficiency of particles based on physical variables such as dp, dm, ρ_s, v, ρ and μ the liquid density and absolute viscosity, porosity of filter medium and concentration of suspended solids and chemical characteristics of water and filtering medium. An analysis of these analytical expressions indicates that filter efficiency may improve by decreasing the size of filter media, reducing the rate of filtration and at higher temperatures. The suspended particles falling into the categories of significantly more than and less than 1μm dia are efficiently removed. A particle with a size of 1 μm has the lowest efficiency of removal. Further, ample attachment opportunities exist in conventional deep granular filters. If such filters do not remove solids efficiently, pretreatment should be changed to improve attachment of suspended particles to filter media grains.

7.6.3.3 Rate Of Filtration

The standard rate of filtration through a rapid sand filter is usually 80 to 100 lpm/m^2 (4.8-6m/hr). Practice is tending towards higher rates (upto 10 m/hr) in conjuction with greater care in conditioning the water before filtration and with the use of coarser sand (effective size upto 1 mm). A prudent arrangement would be to design the filters on the basis of average consumption at a normal rate of 4.8 m/hr but with the inlet and the outlet control arrangements designed to permit a 100% overload for emergent occasions.

7.6.3.4 Capacity Of A Filter Unit

The capacity of the rapid sand filters should be such that the number of units can take care of the total quantity of water to be filtered and is optimum to keep the filters working without undue overloading at any time. The smaller the number of units, the fewer the appurtenances but the larger the wash-water equipment that will be required. Thus while designing large size filters one must consider the rate at which wash-water must be supplied and the hydraulic problems for securing uniform distribution of wash-water due to the large area. A maximum area of 100 m^2 for a single unit is recommended for plants of greater than 100 mld consisting of two halves each of 50 m^2 area. Also for flexibility of operation a minimum of four units should be provided which could be reduced to two for smaller plants.

7.6.3.5 Dimensions Of Filter Unit

Layout of the plant, economy and convenience determine the relationship between the length and the breadth of the units. Where filters are located on both sides of a pipe gallery, the ratio of length to width of a filter-box has been found to lie, in a number of installations, between 1.11 and 1.66 averaging about 1.25 to 1.33. A minimum overall depth of 2.6m including a free board of 0.5 m is adopted.

The filter shell may be in masonry or concrete to ensure a water tight structure. Except in locations where seasonal extremes of temperature are prevalent, it is not necessary to provide a roofing over the filters, the operating gallery alone being roofed over.

7.6.3.6 Filter Sand

Filter sand is defined in terms of effective size and uniformity coefficient. Effective size is the sieve size in millimeters that permits 10% by weight to pass. Uniformity in size is specified by the uniformity coefficient which is the ratio between the sieve size that will pass 60% by weight and the effective size.

Shape, size and quality of filter sand shall satisfy the following norms:

(a) Sand shall be of hard and resistant quartz or quartzite and free of clay, fine particles, soft grains and dirt of every description.

(b) Effective size shall be 0.45 to 0.70 mm.

(c) Uniformity coefficient shall not be more than 1.7 nor less than 1.3.

(d) Ignition loss should not exceed 0.7 per cent by weight.

(e) Soluble fraction in hydrochloric acid shall not exceed 5.0% by weight.

(f) Silica content should be not less than 90%.

(g) Specific gravity shall be in the range between 2.55 to 2.65.

(h) Wearing loss shall not exceed 3%.

IS:8419 (Part I) 1977 entitled Filtration Media sand and Gravel may be referred to for details.

7.6.3.7 Depth Of Sand

Usually the sand layer has a depth of 0.60 to 0.75 m, but for higher rate filtration when the coarse medium is used, deeper sand beds are suggested. The standing depth of water over filter varies between 1 and 2m. The free board above the water level should be at least 0.5 m so that when air binding problems are encountered, it will facilitate the additional levels of 0.15 to 0.30 m of water being provided to overcome the trouble.

7.6.3.8 Preparation Of Filter Sand

The sand to be used in the filter is specified in terms of effective size and uniformity coefficient. From a sieve analysis of the stock sand, the coarse and fine portion of stock sand that must be removed in order to meet the size specifications, can be computed in terms of p_1, the percentage of stock sand that is smaller than the desired effective size d, which is also equal to 10% of the usable sand and p_2, the percentage of the stock sand that is smaller than the desired 60 percentile size d_2.

The percentage of suitable stock sand p_3, is then $= 2(p_2 - p_1)$ because the sand lying between the d_1, and d_2 sizes will constitute half the specified sand.

To meet the specified composition, this sand can contain $0.1p_3$, of a sand below d_1, size. Hence the percentage p_4, below which the stock sand is too fine to use is

$$p_4 = p_1 - 0.1\,p_3 = p_1 - 0.2\,(p_2 - p_1) = 1.2\,p_1 - 0.2\,p_2$$

Likewise, the percentage p_5 above which the stock sand is too coarse for use is

$$p_5 = p_2 + .40\% \text{ of usable sand}$$
$$= p_2 + 0.4 \times 2(p_2 - p_1) = p_2 + 0.8(p_2 - p_1) = 1.8\,p_2 - 0.8\,p_1$$

From the size-cumulative frequency curve, the grain sizes of stock sand corresponding to p_4, and p_5 are determined (d_4 and d_5). The sizes below d_4, and above d_5 will have to be separated out from the stock sand to bring it to the desired specification. This may be done by sieving. The finer portion can also be removed in a sand washer designed to float out the particles of size smaller than d_4, by maintaining velocity in the upward flow washers slightly less than the hydraulic subsidence value corresponding to d_4, size, such that all particles less than d_4, size are floated out with the flowing water.

7.6.3.9 Filter Bottoms And Strainer Systems

The under-drainage system of the filter is intended to collect the filtered water and to distribute the wash water in such a fashion that all portions of the bed may perform nearly the same amount of work and when washed, receive nearly the same amount of cleaning. Since the rate of wash is several times higher than the rate of filtration, the former is the governing factor in the hydraulic design of filters which are cleaned by backwashing.

The most common type of under-drain is a central manifold with laterals either perforated on the bottom or having umbrella type strainers on top. Other types such as wheeler bottom-a false bottom with strainers spaced for the entire area at regular intervals or a porous plate floor supported on concrete pillars are all satisfactory when properly designed and constructed. Porous plates, however, are likely to be clogged by minute quantities of alumina which can penetrate through the filter bed which might lead to rupture.

In the case of central manifold with lateral system, the manifolds, headers and laterals are of cast iron, plastic, asbestos cement, concrete or other material. The velocity of jets issuing from perforations or orifices is destroyed by directing the openings downwards against the filter bottom and into the coarse gravel surrounding the pipes. The lost head, therefore, will be equal to the driving head during the wash. In practice this controlling head loss is set between 1 to 4.5m. A nonferrous under-drain system is preferable where the water has a low pH and is corrosive and when the correction for pH has to follow the filtration process. However, A.C. Pipes have a tendency to dissolve away in the presence of low pH alum treated waters.

The following values, may be used in design of an underdrain system consisting of central manifold and laterals.

The perforations vary from 5 to 12 mm in diameter and should be staggered at a slight angle from the vertical axis of the pipe.Spacing of perforations along the laterals may vary from 80 mm for perforations of 5 mm to 200 mm for perforations of 12 mm.

Ratio of total area of perforations in the underdrain system to total cross sectional area of lateral should not exceed 0.5 for perforations of 12 mm and should decrease to 0.25 for perforations of 5 mm.

Ratio of total area of perforations to the entire filter area may be about 0.3%.The ratio of length to diameter of the lateral should not exceed 60. The spacing of laterals closely approximates the spacing of orifices and shall be 300 mm.

The cross sectional area of the manifold should be preferably 1.5 to 2 times the total area of the laterals to minimize frictional losses and to give the best distribution. It is useful to check the design for uniformity of distribution of wash water in laterals of the under- drains.

The central manifold with lateral type of underdrainage system is shown in Fig. 7.18.

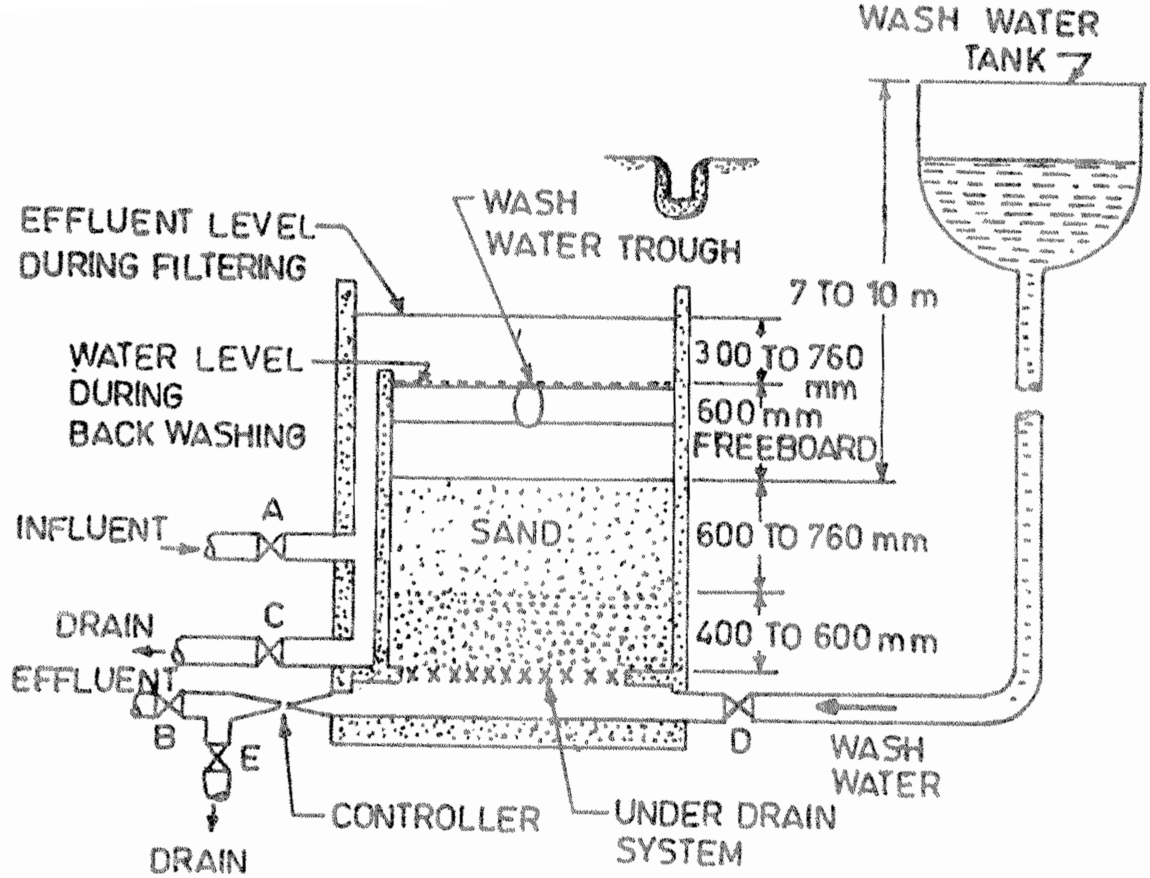
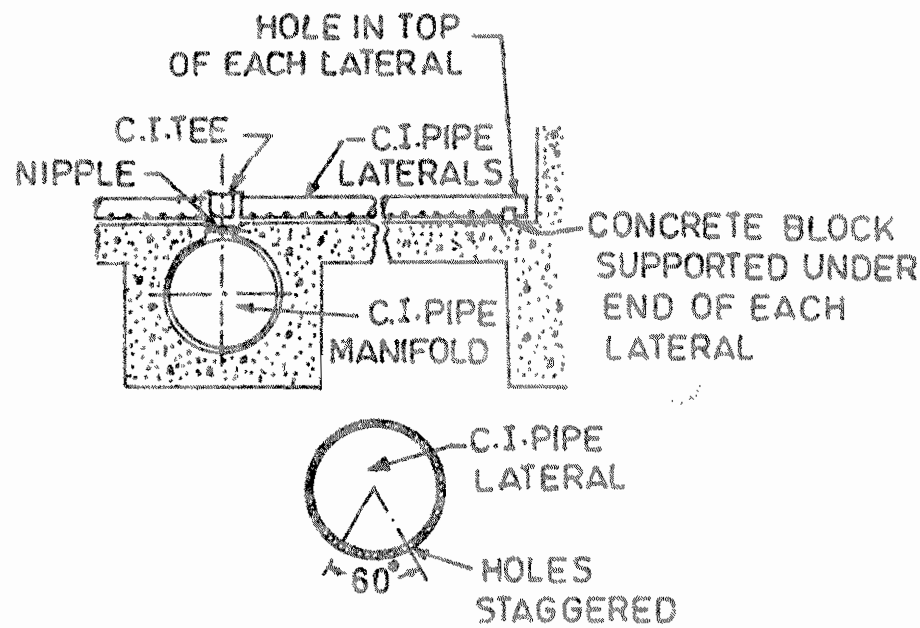


FIG. 7.17 DEFINITION SKETCH FOR OPERATION OF DOWNFLOW, GRANULAR–MEDIUM GRAVITY–FLOW FILTER



DETAIL OF LATERAL DRILLING

FIGURE 7.18 : A PERFORATED PIPE UNDERDRAIN

7.6.3.10 Filter Gravel

Gravel is placed between the sand and the underdrainage system to prevent sand from entering the underdrains and to aid uniform distribution of wash water. The gravel should accomplish both purposes without being displaced by the rising wash water. Sizes of gravel vary from 50 mm at the bottom to 2 to 5 mm at the top with a 0.45 m depth. The faster the rate of application of water, the larger the gravel size required. Reference may be made to IS: 8419 Part (I)-1977 for filter gravel.

The depth will vary according to the type of filter bottom and strainer system used, except in the case of porous bottom where no gravel is required. Wheeler bottoms and other false bottoms may be subsituted for part of coarser layer of gravel. The filter gravel shall be as spherical as possible, hard, clean and uniform in quality and also shall not contain such impurities as dirt and clay. Size of gravel and depth of gravel layer shall be determined in accordance with the following rules:

(a) for strainer or Wheeler type underdrain system, gravel shall be of 2 mm minimum size, 50 mm maximum size and 0.30 to 0.50 m deep; and

(b) for perforated pipe under-drain system, gravel shall be 2 mm minimum size, 25 mm maximum size and 0.50 m, in depth.

The filter gravel shall be classified by sieves into four or more size grades, sieves being placed with the coarsest on top and the finest at the bottom.

7.6.3.11 Wash Water Gutters

Materials used for wash water gutters include concrete, asbestos cement, plastic, cast iron and steel. While the horizontal travel of dirty water over the surface of the filter is kept between 0.6 to 1.0 m before reaching the gutter, there are successful units with troughs eliminated and having only main gutters where the dirty water travel has been as high as 3 m It is uneconomical to place wash water gutters against the side walls of the filter. The upper edge of the washwater gutter should be placed sufficiently near to the surface of the sand so that a large quantity of dirty water is not left in the filter after the completion of washing. At the same time, the top of the wash water gutter should be placed sufficiently high above the surface of the sand so that sand will not be washed into the gutter. The edge of the trough should be slightly above the highest elevation of the sand as expanded in washing. Where this height cannot be determined by test, a convenient rule is to place the edge of the gutter as far above the undisturbed sand surface as the washwater rises in one minute. Air and water should not be applied simultaneously with such gutter heights. The gutter should be large enough to carry all the water delivered to it with at least 50 mm between the surface of the water flowing in the gutter and the upper edge of the gutter. Any submergence of the gutter will reduce the efficacy of the wash. The gutter may be made with the same cross-section throughout its length or it might be constructed with varying crosssection increasing in size towards the outlet end. The bottom of the gutter should clear the top of the expanded sand by 50 mm or more.

The troughs are designed as free falling weirs or spillways, For free falling rectangular troughs with level invert, the discharge capacity Q in m^3/s may be computed from the formula

$$Q = 1.376\ bh^{3/2} \qquad (7.32)$$

Where b is the width of the trough in m and h is the water depth in m.

7.6.3.12 High Rate Backwash

Back wash should be arranged at such a pressure that sand should expand to about 130-150% of its undisturbed volume. The pressure at which the wash water is applied is about 5m head of water as measured in underdrains.

Normally, the rate at which wash water is applied, where no other agitation is provided, is 36 m (600 lpm/m^2) for a period of 10 minutes. The tendency in design is towards higher rates of washing, primarily because of the larger sizes of sand being used, which require a faster application of water for equal expansion unless surface agitation by auxiliary means is provided. The maximum friction that particles which are free to move and expand can offer is their submerged weight in water. Increasing the flow, any further beyond this point, may lead to the carryover of the grains along with the wash water. For high rate wash, the pressure in the underdrainage system should be 6 to 8 m with the wash-water requirement being 40-50 m/hr for a duration of 6 to 10 minutes.

The supply of wash water can be made through an overhead storage tank or by direct pumping or by tapping the rising main of the treated water if the clear water motors are not overloaded. The capacity of the storage tank must be sufficient to supply wash water to two filter units, at a time, where the units are four or more.

7.6.3.13 Surface Wash

The upper layer of the filter bed become the dirtiest and any inadequate washing will lead to the formation of mud balls, cracks and clogged spots in the filters. These troubles are overcome by adequate surface wash which can be accomplished by stirring the expanded filter bed mechanically with rakes, hydraulically with jets of water directed into the suspended sand or pneumatically with air, either during or more commonly before expansion. The latter two methods being common, are discussed.

(a) Hydraulic System

1. The fixed type surface wash system shall consist of pipes not less than 25 mm in diameter arranged vertically at intervals of 0.6 to 0.9 m. The lower ends of the pipes are to be situated to 0.1 m approximately over the sand surface and nozzles shall be located on the lower ends of pipes. An alternate fixed type may consist of piping horizontally arranged at intervals of 0.6 m approximately at a height of 0.05 to 0.1 m over the sand surface. The horizontal pipes shall be perforated at intervals of 0.3 m approximately and provided with non-clogging orifice units to prevent entry of the filter media,

2. The rotary type shall consist of rotating units suspended at a height of 50 to 75 mm at adequate intervals over the bed to provide complete coverage. jet nozzles

shall be located on side and bottom of arms and jet action of water causes the arms to rotate at a rate of 7 to 10 rpm.

(b) Air Wash System

In the air wash system, compressed air is used to secure effective scrubbing action with a smaller volume of wash water. The air may be forced through the under-drains before the wash-water is introduced or through a separate piping system placed between the gravel and the sand layer. Though the former results in better washing, the gravel is likely to be disturbed. With the former procedure, free air of about 36 to 45 m/hr (600 to 900 lpm/m^2 of the filter area) at 0.35 kg/cm^2 is forced through the underdrains until the sand is thoroughly agitated, for a duration of about 5 minutes following which, wash water is introduced through the same under-drains at a rate of 24 to 36 m/hr (400 to 600 lpm/m^2 of area). On the other hand, with the latter procedure, while water is forced through the under-drains, about the same volume of air is forced simultaneously through a separate piping. In the practice of backwashing employing conjunctive air and water wash, air is usually applied at a rate of 45-50 m/ hr and water at 12-15 m/hr.

7.6.3.14 Operation Of Filters

Before starting a filter it is backwashed at increasing rates until the sand bed has been stratrified vertically by the wash water which carries various sizes of sand to different levels. The filter run of rapid gravity filters depends on several attendant factors. It is necessary to calculate the total loss of head in backwashing to arrive at the pump capacity and staging height of backwash water reservoir. The total loss of head includes loss due to expansion of sand, loss in orifices or underdrainage system, loss in incoming pipe and height of wash water gutter with respect to underdrainage system. The loss of head immediately after washing should not exceed 0.3 m. The finer the sand or the greater the rate of filteration, the greater the initial head loss. The head loss builds as the filter grows dirty during a run. It is usual to allow a filter head loss of 1.8 to 2 m before cleaning such filters. Under no circumstances, a build-up of negative head within the filter media be allowed.

The duration of the washing process varies for different conditions of cleaning of the filter, sizes and character of filter media, rate of wash and the desired quality of filtrate. It should not normally exceed 10 minutes.

In a properly operated plant, the quantity of wash water used should not exceed 2% of the total amount of water filtered. Lower amounts are possible where large quantities of wash water are involved and water is scarce, feasibility of the reuse or recycling of wash water after requisite treatment has also to be explored. Following the washing process, it is usually advisable to waste the first few minutes of flow through the filter, unless the quality of the filter effluent immediately following the wash may make this unnecessary. A turbidity of 1.0 NTU or less, measured by an accurate instrument, is the best criterion of the stability of effluent of a freshly washed filter. The test can be used to advantage in most plants.

The water standing on the bed at the close of of wash should be clear with a turbidity not exceeding 10 NTU. In a well designed and operated filter, there should be no air binding either during filtration or during washing; there should be no carry away of sand with the washed water and the sand bed should settle down fairly uniformly without undulations.

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Formation of mud balls and their retention in the bed even after washing indicates poor performance. At the commencement of the filter run after a wash, the initial loss of head should not exceed 0.3 m.

7.6.3.15 Hydraulics Of Filtration

The head loss, h, through a clean filter bed of depth l can be computed using Kozeny's equation

$$\frac{h}{l} = \frac{k}{g} \cdot \frac{\mu}{\rho} . v . \frac{(l-f)^2}{f^3} \left(\frac{A}{V}\right)^2 \qquad (7.33)$$

Where,

k = residual dimensionless coefficient, about 5 under most condition of water filtration,

μ = absolute viscosity of water, $\frac{(N.s)}{m^2}$

ρ = density of water, (Kg/m^3)

v = macroscopic velocity of filtration, (m/s)

f = porosity of clean sand bed, dimensionless

A = surface area of the grains (m^2)

V = volume of the grains (m^3)

For unisize sperical medium particles of diametere d

$$\frac{A}{V} = \frac{6}{d}$$

For non-sperical grains, sphericity is defined as the surface area of the equivalent volume sphere to actual surface of non-spherical particles. The sphericity, ψ assumes values of 1.0 for spherical grains, 0.98 for rounded grains, 0.94 for worn grains, 0.81 for sharp grains, 0.78 for angular grains and 0.70 for crushed grains for sand medium

For stratified beds, as obtainable in rapid sand filters after back washing, the head loss in a clean bed is the sum of the head losses in successive sand layers. If p_i, is the fraction of medium of sieved size di, the head loss is given by

$$\frac{h}{l} = \frac{k}{g} \cdot \frac{\mu}{\rho} . v . \frac{(1-f)^2}{f^3} \left(\frac{6}{\psi}\right)^2 \sum_{i=1}^{n} \frac{p_i}{di^2} \qquad (7.34)$$

For unstratified beds e.g. slow sand filter, the head loss becomes

$$\frac{h}{l} = \frac{k}{g} \cdot \frac{\mu}{\rho} . v . \frac{(1-f)^2}{f^3} \left(\frac{6}{\psi} \sum_{i=1}^{n} \frac{p_i}{di}\right)^2 \qquad (7.35)$$

7.6.3.16 Hydraulics Of Backwashing

High rate granular filters are backwashed to remove the impurities lodged in the medium matrix. The hydraulics of backwashing concerns with the determination of head loss across the filter bed during backwashing and to estimate backwash velocity at any required level of bed expansion and con-commitant porosity of expanded bed.

As the water is applied in upflow mode to a granular medium or media, frictional resistance is offered by the filter grains due to skin friction and form drag. The initial effect at low velocities of flow is to result in reorientation of the particles to minimize frictional resistance. At low backwash velocities, the filter bed does not expand and its porosity does not change. The head loss or pressure drop is a linear fuction of upward flow velocity at low velocities. As the water velocity is increased, the frictional resistance also increases till it reaches a value equal to the gravitational force acting upon the filter grains. Any further increase in the velocity of water fluidizes the filter bed resulting in bed expansion and increasing porosity of filter bed.

(a) Head loss across filter bed

The maximum frictional resistance that can be offered by the filter grains in fluidized state is their submerged weight. The head loss across the filter bed in fluidized condition is given by the equation:

$$\frac{h_b}{l_e} = \frac{(\rho_m - \rho)}{\rho}(1 - f_e) \tag{7.36}$$

Where,

h_b = headloss across filter bed during backwashing, (m)

l_e = height of the expanded bed, (m)

ρ_m = mass density of the filter grains (Kg/m^2)

ρ = mass density of water, (Kg/ml)

f_e = porosity of expanded bed, dimensionless

Since the grain volume does not change before and during backwashing,

$(1-f)l = (1-f_e)l_e$

Eq. (7.36) can be rewritten using eq. (7-37)

$$h_b = \frac{(\rho_m - \rho)}{\rho}(1 - f)l \tag{7.38}$$

(b) *Estimation of Backwash Velocity*

Several approaches are available for computation of backwash velocity to achieve a desired degree of bed expansion and attendant expanded bed porosity or to estimate bed expansion and expanded bed porosity at a given backwash velocity.

According to one of the approaches, first minimum fluidization velocity (v_{mf}) which is the superficial fluid velocity required to initiate fluidization of the bed is computed from the empirical nonhomogenous equation:

$$V_{mf}\,(gpm\ /\ ft^2) = \frac{0.00381\,(d_{60})^{1.82}\left[W_s(W_M - W_S)\right]^{0.94}}{\mu^{0.88}} \tag{7.39 (a)}$$

where,

v_{mf} = minimum fluidization velocity, U.S. gallons/minute/ft^2

d_{60} = 60% finer size of sand, mm

W_m, W_s = Specific weights of filter medium and water, lbs/ft^3

μ = absolute viscosity, centipoise

or in SI units

$$V_{mf}\,(m\ /\ h) = \frac{1.185 \times 10^{-7}(d_{60})^{1.82}\left[\rho(\rho_m - \rho)\right]^{0.94}}{\mu^{0.88}} \tag{7-39 (b)}$$

Where,

d_{60} = 60% finer size of sand (mm)

ρ = mass density of water, (kg/m^3)

ρ_m = mass density of filter medium, (kg/m^3)

μ = dynamic viscosity of water, (kg/m.s)

The minimum fluidization velocity is used to compute Reynolds number, Re_f.

$$Re_f = \frac{\rho V_{mf} d_{60}}{\mu} \tag{7.40}$$

If Re_f is greater than 10, a multiplying correction factor K_R must be applied to V_{mf}:

$$K_R = 1.775\ Re_f^{-0.272}$$

The unhindered settling velocity, V_S, of the hypothetical average particle is then calculated using equation:

$$V_S = 8.45\ V_{mf} \tag{7.41}$$

The Reynolds number for this particle, based on V_S, is determined

$$Re_0 = \frac{(\rho\ V_s\ d_{60})}{\mu} = 8.45\ Re_f \tag{7.42}$$

Using the value of Re_0 the expansion coefficient, n is computed using the equation:

$$n = 4.45\ Re_0^{-0.1} \text{ for } 1<Re_0<500 \tag{7.43}$$

This value of expansion coefficient is finally used to compute the required backwash velocity, V_b, for a given bed expansion or expanded bed porosity.

$$V_b = V_{mf}\left(\frac{f_e}{f}\right)^n \tag{7.44}$$

The expanded bed porosity, f_e, can be determined from the eq. (7.37) for a given bed expansion.

The expansion of graded filter sands at different temperatures has been predicted to an excellent degree by the above calculations. However, the application of this procedure to graded coal beds has yielded rather poor expansion prediction.

In another approach given in literature the expanded bed porosity of a fluidized bed is expressed by the equation:

$$f_e = \left(\frac{V_b}{V_s}\right)^{0.2} \qquad (7.45)$$

Where, V_s is the unhindered settling velocity of the filter grain particle, determined using stokes law.

7.6.3.17 Optimum Backwashing

It is opined that collisional interactions between media grains do not occur in the fluidized state during backwash and the principal mode of cleaning is by hydrodynamic shear. Theoretically, supported with experimental evidence, it has been demonstrated that maximum hydrodynamic shear occured at expanded bed porosities of about 0.7.

According to Camp and Stein's equation, applied to a backwashed filter,

$$\frac{dv'}{dl} = \left[\frac{gv'}{\nu}\frac{dh}{dl}\right]^{\frac{1}{2}} \qquad (7.46)$$

Where $\frac{dv'}{dl}$ = velocity gradient within pores

v' = velocity within pores

$\frac{dh}{dl}$ = head loss per unit length

Hydrodynamic shear 'γ' is given by

$$\gamma = \mu\left(\frac{dv'}{dl}\right) \qquad (7.47)$$

combining equations (7.47) and (7.46)

$$\gamma = \left[\mu\left(\frac{gv'}{\nu}\cdot\frac{dh}{dl}\right)\right]^{\frac{1}{2}} \qquad (7.48)$$

The velocity within pores can be expressed by

$$v' = \frac{v_e}{f_e} = K_e(fe)^{n-1} \qquad (7.49)$$

$$and = \frac{dh}{dl} = \frac{(\rho_s - \rho)}{\rho}(1 - fe) \tag{7.50}$$

$$\gamma = K.[f_e^{\,n-1} - f_e^{\,n}]^{1/2} \tag{7.51}$$

where $K = [\mu g\, Ke(\rho_s - \rho)^{0.5}]$

Differentiating Eq. 7.51 and equating to zero:

$$\frac{d\gamma}{df_e} = K.\frac{1}{2}\left[f_e^{\,n-1} - f_e^{\,n}\right]^{3/2}\left[(n-1)f_e^{\,n-2} - nf_e^{\,n-1}\right] = 0$$

Optimization of the equation for shear intensity, by differentiating the equation and equating to zero yields the following expression for the porosity of maximum hydrodynamic shear:

$$f_e = \frac{n-1}{n} \tag{7.52}$$

According to this equation, the maximum hydrodynamic shear occurs in a fluidized bed at porosities of 0.68 to 0.71 for typically sized filter sands which corresponds to an expansion of 80 to 100%. However, the curve of the hydrodynamic shear versus porosity is quite flat, indicating that washing at porosities different from the theoretical optimum does not result in a major decrease in the efficiency of cleaning process. Optimal cleaning has been observed in some cases at expansion of 16-18% only.

It has been found that there is lack of abrasion during water backwash and therefore a backwashing with water alone is inherently a weak cleaning process. For effective cleaning, abrasion resulting from collision, between grains is achieved by auxiliary process like surface wash or air scour (Section 7.6.3.13).

7.6.3.18 Appurtenances

Filter appurtenances include manually, hydraulically or electrically operated sluice valves on the influent, effluent, drain and wash water lines; measuring devices such as venturi meters; rate controllers activated by measuring device; loss of head and rate of flow gauges; sand expansion indicators; wash water controllers and indicators; operating tables and water sampling devices; and ejectors and sand washers; wash water tanks and pumps.

(a) Rate of Flow Controllers

The primary purpose of rate of flow controllers is to regulate the flows of liquids in the lines and specifically, in filter plant, to maintain at all times a uniform rate of filtration through each filter unit. Without these control features in the filter effluent lines, raw water will pass through the sand bed at different velocities, higher when the sand bed is clean and lower when coagulated deposit has accumulated on its surface.

Sudden changes of rate of flow also must be avoided if the filter medium is to be maintained in an unbroken and efficient condition. Any changes in rate must be gradual and predetermined maximum must not be exceeded. Such unfavourable operating conditions may be eliminated by the use of rate of flow controllers.

The flow can also be controlled by means of a V-notch or a rectangular weir or a venturi tube.

Rate of flow controller may be either of double beat type or venturi type. The later type consists of a venturi section, diaphragm chamber arrangement, valve mechanism and casing, counter-weighted scale beam group and recovery out-let section. By virtue of the arrangement of the parts, straight line flow through the unit is simulated.

Water flowing through the venturi section produces different pressures at the main and throat, due to the difference of velocities at these points. Since connections from the main and throat lead to the upper and lower halves, respectively, of the diaphragm chamber, these differential pressures are reflected directly on the piston, moving it a certain distance, dependent on the difference between the pressures being exerted. Since downward pressure on the top of the piston is greater than upward pressure from below, a downward pull that is balanced by the counter-weight on the long arm of the beam is transmitted to the scale beam. This balance of counter-weight and piston load regulates the valve opening and limits the maximum rate of discharge through the controller.

In filter operation, the controller, by virtue of its throttling action, uses up all the head due to the difference in raw and filtered water which is not required to overcome friction due to sand, piping, velocity head, etc., and as the loss of head through the sand increases, the head consumed by the controller diminishes by a corresponding amount. During the entire operation, therefore, the rate of filtration remains practically constant.

However, it must be emphasized that rate of flow controllers require proper operation and maintenance to ensure that filtration is done at constant rate. These devices are omitted where declining rate of filtration is adopted.

(b) Filter Gauges

Filter gauges are essential to the operation of the modern filter plant in order to measure accurately the rate of flow through each filter box and to determine the loss of head occuring at any given time during the filter run. Gauges are available in various combinations of rate of flow and loss of head, both indicating and recording or as single recording or indicating units.

These gauges use the float and mercury principle for the conversion of differential pressure into measurement of loss of head or rate of flow. The primary pressure differential producing device required for the rate gauge usually is the venturi section of the effluent rate controller, connections to the high and low pressure sides of the gauge cylinder being made to the main and throat sections of the controller. The differential pressure for the gauge is the difference between the water level in the filter box and the pressure head in the effluent pipe, pressure connections being led from these sources to the high and low pressure gauge cylinder taps.

Piezometers can also be used for the purpose, though they suffer from the disadvantage that they have to be cleaned from time to time. They are simpler, more positive and much less expensive than the conventional types of instruments.

(c) Sand Expansion Gauges

Properly designed sand expansion gauges accurately determine the correct percent of sand expansion and the proper washing cycle for each filter bed. This form of gauge operates by means of a conical metal float.

The conical float, being counter-weighted to suit the specific gravity of the filter media moves upward and continually rests upon the surface of the media as it is expanded. The float is so designed that it will faithfully follow the surface of the media as it rises or falls in the filter during the washing cycle.

In some plants, the expansion of sand is not given emphasis and the fluidization is checked by means of probing with a vertical rod during the backwash cycle to see whether the probe will easily go down to the gravel.

7.6.3.19 Pipe Gallery

The influent, effluent, wash and waste water pipes together with rate controllers and appurtenances are placed in the pipe gallery. Galleries should be well designed to provide adequate space, ventilation drainage and easy accessibility to all pipe-work and other fittings.

7.6.3.20 Limitations Of Rapid Sand Filters

The inherent drawback of the rapid sand filtration system is the surface clogging tendency due to unfavourable stratification of sand medium. A rapid sand filter consists of a sand bed which becomes stratified after back washing The size gradation is from fine to coarse with finest sand particles being at the top of bed. Since majority of the impurities are removed and stored in the limited pore space available in top sand layers, it leads to surface clogging with relatively quicker build up of head loss at higher velocities of filtration leading to the under utilisation of sand bed. Consequently, the rapid sand filters have been operated at lower filtration rates (around 5 m/hr) with filter runs of the order of 24 hours. Another drawback of fine to coarse size gradation of filter medium is the possibility of poor filtrate quality resulting from the non-removal of finer floc particles which escape the top sand layers also break through the lower layers containing larger-size sand medium.

Various approaches have been recommended to overcome the above limitations of the rapid sand filters. These include up-flow filtration, horizontal-flow filtration and dual-media and multimedia filtration. Central to the development of these concepts is the principle of contacting the impurity-laden water first with the layers of filter medium having maximum pore size and pore space to accommodate the arrested impurities. As water travels deeper into the filter bed, it comes in contact with filter bed layers containing smaller pore sizes resulting in removal of even very fine floc particles. This leads to better-quality filtrate and greater utilisation of lower layers to remove impurities. The dual media and multi-media filters which are being increasingly used can be operated at higher rates of filtration with production of higher quantities of filtered water of good quality per filter run compared to rapid sand filters.

7.6.4 Rapid Gravity Dual Media Filters

The rapid gravity dual media filters are filters containing two media, normally coal and* sand, and water is applied in downward direction under gravity.

7.6.4.1 Constructional Features

The enclosure tank containing filter media is usually a rectangular box, made of concrete or masonry. The plan area of these filters may range between 40 and 200 m^2 with depths between 2.5 and 3.5 m. The filter media is supported on gravel laid over top of the under-drainage system. In addition to the under-drainage system, used for collecting filtered water and distributing the backwash water, the tanks have troughs spanning across the length or width of filter for distribution of water to be filtered and for collection of wash water. The troughs remain submerged during filtration and their top edge is normally kept 600 mm above the filter medium to prevent loss of medium during backwash and to minimise the amount of dirty water left above the filter bed at the end of the wash.

The filters are commonly arranged in rows on one or both sides of a pipe gallery. The gallery houses the influent, effluent, wash water supply, wash water drainage piping, valves and other appurtenances including rate of flow controller. The pressure gauges to indicate head loss and venturimeter or rate of flow recorder are also located above and/or below the gallery floor.

7.6.4.2 Filtration Media

With a view to maintain coarse to fine gradation of pore sizes and pore volume with increasing depth of filter bed, two media of different density and sizes are chosen. The top layer consists of a lower density material like coal having larger particle size over a layer of higher density material like silica sand having smaller diameter particles. Since in India anthracite coal is not easily available. the coarse medium may consist of high grade bituminous coal or crushed coconut shell which have been recommended for use after laboratory and field trials. The effective size (E.S.) of coal (specific gravity 1.4) is usually 1mm (0.85-1.6 mm range) with uniformity coefficient (U.C.) of 1.3 to 1.5. Depths of 0.3 to 0.4 m have been reported to be satisfactory without excessive head loss build up and these depths can flocculate particles besides removing large flocculated impurities. The finer media-layer usually consists of 0.3 – 0.4 m thick silica sand (specific gravity 2.65) with effective size of around 0.5 mm (0.45 to 0.6 mm range) and uniformity coefficient of 1.3 to 1.5.

The basic principle in designing the dual media bed is to have coal as coarse as is consistent with solids removal to prevent surface blinding but to have the sand as fine as possible to provide maximum solids removal subject to the constraint that the finer sand should not be present in the upper layers after backwashing in appreciable quantity.

In addition to high grade bituminous coal, crushed coconut shell has been effectively used as coarse media in dual media filters. The size ranges from 1.0 to 2.0 mm with depths of 03 - 0.4 m. The uniformity coefficient is below 1.5 and specific gravity 1.4. The sand used in conjunction with crushed coconut shell has effective size varying between 0.44 to 0.55 mm with uniformity coefficient below 1.5. The sand depths may very between 0.30 and 0.4 m. Water treatment plants with capacities ranging between 1 to 26 mld have been constructed employing dual media filters using crushed coconut shell and sand.

Anthracite coal has been extensively used in dual media filters. It is recommended that 0.4 -0.75m of anthracite coal of effective size of 1.0 to 1.6 mm (specific gravity 1.45-1.55) be used above a sand layer, 0.15-0.30 m. The effective size of sand may very between 0.45 to 0.8 mm with 0.45 mm being preferred. The sand is silica sand with specific gravity of 2.65.

7.6.4.3 Design Of Media Depth And Media Sizes

a) Design of Media Depth

The efficiency of removal of suspended particles is a function of the surface area of the media grains. For a filter of depth l comprising of N particles of average size d and sphericity ψ

$$A = \frac{1(1-f)}{\frac{\pi(\psi d)^3}{6}} \cdot \pi(\psi d)^2 \tag{7.53}$$

$$A = \frac{6(1-f)}{\psi}\left(\frac{1}{d}\right) \tag{7.54}$$

The equation can be employed to design the depths of filter. For example, for typical high rate filters $f=0.45, \psi = 0.8$ and $\left(\frac{1}{d}\right) = 680$, $A \cong 2800\ m^2$. Since the effective size of sand is normally specified, $\left(\frac{1}{d}\right) = 680$ corresponds to $\left(\frac{1}{d_{10\%}}\right) = 950$ where $d_{10}\%$ is the effective size. Figures can be developed for predetermined value of A, based on pilot data, between the effective size of medium and filter medium depth for different values of V. These figures can be used to estimate depths of various combinations of dual media.

(b) Design of Media Sizes

The dual media filters, consisting of coarse but lighter medium particles on top of finer but heavier particles, must retain their stratified character during backwashing and resettling. Equal expansion during backwashing for dual media comprising of coal and sand indicate equal fluidization velocity for both media. It can be shown that

$$\frac{d_u}{d_l} = \left(\frac{\psi_l}{\psi_u}\right)\left[\frac{\rho_l - \rho}{\rho u - \rho}\right]^{\frac{1}{2}} \tag{7.55}$$

Where subscripts u and l respectively denote the largest grain within the upper layer (coal) and the smallest grains within the lower layer (sand). It follows that mixing during settling as well as during expansion determines the maximum allowable ratio of the grain sizes in the two layers.

For sharp interface and no intermixing, the ratio of maximum diameter of coal to the minimum diameter of sand that will ensure both equal expansion and equal settling can be computed using above mentioned equation. For a density of coal of 1.5 and its sphericity 0.70 and sphericity and density of sand of 0.85 and 2.65, this ratio is

$$\frac{d_u}{d_l} = \frac{0.85}{0.70} = \left[\frac{(2.65-1)}{(1.5-1)}\right]^{0.5} = 2.26 \cong 2.3$$

If partial intermixing is to be achieved the size of the coarsest coal must be more than 2.3 times the minimum diameter of sand for characteristic of coal and sand given.

7.6.4.4 Filtration Rates And Filtrate Quality

Dual -media and multimedia filters have been successfully operated at rates of filtration ranging from 15 to 20 $m^3/m^2/hr$ with acceptable filtrate quality. Field trails in India using high grade bituminous coal indicate that even with inadequate pretreatment of filter influents as obtainable in Indian conditions, filtration rates of 16 $m^3/m^2/hr$ could be recommended with filter run of at least 12 hours, though much higher rates of filtration upto 24 $m^3/m^2/hr$ could be employed if proper pretreatment is available. Filtrate turbidities are generally less than 1 NTU and coliform removal is around 95.

In general, it may be recommended to operate dual media filters at higher rates of 7.5 to 12 $m^3/m^2/hr$. The backwash rates of 42 to 54 $m^3/m^2/hr$ (700-900 lpm/m^2) have been employed to clean the filters.

7.6.5 MULTIMEDIA FILTERS

The multimedia filters normally contain three media such as anthracite coal, silica sand and garnet sand with specific gravities being around 1.4, 2.65 and 4.2. The size of media may vary from 2 mm at the top to 0. 15 mm at the bottom. A typical trimedium filter may contain 0.45 m of coal with an effective size of 1.4 mm, followed by 0.23 m of silica sand of effective size of 0.5 mm and 0.08 m of garnet sand having an effective size of 0.3 mm.

Media of polystyrene, anthracite, crushed flint sand, garnet and magnetite whose specific gravities are 1.04, 1.40, 2.65, 3.83 and 4.90 respectively are being tried.

7.6.6 PRESSURE FILTERS

7.6.6.1 General

Based on the same principle as gravity type rapid sand filters, water is passed through the filter under pressure through a cylindrical tank, usually made of steel or cast iron, wherein the underdrain, gravel and sand are placed. They are compact and can be prefabricated and moved to site. Economy is possible in certain cases by avoiding double pumping. Pretreatment is essential. The tank axis may be either vertical or horizontal.

7.6.6.2 Disadvantages

Pressure filters suffer from the following disadvantages:

(a) The treatment of water under pressure seriously complicates effective feeding, mixing and flocculation of water to be filtered.

(b) In case of direct supply from pressure filters, it is not possible to provide adequate contact time for chlorine.

(c) The water under filtration and the sand bed are out of sight and it is not possible to observe the effectiveness of the back wash or the degree of agitation during washing process.

(d) Because of the inherent shape of the pressure filters it is difficult to provide wash water gutters effectively designed so that the material washed from the sand is discharged to waste and not flushed back to other portions of the sand bed.

(d) It is difficult to inspect, clean and replace the sand, gravel and underdrains of pressure

filters.

(e) Because the water is under pressure at the delivery end, on occasions when the pressure on the discharge main is released suddenly, the entire sand bed might be disturbed violently with disastrous results to the filter effluent.

In view of these disadvantages, pressure filters are not recommended for community water supplies, particularly for large ones. They may be used for industrial needs and swimming pools.

7.6.7 DIATOMACEOUS EARTH FILTERS

Diatomaceous fiters, are not advocated for publiuc water supplies. Their utility is restricted to temporary and emergency water supplies of a limited nature where other arrangements are not easyor feasible.

The medium consists of diatomaceous earths which are skeletons of diatoms mined from deposits laid down in seas.

The filtering medium is a layer of diatomaceous earth built up on a porous septum by recirculating a slurry of diatomaceous earth until a firm layer is formed on the septum. The precoat thus formed is used for straining the turbidity in water. For this, diatomaceous earth is applied at 0.5 to 2.5 kg/m^2 of septum. Some times, when the turbidity is very high, the diatomaceous earth will have to be added to the incoming water as body feed. Body feed is added at three times the solids when organic slimes are present. Filtration rates range from 7.2 to 18 $m^3/m^2/hr$.

7.6.8 Additional Modifications of Conventional Rapid Gravity Filters

7.6.8.1 Constant And Declining Rate Filtration

(a) Constant Rate Filtration by Influent Flow Splitting

In conventional rapid sand filters, constant rate of flow is maintained by installing a rate of flow controller on the effluent line. These rate of flow controller can be quite complex and high in initial and maintenance Costs. Alternative systems have been proposed which are relatively simple to build, operate and maintain.

One of the simplest methods is rate control by influent flow splitting which is depicted in Fig. 7.19. The filter influent is divided equally among all the operating filters in series by means of a weir at each filter inlet. The size of the filter influent conduit is kept relatively large so that the head loss is not significant and the water level does not vary significantly along the length of the conduit. This helps in maintaining nearly same head on each of the weir and filter influent is equally split among all the operating filters. The filtration rate is controlled jointly for all the filter units by the inflow feeding rate. At the beginning of filter run when a backwashed filter is put into service, the level of water in that filter is minimum. As the filtration proceeds and head loss builds up, the water level rises in the filter till it reaches the maximum permissible level above the filter bed, which may be, for example, equal to the level of influent weir. The filter is then taken out of service for backwashing.

The advantages of this system include elimination of rate controllers and slow and smooth changes in rates due to gradual rise and fall of water level above filter bed with less

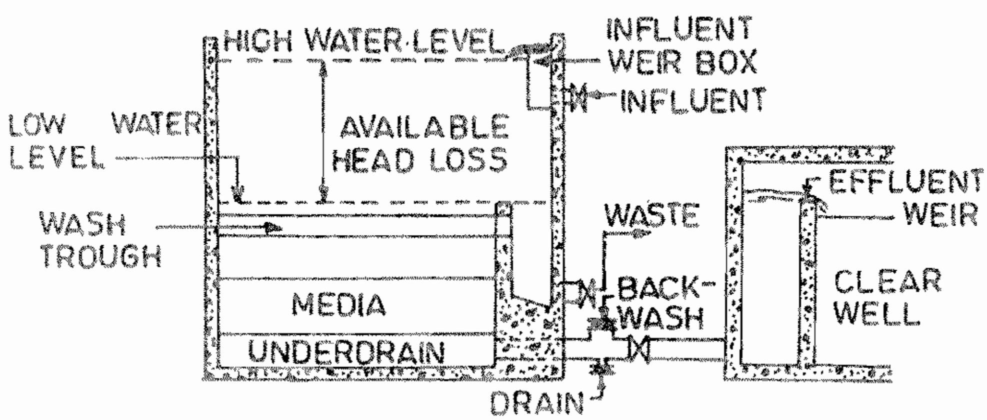

FIGURE 7.19 : GRAVITY FILTER ARRANGEMENTS FOR RATE CONTROL BY INFLUENT FLOW SPLITTING

harmful effects on filtrate quality in comparison to filters having rate of flow controllers. To completely eliminate the possibility of negative head in the filter, the effluent control weir must be located above filter media as depicted in the Figure.

The only disadvantage of the influent flow splitting system is the additional depth of the filter box which is 1.5 to 2 m more than in conventional filters.

(b) Declining Rate Filtralion

This is also referred to as variable declining rate filtration. In this system, the filter influent enters below the low water level of the filters and not above as in the case of influent flow splitting system described in section 7.6.8.1 (a). A relatively large influent header (pipe or channel) serves all the filters and a relatively large influent valve is used for each individual filter. This results in relatively small head losses in the influent header and influent valve and water level is essentially the same in all operating filters at all times. The essential features for variable declining rate filtration system are shown in Fig. 7.20 No rate of flow controllers are used in this system also.

During the course of filtration by a series of filters being served by a common header, as the filters get clogged, the flow through the dirtiest filters decreases most rapidly. This causes redistribution of load among all of the filters increasing the water level providing the additional head needed by the cleaner filters for handling additional flow. Therefore, the capacity lost by the dirtier filters is picked up by the cleaner filters.

The advantage claimed for this system include significantly better filtrate quality than obtained with constant-rate filtration, and less available head needed than that required for constant-rate operation.

Another type of declining rate filtration is called "controlled-head" operation. In this type of filters, the filter effluent lines are connected to a common header. A fixed orifice is built into the effluent piping for each filter so that no filter, after washing, will take an undue share

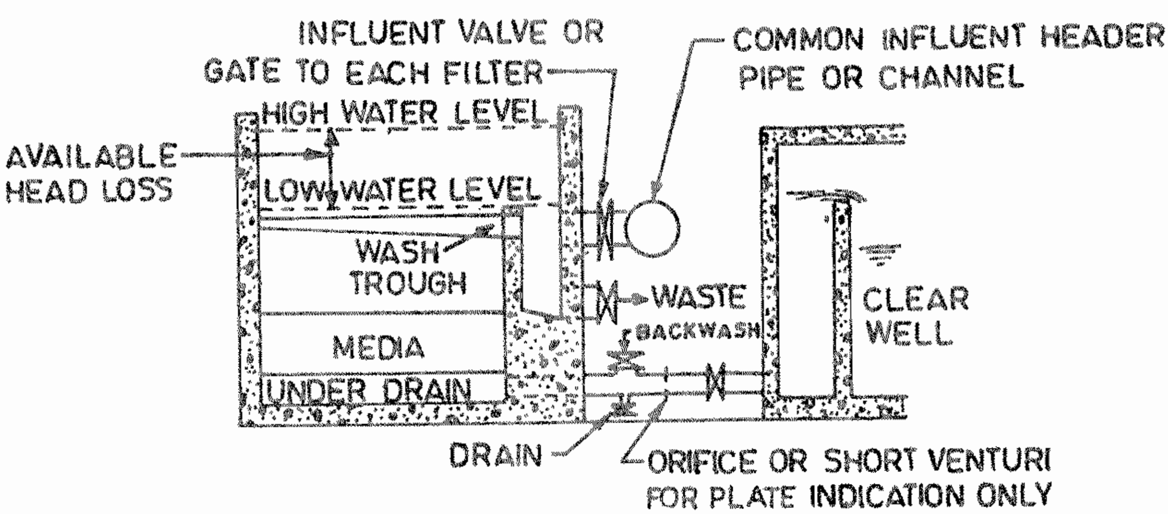

FIGURE 7.20 : GRAVITY FILTER ARRANGED FOR VARIABLE DECLINING RATE OF FILTERATION

of the flow. The filtered water header pressure may be regulated by a throttle valve which discharges to a filtered water reservoir. Costly rate controllers are replaced with fixed orifices and, therefore, would make the units economical particularly in large water works involving batteries of filters. The quality of water produced by the declining rate filters and filters controlled by conventional rate controllers are reported to be almost the same. For equal durations of filter runs the total output per day from a declining rate filter is higher than that in the conventional one. In a group of filters operating at an average rate of 6 $m^3/m^2/hr$, fixed orifice will be so designed that a recently cleaned filter will begin operation at 9 $m^3/m^2/hr$ while the filter next in line for backwashing will have slowed down to about 3 $m^3/m^2/hr$. Usually the depths of filter boxes for declining rate filters are more than those for the conventional ones. These would permit longer filter runs and consequent reduced wash water requirements. The possibility of "break through" resulting in increased concentration of suspended solids in the effluent in filters with rate controllers is avoided in this system.

7.6.9 UP-FLOW FILTERS

In up-flow filtration, the water is passed under pressure in an upward direction through the coarser medium followed by finer medium. Thus larger size suspended solid particles are first retained in the larger interstices of the lower part of bed and as the water percolates upwards, it receives a progressive polishing until it emerges in a fully filtered condition at top of the filter bed. Thus the entire depth of media is made effective in removal of suspended solids and as a result low head loss and longer filter runs could be expected. Besides, many other advantages are claimed for up-flow filtration such as elimination of the rate controller and absence of negative head. Unfiltratered water can be used for washing filter since the first few minutes of flow through the filter after washing has to be necessarily run to waste. Filter depths as low as 0.6 m and as high as 1.5 m have been successfully used. Although wash water rate and consumption are greater per wash cycle than the conventional filter, wash water used as a percentage of finished water is much less because of low loss of head and long filter runs. But initially compressed air scouring is desirable to dislodge the impurities collected in the lower portions of the bed. The only disadvantage is fluidization of the top fine layers of the sand bed which results in the deterioration of the filtrate quality. Complete bed fluidization occurs when the headloss equals the depth of bed. Control of headloss is much more significant than the upward velocity through the filter. It is desirable that the hydraulic gradient through the upflow sand bed is restricted to 0.6.

7.6.10 GRID OR IMMEDIUM TYPE FILTERS

The problem of bed fluidization in an upflow filter is eliminated in this type by providing a 'grid'. The grid is a system of parallel vertical plates placed within the bed a few centimeters below the top of the medium. This grid provides sufficient resistance to prevent expansion of the bed and breakthrough or channelling at relatively higher rates of filtration. The exact mechanism of how this plate grid restrains sand expansion has not been proved but it is believed that the upward flow of water causes formation of inverted arches of sand which bridges the gap between the adjacent vertical plates. Being in compression, these arches are strong enough to resist the upward force of liquid being filtered ending to break through the bed, thus minimising fluidization of sand. Generally the grid spacing is 100 150 times the size of fine sand at the top of the bed. They operate at the rate of 9-12 $m^3/m^2/$ hr and deliver four times the filtered water per run of down flow filter of the same size. Recent researches indicate that higher rates of 15-30 $m^3/m^2/hr$ could be employed. The operating rate, however, depends on quality of water to be filtered and the effluent quality desired.

7.6.11 Bi-flow Filters

The possibility of fluidization of the finest sand layers in up-flow filtration is solved in this type by placing the effluent collecting pipe in the upper layers of sand bed and filtering simultaneously from below through the bulk of the media and from above through top layers of sand. With the same hydraulic head applied on top and bottom of the filter, the headloss across both the upper and lower beds to the centres of effluent pipe is the same and hence the effluent pipe is hydraulically balanced. The downward hydraulic gradient on the top portion prevents fluidization. The inflow filter's use sand of depth 1. 5 to 1.65 m, the filtrate collection system being placed 0.45 to 0.60 m below the top sand. Total filtration rate is 18 $m^3/m^2/$ hr , downward rate being half to two-thirds of the upward rate. Backwash rate adopted is 54 to 66 $m^3/m^2/hr$ for a period of 6 minutes. Initial cost is estimated to be 15-30% less than conventional filters even though piping, valves and controls are more complicated.

7.6.12 Submerged Filters

These filters have no rate controllers. This method employs direct pumping to permit automatic adjustment of treatment plant input and high level pumping to meet the varying and continous demands of a large city. These filters operate under varying rates as demand varies and through the use of a slow moving butterfly valve, the filtration rate can be changed without deterioration of water quality. The butterfly valve closes slowly over a period of time usually 5 to 10 minutes. These filters have proved satisfactory on a plant scale.

7. 6.13 RADIAL FLOW FILTERS

In these filters, flow comes in radially and washing is done continuously. The filter medium is sand, contained in the annular space ofprise of a closed cylindrical shell. Chemically treated water enters into a central hollow column and permeates radially through sand and is collected through peripheral ducts and flows out of the shell. The dirty sand is continously drawn from the bottom and airlifted to a compartment at the top of the filter where it is washed, the sand sinking to the cylinder while the washed water carrying dirt escapes out through an overflow device.

7.6.14 Automatic Valveless Gravity Filters

These filters operate without butterfly valves, pilot mechanisms, rate controllers, gauges and air compressors. They have two compartments, the filtering section and wash water storage compartment. As the incoming water is admitted to the filter, a head gets built upon the top of the sand and causes the water level to rise in the backwash pipe. When the water level reaches the top of the loop, usually designed with a 2 meters differential, syphon action is started and backwashing begins at the required rate of 30 to 42 $m^3/m^2/$ hr . Wash water flows from the storage tank up through the sand bed and is discharged through the backwash pipe. A syphon breaker ends the washcycle. The filter washes itself automatically, at the proper time at a given loss of head, without any mechanical instrument or operating tables. There is no maintenance from a mechanical standpoint of view. These filters are useful for low turbidity waters and for small installations.

7.7 DISPOSAL OF WASTES FROM WATER TREATMENT PROCESSES

Disposal of wastes from the water treatment plants has become increasingly important with the availability of technology and the need for protection of the environment. Treatment of waste solids adds to the cost of construction and operation of treatment plants.

Wastes from water treatment plants comprise of :

(a) sludge from sedimentation of particulate matter in raw water, flocculated and precipitated material resulting from chemical coagulation, or residuals of excess chemcial dosage, plankton etc.;

(b) wastes from rinsing and backwashing of filter media containing debris, chemical precipitates, strainings of organic debris and plankton and residuals of excess chemical dosage etc.; and

(c) wastes from regeneration processes of ion exchange softening treatment plant containing cations of calcium, magnesium and unused sodium and anions of chlorides and sulphates originally present in the regenerant.

7.7.1 DISPOSAL METHODS

In continuous sludge removal, the feasibility of discharging of water treatment plant sludge to existing sewers nearby should be considered. For lime softening plant sludge, the reclamation by calcining and reuse can be explored [8.4.2.1 (a) (3)]. Sludge from clarification units using iron and aluminium coagulants can be dewatered by vacuum filtration using lime as the conditioner, to a cake that can conveniently be trucked for landfill. The material will be still greasy and sticky. Recovery of alum from sludge by treatment with sulphuric acid offers possibilities of reducing the quantity of sludge to be handled. Sand drying beds are an acceptable method for dewatering certain types of sludge from settling tanks or clarifiers for further disposal by landfill. Simple lagooning of sludge does bring about a reduction in the bulk of the sludge to be handled and further disposal as landfill is necessary. Backwash water from filters can sometimes be recycled back to the plant inlet which can possibly improve settling and filtration. Reclamation of backwash water from filters can be adopted in areas of water scarcity. Simultaneously this reduces the disposal problem of the waste.

7.8 PERFORMANCE CAPABILITIES

7.8.1 SLOW SAND FILTERS

The following standards of performance for slow sand filters are recommended:

(a) The filtrate should be clear with a turbidity of 1NTU or less.

(b) The filtrate should be free from color (3 or less on the cobalt scale).

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(c) When the raw water turbidity does not exceed 30 NTU, the filter runs should normally be not less than 6 to 8 weeks, with the filter head not exceeding 0.6 m.

(d) The initial loss of head should not normally exceed 5 cm. A higher head will indicate that the entire sand bed needs overhauling.

7.8.2 Rapid Sand Filters

For rapid sand filters performance standards may be based on the following criteria:

(a) The filtrate should be clear with the turbidity of 1NTU or less.

(b) The filtrate should be free from colour (with 3 or less on the cobalt scale).

(c) The filter runs should normally be not less than 24 hours with a loss of head not exceeding 2 m.

(d) For an efficient filter, the wash water consumption should not exceed 2 per cent of the quantity filtered in between washing.

CHAPTER 8
DISINFECTION

8.1 INTRODUCTION

Water Treatment processes such as storage, coagulation, flocculation, sedimentation, filtration, aeration and water softening are specifically designed to produce waters that are aesthetically acceptable and economical to use. Though these physico-chemical processes assist in removal or killing of microorganisms to varying degree, these cannot be relied upon to provide safe water. For utmost safety of water for drinking purposes, disinfection of water has to be done for killing of disease producing organisms. Bacteria, viruses and amoebic cysts constitute the three main types of human enteric pathogens and effective disinfection is aimed at destruction or inactivation of these and other pathogens such as helminths responsible for water-borne diseases. The need for disinfection in ensuring protection against transmission of water-borne diseases cannot be overemphasized and its inclusion as one of the water treatment processes is considered necessary.

Historically, boiling of water or use of copper and silver vessels for storing water which effect some measure of disinfection have been employed in this country and elsewhere. Broadly, modern disinfection processes include use of :

(i) Physical methods such as thermal treatment and ultrasonic waves.

(ii) Chemicals including oxidizing chemicals such as Chlorine and its compounds, Bromine, Iodine, Potassium Permanganate, Ozone and metals like Silver

(iii) Radiation.

Disinfection and sterilization are different processes. While disinfection aims at selective destruction of disease producing organisms, sterilization is employed to completely destroy or inactivate all micro-organisms including bacteria, amoebic cysts, viruses, algae and spores.

8.2 CRITERIA FOR A GOOD DISINFECTANT

For a chemical or an agent to be potentially useful as a disinfectant in water supplies, it has to satisfy the following criteria:

(a) Be capable of destroying the pathogenic organisms present, within the contact time available and not unduly influenced by the range of physical and chemical properties of water encountered particularly temperature, pH and mineral constituents;

(b) Should not leave products of reaction which render the water toxic or impart colour or otherwise make it unpotable;

(c) Possess the property of leaving residual concentrations to deal with possible recontamination;

(d) Be amenable to detection by practical, rapid and simple analytical techniques in the small concentration ranges to permit the control of disinfection process.

8.3 MECHANISMS OF DISINFECTION

The mechanism of killing the pathogens depends largely on the nature of the disinfectant and on the type of microorganisms. In general four mechanisms are proposed to explain the destruction or inactivation of organisms.

(i) Damage to cell wall.

(ii) Alteration of cell permeability.

(iii) Changing the colloidal nature of the cell protoplasm.

(iv) Inactivation of critical enzyme systems responsible for metabolic activities.

Damage to cell wall leads to cell lysis and death. Alteration of cell permeability refers to the destruction of selective permeability of cytoplasmic membrane and causes outflow from the cells of such vital nutrients, as nitrogen and phosphorus. Denaturization of cell proteins by acids and bases leads to destruction of cells. Inactivation of critical enzyme activity vital for cell growth and survival is normally brought about by oxidizing chemicals.

Chemical disinfection normally proceeds in at least two steps:

(i) Penetration of the disinfectant through the cell wall and

(ii) Reaction with enzymes within the cell.

8.4 FACTORS AFFECTING EFFICIENCY OF DISINFECTION

The efficiency of chemical disinfection is influenced by the following factors:

(a) Type, condition, concentration and distribution of organisms to be destroyed.

(b) Type and concentration of disinfectant.

(c) Chemical and physical characteristics of water to be treated.

(d) Contact time available for disinfection.

(e) Temperature of water.

8.4.1 TYPE, CONDITION AND CONCENTRATION OF ORGANISMS TO BE DESTROYED

The disinfectant has to diffuse through the cell wall before it can react with the enzyme systems. Since the different types of organisms have different cell structures and different enzyme systems, the action of the disinfectant must necessarily vary. Among intestinal organisms, pathogens are less resistant than the coliform group and hence the latter can serve as a convenient index of the efficiency of disinfection.

Viruses appear to be more resistant than bacteria and require longer periods of contact as well as higher concentration of disinfectant. Spores are relatively resistant but fortunately are not of such significance as pathogens. Cysts are extremely resistant. The condition in which the organisms occur may also affect the efficiency of disinfection. Thus, when the bacteria are clumped together, the cell inside the clump may be protected against the action of disinfectant. The density of the organisms affects the efficiency only when the number is so high that there is a deficiency of available disinfectant. Such a condition may occur in disinfection of sewage but is not usual in water works practice.

8.4.2 Type and Concentration of Disinfectant

The efficiency of disinfection will obviously depend on the nature of the disinfectant. The added chemical undergoes several transformations so that the disinfecting action is really exerted by the end products of reaction. The course of these reactions is largely influenced by the character of the water and its constituents. These reactions that may occur under different conditions will determine the type and proportion of the active disinfectants. Higher the concentration of a chemical disinfectant, the higher is the destruction of organisms.

8.4.3 Chemical and Physical Characteristics of Water to be Treated

Organic matter and certain oxidising constituents in water reduce the availability of the active products for disinfection. Embedded organisms in suspended materials in water may be sheltered from the action of disinfectant.

8.4.4 Time of Contact Available for Disinfection

The destruction of organisms increases with contact time available for disinfection. In practice, the contact period is limited by the design of the plant and is usually not less than 30 minutes.

Adequate period of contact is available in most plants because the chlorinated water has a considerable detention in the clear water reservoirs before it is supplied. However, in small plants where such storage is not provided, the contact period is determined by the time taken for the water to flow from the point of application of chlorine to the point of drawal of water by the first consumer. If the minimum contact time is not available the dose of disinfectant should be suitably increased.

8.4.5 Temperature of the Water

Rates of chemical reactions are speeded up as the temperature of the reaction is increased. The higher the temperature, the more rapid is the destruction of organisms.

8.5 MATHEMATICAL RELATIONSHIPS GOVERNING DISINFECTION VARIABLES

The kinetics of disinfection is affected by several variables as enunciated in section 8.4. The effect of some of these disinfection variables can be quantified by empirical mathematical relationships. Under ideal conditions, three main variables alter the rates of disinfection, namely (i) the time of contact, (ii) the concentration of the disinfectant and (iii) the temperature of water.

8.5.1 CONTACT TIME

Contact time is an important variable affecting the rate of destruction of organisms. Generally speaking, under ideal conditions and at constant temperature. The number of organisms (Nt) surviving after a period of time t is related to the initial number (No) by Chick's law

$$\log \frac{N_0}{N_t} = k.t \quad (8.1)$$

Where k is constant with dimension (T^{-1})

Departures from Chick's law are not uncommon. Rates of kill have been experimentally observed to increase with time in some cases and decrease with time in other cases. To account for these departures from Chick's law the following modified equation has been suggested:

$$\log \frac{N_0}{N_t} = k.t^m \quad (8.2)$$

Where m is a constant. If m is less than 1, rate of kill decreases with time and if m is greater than 1, the rate of kill increases with time. Laboratory analysis and subsequent interpretation of data may provide useful information for design purposes.

8.5.2. CONCENTRATION OF DISINFECTANT

Rate of disinfection is affected, within limits, by changes in concentration of disinfectant. The relationship between disinfectant concentration and time required for killing a desired percentage of organisms is generally expressed by the following equation:

$$C^n \times t_p = \text{Constant} \quad (8.3)$$

Where C is the concentration of disinfectant, n is a coefficient of dilution and t is the time required for a constant percentage kill of the organisms. Values of n greater than one indicate rapid decrease in the efficiency of disinfectant as its concentration is reduced, if n is less than 1, contact time is more important than concentration and for n equal to 1, both affect the efficiency of disinfection to the same extent. Concentration-time relationship for HOCl at 0 – 6 °C resulting in 99% kill for E.Coli and several different viruses are:

$$\begin{aligned} C^{0.86} \times t_{99} &= 0.098 \text{ for adenovirus 3;} \\ &= 0.24 \text{ for E-Coli} \\ &= 1.2 \text{ for poliomyletis virus} \\ &= 6.3 \text{ for coxsackie virus } A_2 \end{aligned}$$

Since value of n is less than 1, changes in contact time have more effect on killing of organisms than corresponding changes in concentration of HOCl. Further these equations also indicate that some viruses (e.g. Coxsackie Virus A_2 are more resistant to killing by HOCl than commonly used indicator organisms, E-Coli.

8.5.3 Temperature Of Water

The effect of temperature on rate of kill is usually expressed by the Vant Hoff-Arrhenius relationship assuming that the rate of disinfection is controlled either by the rate of diffusion of disinfectant through the cell wall or by rate of chemical reaction with cellular enzymes:

$$Log\,\frac{t_1}{t_2} = \frac{E(T_2 - T_1)}{2.303RT_1T_2} \tag{8.4}$$

Where t_1,t_2 are time for given percentage of kill at temperatures T_1 and T_2 in °K, E is the activation energy (in J/mol or cals/ mol) and R is the gas constant equal to 8.314 J/mol.°K or 1.99 cals/mol.°K. Typical values of E at pH 7.0 for aqueous chlorine and chloramines are 34,332 and 50,242 J/mol respectively. As the pH increases, value of E also increases. This type of relationship indicates that at lower temperature of water (e.g. in winter season) the time required for achieving the same percentage of kill for the same concentration of disinfectant would be higher than those for higher temperature (e.g. in summer months). If time of contact cannot be changed due to design constraints, doses of disinfectants will have to be changed to account for changes in temperature to achieve same percentage of kill.

8. 6 CHLORINATION

8.6.1 Chlorine and its Properties

Chlorine is an element, having the symbol Cl with an atomic weight of 35.45, melting point -101.5 ^{0}C and boiling point 34.5^{0}C. Gaseous chlorine is greenish yellow in colour and is approximately 2.5 times heavier than air. Under pressure, it is a liquid with an amber colour, oily nature and approximately 15 times as heavy as water. Liquefaction of chlorine gas is accomplished by drying, cleaning and compressing the gas to 35 kg/cm^2. During this process, the resultant liquid chlorine is separated from noncondensable gases. The temperature of the liquid chlorine in the container influences the internal pressure of the chlorine gas and hence its flow from the container. Liquid chlorine must be vaporised in order to be withdrawn as gas and this tends to reduce its temperature and thereby its vapour pressure. At too high discharge rates, the liquid will be cooled excessively resulting in the formation of frost on the outside of the container. While dry chlorine is non-corrosive, moist chlorine is highly corrosive. Chlorine gas is harmful to human beings since it is a powerful irritant to lungs and eyes. Fatal concentrations are generally avoided as its irritant nature is recognisable at much lower concentrations, the odour threshold being 3.5 ppm by volume. The safety limit for a working environment permits the maximum allowable concentration of chlorine in air of 1 ppm by volume for an exposure period of 8 hours.

8.6.2. CHLORINE-WATER-REACTIONS

8.6.2.1 Free Available Chlorine

Chlorine reacts with water to from hypochlorous acid (HOCl) and Hydrochloric acid (HCl) according to the equation:

$$Cl_2 + H_20 \rightleftharpoons HOCl + H^+ + Cl^-$$

This hydrolysis reaction is reversible. The hypochlorous acid dissociates into hydrogen ions (H^+) and hypochlorite ions (OCl^-) according to the equation:

$$HOCl \rightleftharpoons H^+ + OCl^-$$

This reaction is also reversible. Free available chlorine may be defined as the chlorine existing in water as hypochlorous acid and hypochlorite ions. The undissociated HOCl is about 80 to 100 times more potent as a disinfectant than the OCl ion.

Both the above reactions are dependent upon the pH of the water. When the pH value of the chlorinated water is above 3, which is normally the case, the hydrolysis reaction is almost complete and the chlorine exists entirely in the form of HOCl The influence of pH on the disinfectant action, therefore is governed by the second reaction as waters with pH value below 3 are very rare. From a consideration of the second equation, it is evident that as the pH increases, more and more HOCl dissociates to form OCl^- ion. At pH values of 5.5 and below it is practically 100% unionised HOCl while above pH 9.5, it is all OCl^- ions. Between pH 6.0 to 8.0, there occurs a very sharp change from undissociated to completely dissociated hypochlorous acid with 96% to 10% of HOCl, with equal amounts of HOCl and OCl^- being present at pH 7.5 (Fig. 8.1). The addition of chlorine does not produce any significant change in the pH of the natural waters because of their buffering capacity.

8.6.2.2 Combined Available Chlorine

The free chlorine can react with compounds such as ammonia, proteins, amino acids and phenol that may be present in water to form chloramines and chloro-derivatives which constitute the combined chlorine. This combined available chlorine possesses some disinfecting properties though to a much lower degree than the free available chlorine. Theoretically some free available chlorine can exit along with combined available chlorine since these reactions do not go to 100% completion. The reactions with ammonia are:

$$Cl_2 + H_2O \rightleftharpoons HOCl + HCl$$

$$NH_3 + HOCl \rightleftharpoons NH_2Cl + H_2O$$

(Monochloramine)

$$NH_2Cl + HOCl \rightleftharpoons NHCl_2 + H_2O$$

(Dichloramine)

$$NHCl_2 + HOCl \rightleftharpoons NCl_3 + H_2O$$

(Trichloramine or Nitrogen chloride)

The monochloramine (NH_2Cl) and the dichloramine ($NHCl_2$) have disinfectant properties, though twenty five times less than that of free chlorine, while the trichloramine has no disinfectant properties at all. The pH of the water generally determines the ratio between the amount of mono and dichloramines formed which have nearly equal bactericidal powers. Below pH 4.4., trichloramine is found. Between pH 4.4 to 5.5, only dichloramine exists and in the range of 5.5 - 8.4, both mono- and dichloramines prevail in a ratio fixed by the pH. At pH 7.0, equal quantities of mono and di-compounds and above pH 8.4 only mono-chloramines are noticed.

8.6.2.3 Chlorine Demand

Chlorine and chlorine compounds by virtue of their oxidizing power can be consumed by a variety of inorganic and organic materials present in water before any disinfection is achieved. It is, therefore, essential to provide sufficient time and dose of chlorine to satisfy the various chemical reactions and leave some amount of unreacted chlorine as residual either in the form of free or combined chlorine adequate for killing the pathogenic organisms.

The difference between the amount of chlorine added to water and the amount of residual chlorine after a specified contact period is defined as the chlorine demand. The chlorine demand of any given water varies with the amount of chlorine applied, the time of contact, pH, temperature, and type and quantity of residual desired.

8.6.2.4 Estimation Of Chlorine

The usual tests practised for estimating the residual chlorine in water are the orthotoulidine test (OT) and orthotoulidine arsenite test (OTA), the former used for total residual chlorine concentration and the latter for free available chlorine. When orthotoulidine reagent is added to water containing chlorine; a greenish yellow colour develops, the intensity of which is proportional to the amount of residual chlorine present. Soluble tablets of DPD (diethylphenylene-diamine) have also been used satisfactorily in place of orthotoulidine reagent.

O.T AND O.T.A. METHODS

The orthotoulidine test procedure does not overcome errors caused by the presence of nitrates, iron and manganese, all of which produce a yellow colour with orthotoulidine nor is it able to discriminate between "Free Chlorine" and "Combined Chlorine". The O.T.A. method permits these differentiations. The principle of the method is that chlorine either free or combined is destroyed on addition of sodium arsenite whereas the colour produced by the reaction of chlorine with orthotoulidine as well as the interfering agents is unaffected. The reaction of orthotoulidine with free chlorine is instantaneous while with combined chlorine it is very slow and does not begin until about 10 seconds. This property is used for distinguishing free from combined chlorine. The test is carried out as follows:

a) Take three tubes marked to hold 10 ml and label them 'A', 'B' and 'C'

b) To tube 'A' add 0.5 ml. of orthotoulidine solution. Then add 10 ml. of water sample and mix. Add 0.5 ml or 0.5% sodium arsenite ($NaAsO_2$) immediately. Mix and compare with standards as rapidly as possible. Record the result (A).

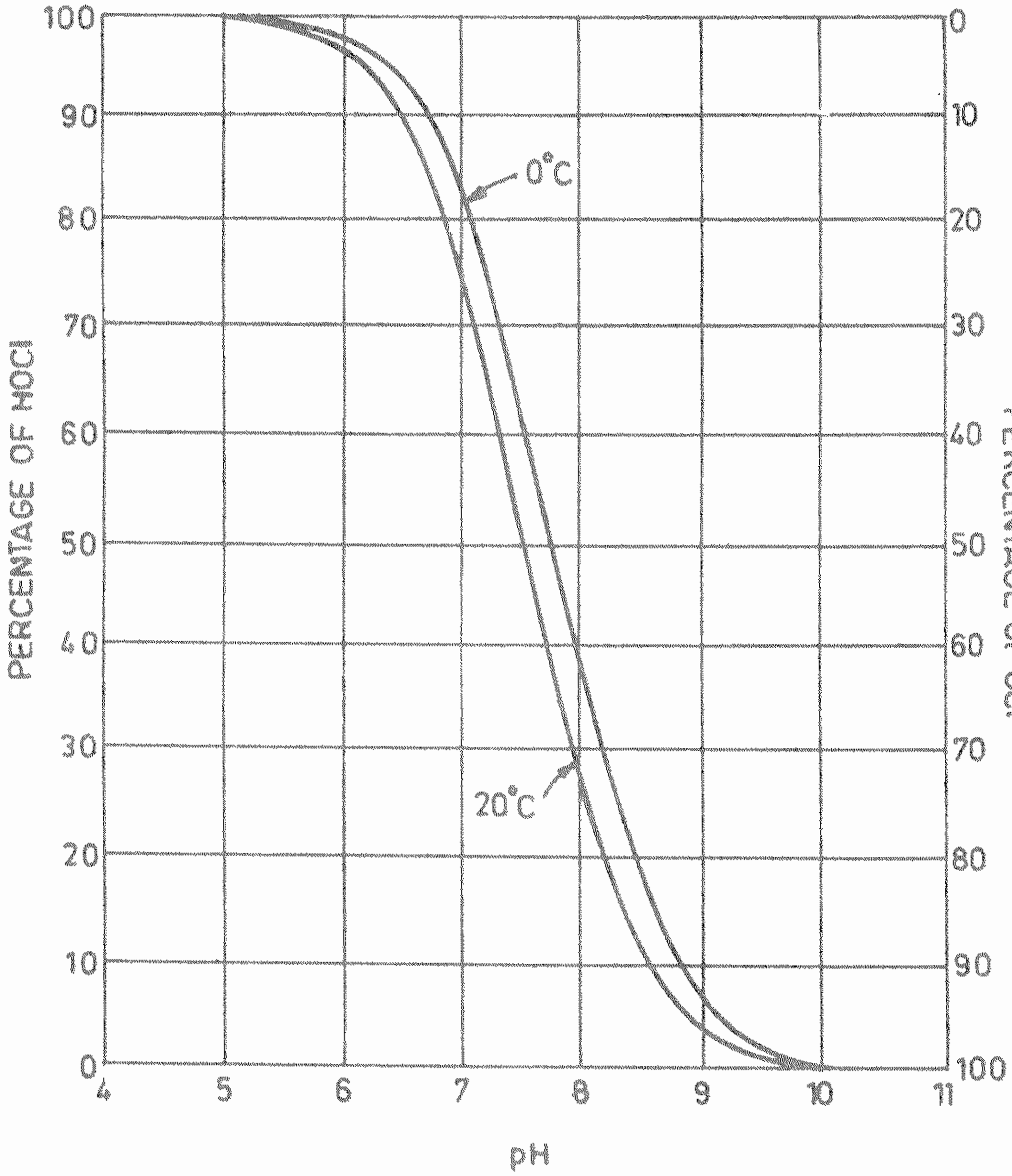

FIG. 8.1 RELATIVE DISTRIBUTIONS OF HOCl AND OCl^- AT DIFFERENT pH VALUES

c) To tube 'B' containing 0.5 ml of arsenite solution, add 10 ml of water sample. Mix quickly and immediately add 0.5 ml of orthotoulidine reagent. Mix and compare with colour standards as quickly as possible. Record the reading (Bl). Compare with colour standards again in exactly 5 minutes and record result (B2). B1 and B2 are due to interfering substances.

d) To tube 'C' add 0.5 ml of orthotoulidine reagent and then add 10 ml of the sample. Mix and allow to stand for exactly 5 minutes. Compare the colour with standards. Record reading (C).

e) Compute different values as follows:

- Total residual chlorine = (C - B2)
- Free residual chlorine = (A - B1)
- Combined residual chlorine = (C - B2) - (A - B1)

8.6.3 CHLORINATION PRACTICES

8.6.3.1 Free Residual And Combined Residual Chlorination

The type of available chlorine residual required and the characteristics of the water being treated determine the process of disinfection to be employed. All chlorination practice, irrespective of the point of application may be classified as free available residual chlorination (i.e. break point or superchlorination) or combined residual chlorination, depending on the nature of the chlorine residual formed. From practical viewpoint, both are not equally applicable to all water sources for disinfection or to improve the quality.

(a) Free available residual Chlorination

(1) Plain or simple chlorination:

This involves the application of chlorine to water as the only type of treatment to afford the necessary public health protection. Plain chlorination can be carried out in situation where:

(i) Turbidity and colour of the raw water is low, turbidity not exceeding 5 to 100 NTU;

(ii) Raw water is drawn from relatively unpolluted sources;

(iii) Water contains little organic matter and iron and manganese do not exceed 0.3 mg/l; or

(iv) Sufficient contact period between the point of chlorination and the consumer end is available.

(2) Super-Chlorination:

This is adopted in case of an emergency like a break down or in case of waters which are heavily polluted or fluctuate rapidly in quality. It can give excellent results in waters where:

(i) Plain chlorination produces taste and odour;

(ii) The water is coloured; or

(iii) Iron and manganese have to be oxidized

It may be resorted to on special occasions when available contact time is limited at the pre-chlorination stage. Super chlorination can effectively destroy the relatively resistant organisms such as viruses and amoebic cysts. The dose of chlorine may be as high as 10 to 15 mg/l with contact periods of 10 to 30 minutes. Excess chlorine will have to be dechlorinated.

(3) Dechlorination:

When superchlorination is employed, the water usually contains excess of free available chlorine which must be removed before it becomes acceptable to consumers. Dechlorination is the partial or complete reduction of undesirable excess chlorine in water by any chemical or physical treatment.

Prolonged storage and absorption on charcoal, granulated carbon and activated carbon are effective. Also reducing compounds like sulphur dioxide, sodium thiosulphate and sodium bisulphate are frequently used as dechlorinating agents. Dechlorination by sulphur doxide and its derivatives is feasible, rapid and precise. About one part of SO_2 (by weight) is required for each part of chlorine to be removed, the exact amount to be determined by the Stoichiometric relationship:

$$SO_2 + Cl_2 + 2H_2O \rightleftharpoons H_2SO_4 + 2HCl$$

(4) Breakpoint Chlorination:

As already explained in section 8.6.2.2., the addition of chlorine to ammonia in water produces chloramines which do not have the same efficiency as free chlorine.If the chlorine dose in this water is increased, a reduction in the residual chlorine occurs, due to the destruction of chlorine by the added chlorine. A few possible schemes are as below:

$$2NH_2Cl + HOCl \rightleftharpoons N_2 + 3HCl + H_2O$$

$$2NH_2Cl + 2NHCl_2 + HOCl \rightleftharpoons N_2O + N_2 + 7HCl$$

The end products do not represent any residual chlorine. This fall in residual chlorine will continue with further increase of chlorine dose and after a stage the residual chlorine begins to increase in proportion to the added dose of chlorine. This point at which the free residual chlorine appears after the entire combined chlorine residual has been completely destroyed is referred to as breakpoint and corresponding dosage is the breakpoint dosage. Breakpoint chlorination achieves the same results as superchlorination in a rational manner and can therefore be construed as controlled superchlorination.

(b) Combined Available Residual Chlorination

This method involves the application of chlorine to water to produce with natural or added ammonia, a combined available chlorine residual and to maintain the residual through part or all of a water treatment plant or distribution system. They are less effective disinfectants and oxidants than free available chlorine forms. The residual, however, will persist much longer than free available chlorine which has a tendency to diffuse and be lost. A minimum of 30 to 60 minutes contact time must be provided before delivery to the consumer. Depending upon the characteristics of water this can be accomplished as follows:

(i) application of chlorine only, if sufficient ammonia is present in the water;

(ii) addition of both chlorine and ammonia if it contains little ammonia; or

(iii) addition of ammonia if free available residual chlorine is already present in water.

In order to control chlorine-ammonia treatment effectively the optimum ratio of chlorine to ammonia has been found to be 3:1 or more to ensure the presence of an excess of ammonia.

This practice is useful after filtration for controlling algae and bacterial growths, for reducing red water troubles in distribution systems at dead ends and for providing and maintaining a stable residual throughout the distribution system.

(c) Points of Chlorination

The use of chlorine at various stages of water supply system right from raw water collection to the distribution network is a common practice and terms like pre-post and rechlorination have come into common usage depending upon the points at which chlorine is applied.

(i) Prechlorination

Prechlorination is the application of chlorine to water prior to any unit treatment process. The point of application as well as dosage will be determined by the objectives viz.; control of biological growths in raw water conduits, promotion of improved coagulation, prevention of mud ball and slime formation in filters, reduction of taste, odour and colour and minimizing the post chlorination dosage when dealing with heavily polluted water.

(ii) Postchlorination

Postchlorination is the application of chlorine to water before it enters the distribution system to maintain the required amount of free chlorine specified in 2.2.9 (c).

(iii) Rechlorination

When the distribution system is long and complex, it may be difficult to maintain the minimum chlorine residual of 0.2 mg/l at the farthest end. To achieve this if a very high dosage is applied at the postchlorination stage, it would, apart from being costly, make the water unpalatable, at the reaches close to the point of chlorination. The maintenance of the required residual, in such cases can be accomplished by a stagewise application of chlorine in the distribution system which is called rechlorination. Rechlorination is carried out in service reservoirs, booster pumping stations or at points where the mains supply to distribution zones.

8.6.4 CHLORINE RESIDUAL

Satisfactory disinfection is obtained by prechlorination to maintain 0.3 to 0.4 mg/l free available residual throughout treatment or 0.2 to 0.3 mg/l free available residual in the plant effluent at normal pH values. At higher pH of 8 to 9, at least 0.4 mg/l is required for complete bacterial kill with 10 minutes contact time. For 30 minutes contact time the dosage reduces to 0.2 to 0.3 mg/l.

The normal concentration of chlorine employed in the water works practice destroys causating organisms associated with typhoid fever, dysenteries and various gastrointestinal disorders. Cysts of E. histolytica, the causative organisms of amoebic dysentery are not destroyed but are inactivated at higher doses of 0.5 mg/l of the free residual chlorine.

Complete data are unavailable upon which to base recommendations of residual chlorine requirements to ensure destruction of water borne viruses. However, in practice 0.5 mg/l of free chlorine for one hour is sufficient to inactivate virus, even in water that was originally polluted and hence this may be adopted to make the water safe.

Where water supply is infested with nematodes, the supply should be prechlorinated for 6 hours to maintain a free available residual of 0.4 to 0.5 mg/l. This treatment attenuates most nematodes and renders them immobile which can be removed by settling processes.

8.7 APPLICATION OF CHLORINE

Chlorine can be applied to water by three methods:

(a) By the addition of a weak solution prepared from bleaching powder, HTH etc. for disinfecting small quantities of water.

(b) By the addition of a weak solution of chlorine prepared by electrolysing a solution of brine.

(c) By the addition of chlorine, either in gaseous form or in the form of a solution made by dissolving gaseous chlorine in a small auxiliary flow of water, the chlorine gas being obtained from pressurized chlorine cylinders.

The first method of chlorine application has the merits of simplicity, non requirement of electrical energy and relative safety in operation and handling as available chlorine is either in powder or solution form. However, the demerits include instability of bleaching powder, its hygroscopic nature and relatively low percentage of available chlorine (25-33%). To overcome these disadvantages, some variants with the basic chemical compound of Calcium Hypochlorite are recommended. These compounds possess a high chlorine content of about 65 – 70% and are stable, easily soluble and non-hygroscopic. However, these are expensive and require safety in handling

The second method of chlorine application requires the deployment of electrochlorinators to prepare a chlorine solution from electrolysis of water containing Sodium Chloride. An electrochlorinator essentially comprises of a direct current (DC) source for providing energy for electrolysis, an electrode pair installed in a container and hypochlorite solution storage and dispensation device. During electrolysis, chlorine is not evolved as a gas but is available in solution form as hypochlorite solution. This is the major advantage of this technique as transportation, storage and application of chlorine gas involves major safety considerations to avoid hazards and fatal accidents.

Electochlorinators are now commercially available in India in capacities ranging up to 1000 g/hr of chlorine using salt solution and up to 100 kg/hr of available chlorine using seawater. It is reported that power consumption may be less than 5 kWh/Kg of available chlorine for units with capacities greater than 500 g/hr and common salt requirements are of the order of 4-4.5 Kg/hr of available chlorine. However, no IS specifications are presently available for these electrochlorinators, and these electrochlorinators are based on emerging technology.

The third method of chlorine application is presently the common practice for medium to large public water supplies. However, it requires elaborate safety practices and use of chlorinators or chlorine evaporators and auxiliary equipments.

8.7.1 Safe Handling Practices

8.7.1.1 Storing Shipping Containers

Chlorine cylinders preferably should be stored upright and secured, and in such a manner as to permit ready access and removal. Ton containers should be stored horizontally, slightly elevated from ground or floor level and blocked to prevent rolling; a storage rack of I-beams is convenient. Ton containers should not be stacked or racked more than one high unless special provision is made for easy access and removal. Full and empty cylinders and ton containers should be segregated.

Storage areas should be clean, cool, well ventilated and protected from corrosive vapours and continuous dampness. Cylinders and ton containers stored indoors should be in a fire resistant building, away from heat sources (such as radiators, steam pipes etc.) flammable substances and other compressed gases. Subsurface storage areas should be avoided, especially for chlorine and sulphur dioxide. If natural ventilation is inadequate, storage and use areas should be equipped with suitable mechanical ventilators. Cylinders and ton containers stored outdoors should be shielded from direct sunlight and protected from accumulations of rain, ice and snow.

All storage, handling and use areas should be of such design that personnel can quickly escape in emergencies. It is generally desirable to provide atleast two means of exit. Doors should open out and lead to outside galleries or platforms, fire escapes, or other unobstructed areas.

8.7.1.2 Emptyping Containers

Chlorine cylinders deliver gas when in an upright position and liquid when in an inverted (or partially so) position. Ton containers in a horizontal position and with the two valves in a vertical line deliver gas from the upper valves and liquid from the lower valve.

To withdraw gas from a cylinder or ton container, the liquid chlorine must be vaporized. The flow rate is a function of the vaporization rate, which, in turn, is dependent on the rate of heat transfer to the liquid.

8.7.1.3 Connecting And Disconnecting Containers

The design and operation of facilities should be such as to minimize all hazards associated with connecting emptying and disconnecting chlorine containers. These operations should be performed in well-lighted places by authorised personnel equipped with gas masks or other suitable respiratory protection devices. Container valve protection goods should always be in place when the container is not in use. Valves should not be left open when operating personnel are not available to maintain proper surveillance of the operations.

Connections to valve outlets on cylinders and ton containers can be made by either a clamp and adapter or a union connector; the former is preferred. In making connections it should be ascertained that the outlet valve is closed before the outlet cap is removed. Gasket surfaces should be thoroughly inspected and cleaned and a new gasket of standard material should be used. Connections that do not fit should never be forced.

Cylinder and ton container valves should be slowly opened by using a special wrench, not more than 150 mm long, for this purpose. One complete turn of the stem in a counter-clockwise direction opens the valve sufficiently to permit maximum discharge. An auxiliary cylinder or ton container valve should be installed adjacent to the container valve between it and the chlorine feeder or gas header on manifold systems. Such a valve serves as an emergency shut off if the container valve should leak. Moreover, it prevents chlorine gas from escaping from the supply line when the container is removed from service. In the interests of safety, the ventilation system should be operating whenever containers are being placed into or removed from service and at all times in which an emergency exists or adjustments and repairs are being made.

Specifications and manufacturing of chlorine cylinders/containers, its transportation, handling, filling, possession and safety shall be governed as per Gas Cylinder Rules; 1981 of Central Government.

8.7.2 CHLORINATORS

A chlorinator is a device designed for feeding chlorine to a water supply. Its functions are:

(a) To regulate the flow of gas from the chlorine container at the desired rate of flow.

(b) To indicate the flow rate of gas feeding.

(c) To provide means of properly mixing the gas either with an auxiliary supply of water or with the main body of the liquid to be disinfected.

8.7.2.1 Types Of Feeders

Chlorinators are used for control and measurement of chlorine in the gaseous state and to supply chlorine as a gas or an aqueous chlorine solution. The principle of operation of these equipments depends on the regulation of flow by establishing a pressure relationship between the upstream and downstream sides of either a constant or a variable orifice in the chlorine flow gas line. Control of the feed rate is affected either by varying the pressure differential across a fixed orifice (variable differential unit) or by varying the size of orifice (constant differential unit).

These feeders are of two types, viz. (a) Pressure type and (b) Vacuum type.

(a) Pressure Type Gravity Feed Chlorinator

In the gravity feed chlorinator, dry gas at slight pressure is introduced into a cylindrical tower made of corrosion resistant material. Water is introduced at the top of tower (Fig. 8.2). As water flows down slowly, it gradually absorbs the chlorine. The resultant solution flows out of the tower by gravity to the point of application. Fig. 8.3 depicts a chlorinator with injector.

(b) Vacuum Type Chlorinator

Chlorine gas is maintained under vacuum throughout the metering apparatus. The opening of the chlorine inlet valve is governed directly by the vacuum induced in the system. This type of chlorination is most common because of safe operation (Fig. 8.4).

The Vacuum Type Chlorinator Consists of:

(i) A differential vacuum regulator flow meter;

(ii) A compensating vacuum regulating valve to maintain a constant down stream pressure;

(iii) Flow regulating pressure valve to maintain constant downstream pressure;

(iv) An injector or educator to create the necessary vacuum in which gas will be mixed with a small quantity of water prior to the point of application;

(v) A vacuum pressure breaker to prevent the possibility of water being drawn into the apparatus, to prevent build up of pressure;

(vi) Variable area flow meter to measure the amount of chlorine gas flowing.

8.7.3 ENGINEERING CONTROL OF HAZARDS

Careful consideration should be given to methods of handing chlorine shipping containers. Ceilings high enough for overhead hoists and floors of sufficient area for ease in handling mechanical equipment should be provided. Storage and use areas should be properly ventilated so as to minimize possible accidental reaction of chemicals, which is based on their previously noted characteristics. Piping systems should be as simple as possible with a minimum number of joints; they should be well supported, protected against temperature extremes and adequately sloped to allow proper drainage.

Long pipelines for liquid chlorine should be avoided. Sections of pipelines that can be isolated or shut off at both ends (such as by valves) must be provided with a suitable expansion chamber to avoid possible hydrostatic rupture due to pressure and volume increase accompanying high temperatures. Condensation or reliquefication can occur in chlorine gas lines that pass through areas where the temperature is below the

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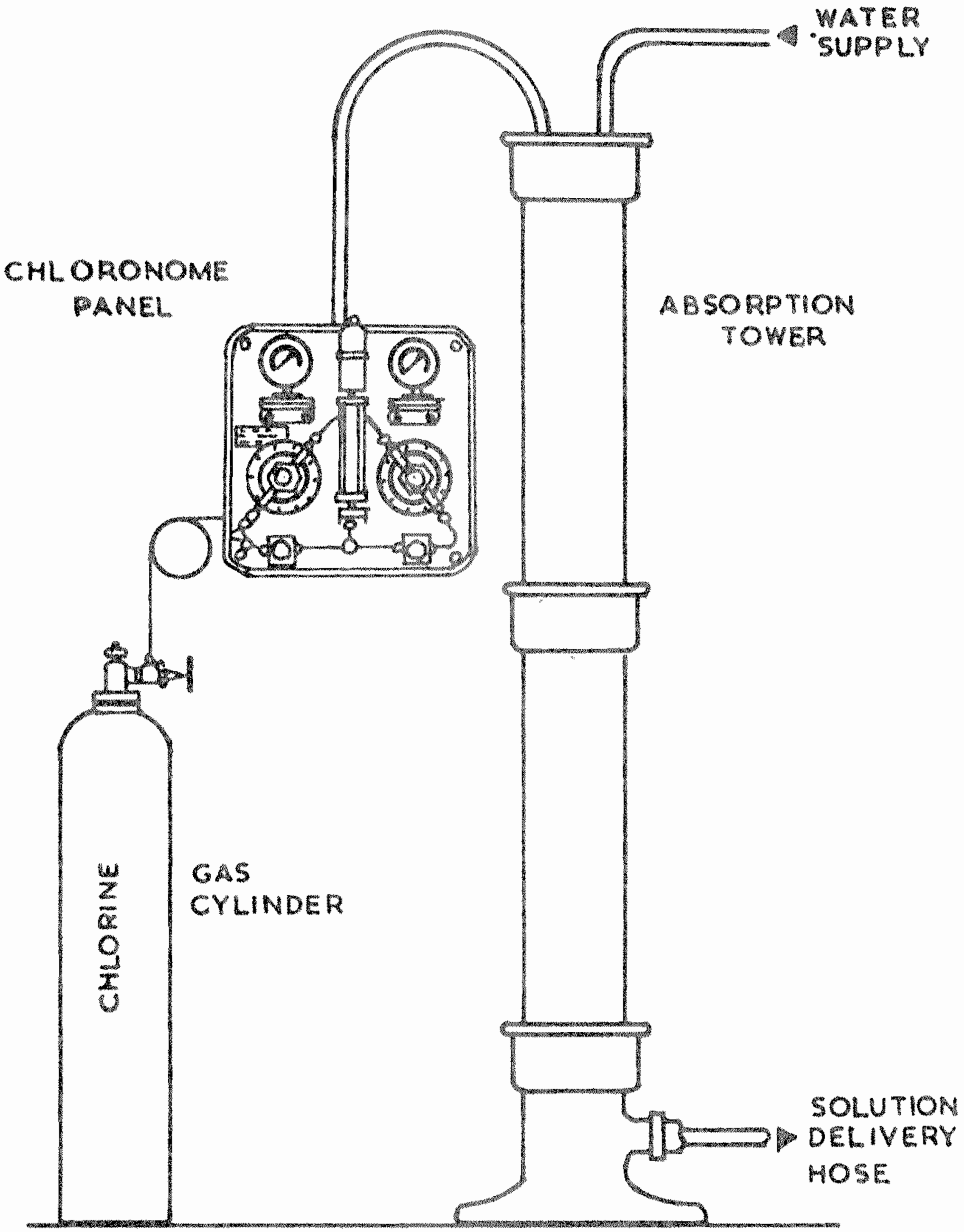

FIG. 8.2 CHLORINATOR WITH ABSORPTION TOWER

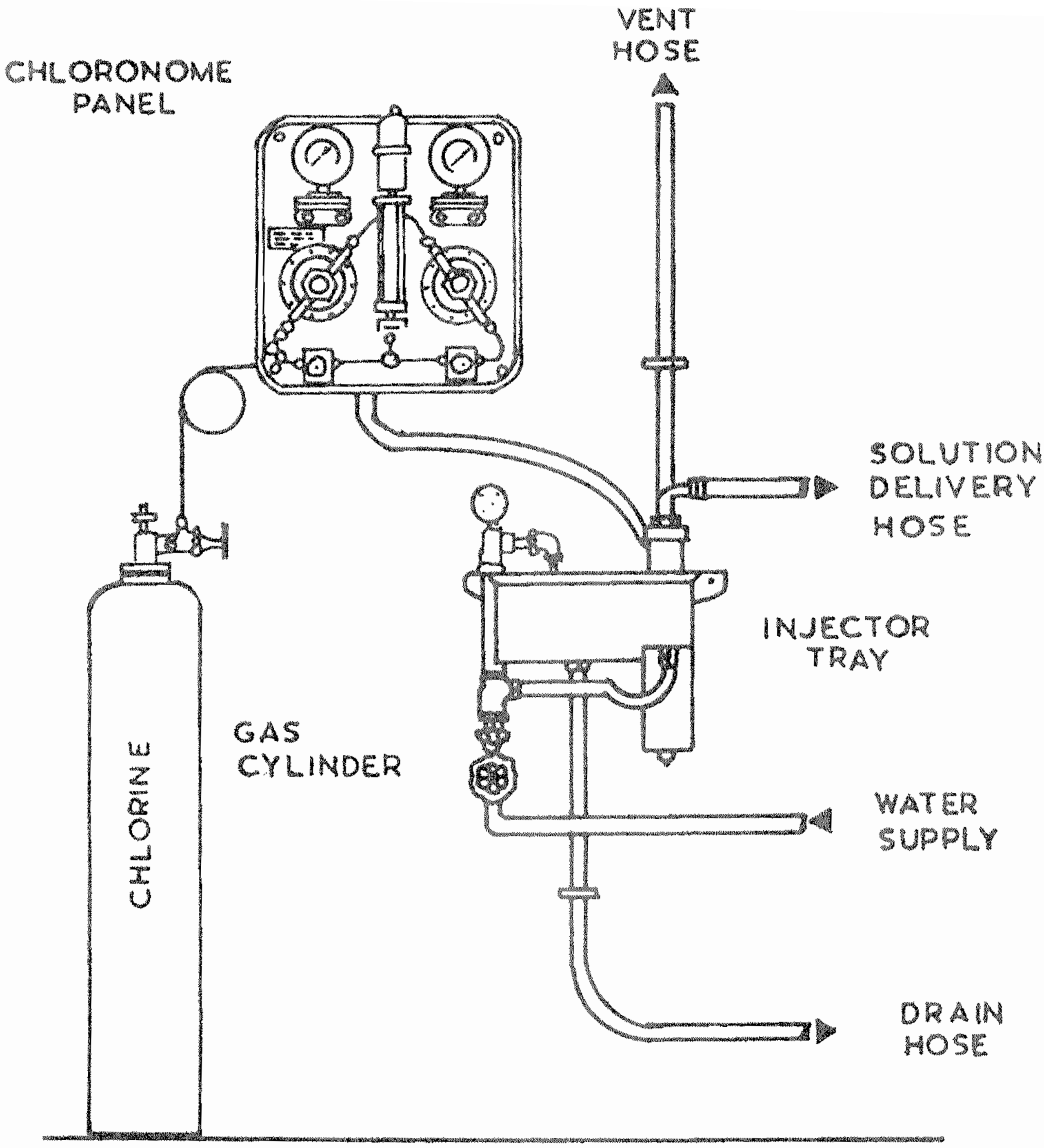

FIG. 8.3 CHLORINATOR WITH INJECTOR

temperature-pressure equilibrium indicated on the chlorine vapour pressure curve. It can be prevented by supplying properly controlled heat or by reducing the pressure. The temperature of chlorine containers and gas pipelines should be lower than the temperature of the chlorinator to prevent condensation; if chlorine gas condenses, the chlorine will be reliquified and result in erratic chlorinator operation. Chlorine pipelines removed from service for even very brief periods should be closed in a suitable manner to preclude the entrance of moisture, which will cause serious corrosion problems.

Equipment cleaning and repair should be performed under the direction of thoroughly trained personnel who are fully familiar with all the hazards and the safeguard necessary for the safe performance of the work. All precautions pertaining to education, protective equipment and health and fire hazards should be reviewed and understood. Repair to chlorine systems should not be undertaken while the system is in operation and while piping systems are in service. Chlorine pipelines and equipment should be first purged with dry air as a safeguard to health; this is especially important where cutting or welding operations are undertaken because iron and steel will ignite in presence of chlorine at about 235 ^{0}C. Immediate drying of a chlorine pipeline into which water or moisture has been introduced or which has been opened for repairs, is essential if corrosion is to be prevented.

8.7.3.1 Piping Systems

Moist chlorine unlike dry gas or liquid chlorine is highly corrosive.; Pipelines, valves and other fittings through which dry chlorine passes should be tightly closed when not in use to prevent absorption of the moisture from the air. Dry chlorine gas or liquid chlorine under pressure should be conveyed through extra heavy wrought iron or steel pipe or flexible annealed copper tubing tested for 35kg/cm^2 working pressure. The discharge line from the chlorine container should be flexible and slopping upwards, especially when chlorine is discharging in the liquid state. Long pipelines should be avoided. Hard rubber, silver or Platinum tubing is necessary for conveyance of moist chlorine gas or aqueous chlorine solutions at low pressure.

To prevent condensation of gas, piping systems and control equipment should be at the same or a higher temperature than the chlorine container. Chlorine gas lines are preferably located overhead rather than along with floor, to take advantage of the warmer ambient temperature. For liquid chlorine piping systems, conditions, which contribute to vaporization, could be avoided.

For pipes 35 mm diameter and smaller, connections may be either screwed, welded or flanged. If flanged, facing should be small tongue or grove. Gaskets should be made of antimony lead (with 2 to 3% antimony) or asbestos sheet. Rubber gaskets are not suitable. Screwed fittings should be of forged steel construction.

Pressure indicators in the system have Teflon diaphragms or silver foil protectors. Pressure reducing valves may be of bronze or silver plated body with silver diaphragm or of monel metal with a Teflon diaphragm.

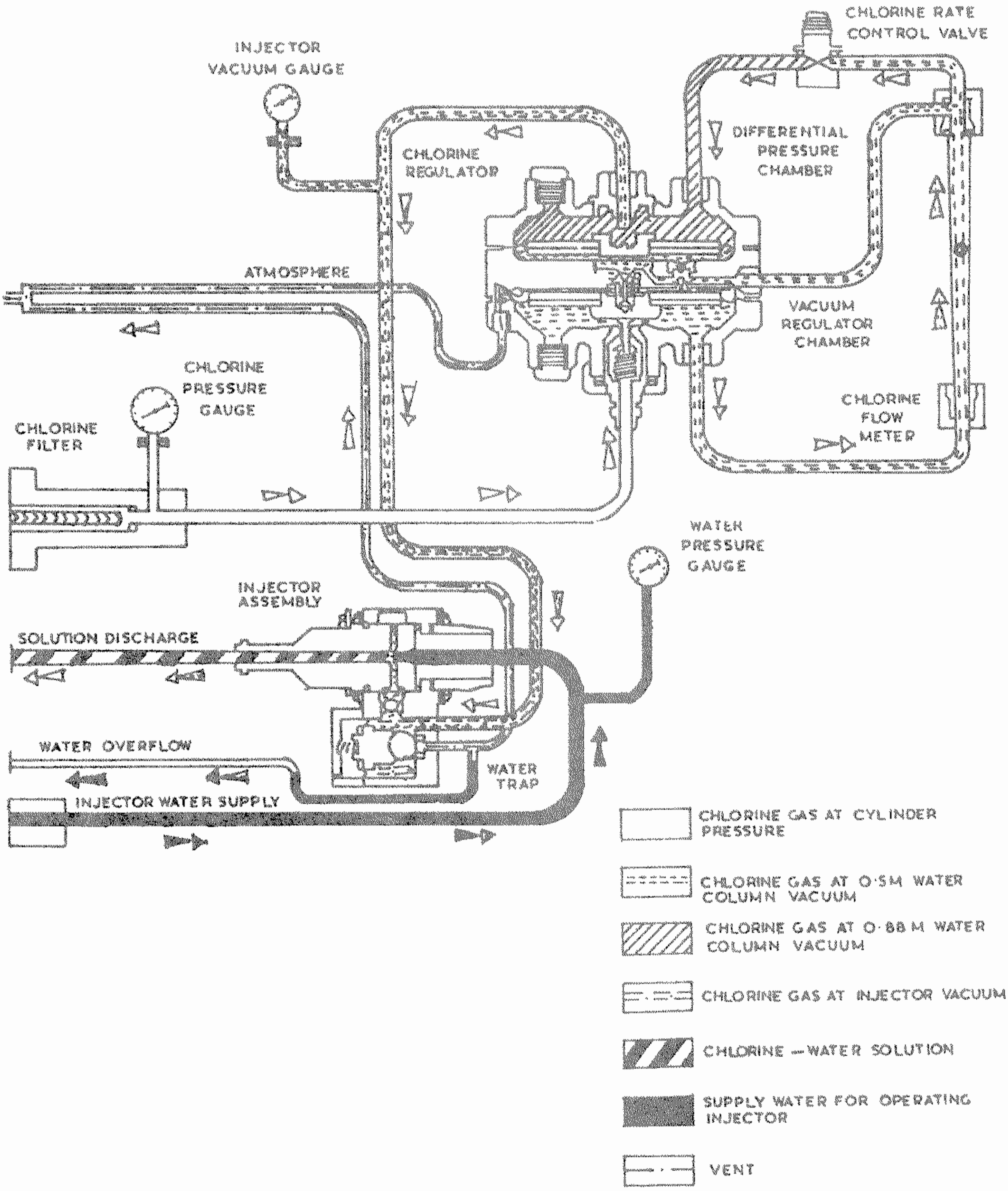

FIG. 8.4 MANUALLY OPERATED VACUUM TYPE CHLORINATOR

8.7.3.2 Number Of Cylinders Or Containers

Normal chlorine dosage required to disinfect water supplies not subject to significant pollution would not exceed 2mg/l. The actual chlorine dosage has to be determined on the basis of chlorine demand tests. The chlorine feed rate is then computed by dividing the expected maximum dosage of chlorine by maximum flow rate.

Total daily chlorine requirements can be estimated from the daily average consumption in a maximum day. The peak and the minimum rate requirements should be taken into consideration when designing a chlorine supply and feeder system and not merely the total daily requirements of chlorine.

When chlorine gas is withdrawn from a cylinder containing the liquefied gas, the pressure drops and the liquid 'boils' liberating more gas till the pressure is restored. This boiling absorbs heat continuously, then producing a cooling effect in the liquid region. If the withdrawal is continued, the liquid may freeze and no more gas will be evolved. It is therefore, essential to keep the atmosphere, round the containers in service warm and to ensure that there is not an abnormal rate of withdrawal from a single container with heavy demand of gas.

The recommended discharge rates are approximately 6.5 to 7.5 kg/hr, from a one ton container and 0.8 kg/hr from cylinders. Equipment should have sufficient capacity to exceed highest expected demand at any time and to provide continuous effective discharge under all prevailing hydraulic conditions. It is good practice to provide for duplicate equipment since disinfection process cannot be stopped at any time.

When the gas discharge rate from a single container will not meet the requirements, two or more can be connected to a manifold and discharge simultaneously. It is advisable not to couple more than four containers to a manifold. When discharging through a manifold, care must be taken that all the containers are at the same temperature, particularly when connecting a new cylinder to the manifold. Where more than 3 or 4 cylinders are used, the connections would be arranged in groups so that one complete group can be changed at a time. Storage of chlorine lasting a month or two should be provided. It is advisable to keep the full cylinders in the same room as the cylinders in service.

8.7.3.3 Maintenance

Every chlorinator is supplied with an instruction book that will include specific steps to follow in servicing. However, following are four areas most often associated with maintenance requirement and cause of trouble.

(a) Moisture

Moisture in chlorine is corrosive to ferrous and most nonferrous metals. Most chlorinators use plastic materials in the sections where gas is handled under vacuum. Metal parts or fittings, which are generally external to the chlorinator, are header valves, header lines and flexible connections. When any connection is broken, even for a short time, the openings should be plugged immediately to exclude moisture. Corrosion is internal and not evident upon external inspection until failure occurs. A good rule to follow is to exclude moisture from any part of the equipment that is normally exposed to dry chlorine only.

Corrosion products, primarily ferric chloride, are a major cause of chlorinator malfunctioning.

(b) Impurities in Chlorine

Even trace amount of impurities can cause problems if they accumulate. Two compounds are frequently found in chlorinators after continuous use. One, ferric chloride, may be present in the chlorine containers or may result when careless operation allows moisture to enter the system. This compound is recognizable as a dark brown, syrupy liquid and is soluble in water. After chlorinator parts are washed to remove impurity, they must be dried thoroughly before reassembly.

The other material, hexachloroethane or a similar compound, is classed as a volatile solid. It tends to deposit in gas lines at points of pressure-temperature drop. This material is not soluble in water, but can be dissolved in trichloroethane, a common industrial solvent.

(c) Flexible Connections

Flexible connection (comprising small diameter metal tubing), used to connect two cylinders or ton containers, need special attention. Because they are flexed every time a cylinder is changed, they are subject to metal fatigue. These connections should be changed once a year.

Each time a connection is made either to a chlorine container or to the chlorinator, a new gasket must be used.

(d) Gaskets

Elastomeric (flexible) materials used for gasket and O-rings generally become brittle in time. If a gasketed joint is not broken, the gasket may last for years. A regular programme of replacement is desirable but guidelines are difficult. Every recommended spare parts list includes spare gaskets. If swelling or hardening of a gasket is noted, it should be replaced. Distorted or hardened gaskets cannot be properly seated.

8.7.4 Chlorine Housing

The chlorine cylinders and feeders should be housed in an isolated room, easily accessible, close to the point of application and convenient for truck loading and safe container handling. The floor should be at least 15 cm above the surrounding ground and drainage should have at least two exit doors or building should have at least two exit doors for cross ventilation that allows an approximate air change in 10 minutes. For small installations, provision of ventilator opening at the bottom, one opposite the other is adequate.

Separate and reasonably gas tight enclosures opening to the outdoors should be provided for housing the chlorine feeding equipment in large installations and in buildings occupied by persons. These enclosures should be vented to the upper atmosphere and equipped with positive means of exhaust (near the floor level, at the center of the room or opposite to the entrance) capable of a complete air change within 2 to 4 minutes in an emergency. A satisfactory ventilation scheme involves a combination of fresh air and exhaust system, consisting of fans that force the fresh air into the enclosure through openings near the ceiling with exhaust fans to clear away any chlorine contaminated air near floor level.

The design of the exhaust system should not include the natural ventilation that may be availed.

8.7.5 CHLORINE EVAPORATORS

Chlorine vaporizers, better known as evaporators, are needed whenever conditions require the withdrawal of liquid chlorine from the containers. Typically, this action is necessary when the daily chlorine requirements exceed 100 to 1000 kg/24 hr and would require the manifolding of an excessive number of containers. Evaporators provide the heat necessary to vaporize or change the liquid chlorine to the gaseous state so that it may be handled in the normal fashion by the other components of the chlorination system.

As evaporator consists of a chlorine pressure vessel to which heat can be applied under controlled conditions, the source of heat may be electricity, steam or hot water with several different models and versions available from different manufacturers. The most commonly used models are electrically heated and have maximum rated capacities of 150 kg/hr in all cases the source of heat is thermostatically controlled to maintain constant temperature and ensure a superheated chlorine gas output.

The liquid absorbs heat from the water chamber through the wall of the pressure vessel until it reaches the vaporization temperature and boils, releasing chlorine gas. With the chlorination system in operation, gas is withdrawn from the pressure vessel, allowing more liquid to enter the system and continue the process. The gas leaving the vessel contacts the hot upper wall and causes the gas to be heated to a higher or superheated temperature. Baffles may be used to assist heat transfer.

As heat is absorbed by the chlorine, the water is cooled until it drops below the control thermostat setting and causes the electric immersion heater to be actuated. The heater remains on until the water reaches the upper limit of the thermostat. At this point, the heater is shut off and this cycle is continued as long as the evaporator is in use. As chlorine is vaporized, any impurities contained in the liquid are left behind, coating the inside surface of the pressure vessel. This coating acts as an insulator that inhibits heat transfer from the water to the chlorine and necessitates higher and higher waterbath temperature settings. Eventually, the pressure vessel will require cleaning as the evaporator will be unable to vaporize the desired amount of chlorine gas at the desired degree of superheat. The frequency of cleaning will vary from installation to installation and is a function of chlorine purity and the chlorine feed rate. The evaporator manufacturer's cleaning instructions should be followed closely for best results.

Typically, evaporators are supplied with various gauges and controls, in addition to the control thermostat, to permit simple, safe operation. Taken individually, they include the following :

- Control thermostat senses water temperature and turns heater on and off to control water bath temperature,
- Water-level gauge is a sight glass that permits the operator to observe the water-bath level.
- Chlorine bath-pressure gauge indicates chlorine gas pressure within the pressure vessel.
- Chlorine-gas-temperature gauge indicates temperature of the superheated chlorine gas leaving the pressure vessel.
- Water-temperature gauge indicates the temperature of the water bath,
- Low-water-temperature switch actuates a persist lower-water bath temperature, which normally indicates loss of the heater system. The switch may be used to actuate an alarm and automatically close the chlorine gas shut off valve to avoid the pulling of liquid chlorine into the gas system.
- High-water-temperature switch actuates a persist high-water bath temperature. Excessive temperature may be caused by failure of the heater controls, which allows the heater to remain on or by loss of water from the water bath. The switch may be used to actuate an alarm and turn off power to the electric heater.
- Low-water-level switch actuates a persist low-water bath level, caused by a failure of the water-supply system. Switches may be used to actuate an alarm and automatically open a water valve.
- Magnetic heater contactor acts in response to the control thermostat to energize or deenergize the electric immersion heater.
- Cathodic protection protects from corrosion all metal surface of the evaporator in contact with water.
- Vent alarm switch actuates whenever gas flow occurs in the vent line and indicates that the gas-pressure-relief valve has opened.
- Liquid chlorine expansion tank protects the liquid chlorine piping system from damage due to over pressure by offering a chamber into which the liquid can expand. This tank is strongly recommended for installation in any section of liquid piping which may be purposely or accidentally isolated by closing valves.

Some or all of the aforementioned controls, gauges and accessories may be supplied with an evaporator.

8.7.6 ANCILLARY EQUIPMENTS

8.7.6.1 Weighing Machines

Weighing scales are necessary to record the weight of chlorine used in 24 hours which would serve as a check of the daily consumption and also enable the cylinders to be changed when they get empty.

8.7.6.2 Personnel Protection Equipment

Severe exposure and potential health hazards exist wherever toxic compressed gas and other respiratory irritants are handled, or used. An approved gas mask should be provided for every employee involved with handling them. Additionally, suitable protective equipment for emergency use should be available outside rooms where hazardous materials are located near the entrance, away from areas of likely contamination. Such equipment might be provided in several locations at larger installations.

Canister type gas masks with a full face piece and specific or all purpose commodity canister should be used only for relatively short exposure periods and only if it has been clearly established that sufficient oxygen is present (not less than 16% in air) and that contamination does not exceed the allowable level (1% of chlorine). Canister masks might not be useful in emergencies since these criteria might not be readily ascertained, especially if suitable forced ventilation schemes are not provided. Regular replacement of overage canisters, even though unused, is recommended.

Self-contained breathing apparatus, with a full face piece and a cylinder of air or oxygen carried on the body or with a canister that produces oxygen chemically, is suitable for high contaminant concentrations and is the preferred means of respiratory protection. Protection is provided for a period that varies with the amount of air, oxygen or oxygen producing chemicals carried.

Respiratory protective equipment should be carefully maintained, inspected and cleaned after each use and at regular intervals. Defective or inoperable equipment is worse than none at all. All such equipment should be used and maintained in strict accord with the manufacturer's instructions. No person should enter contaminated areas unless attended by an observer who can rescue him in the event of respirator failure or other emergencies.

It is good practice to provide eye-protection devices (or masks with full face pieces) and other protective clothing for workers exposed to hazardous materials. Emergency showers, eye baths or other suitable water-flush systems should be provided in convenient locations for use by accidentally exposed personnel. Installation of an automatic chlorine leak detector with or without visible or audible alarm, should be considered.

8.7.6.3 Chlorine Detectors

Continuous monitoring of atmosphere in areas where chlorine is stored and fed is an important aspect of any safety programme. Instruments for this purpose are called chlorine detectors, which are not to be confused with the detectors used for measuring residual chlorine in water.

Concentrations are expressed as ppm by volume in air, not as parts per million parts by weight, as the expression is used to denote concentrations in water. For comparison, 1 ppm of chlorine by volume in air is equivalent to 3 mg/m^3 of air. The threshold of odour perception is about 3 ppm.

Two types of detectors are available. In type 1, the air to be sampled is directed to a rotating drum covered with a strip of sensitized paper. The paper is white and light is reflected from it to a photoelectric cell. The current from the cell is amplified and used to keep an electric relay open in an alarm circuit. If the air sample contains chlorine, the paper darkens, the light is absorbed, the current from the photoelectric cell drops below to that required to keep the relay open and thus the alarm circuit is energized.

In the second type, air from the point or points of sampling is drawn to the detector by an air pump through a filter and flow meter that indicates the sample flow rate. The air sample is directed to an electrochemical sensing cell, the electric output of which increases with the presence of chlorine. A meter movement is incorporated to indicate visually the strength of the chlorine in air and an adjustable switch is included to provide a contact closure for remote audible/visual alarms or an exhaust fan.

8.7.6.4 Automatic Changeover System

Increased emphasis on the need for uninterrupted chlorination has led to the use of automatic changeover systems, particularly at unmanned stations. The basic concept of these systems is to switch from a depleted source of chlorine to a stand by source automatically without the presence of an operator. Several methods have been used to accomplish this. One system consists of electrically operated chlorine shut off valves, actuated by a chlorine-pressure switch that senses the loss of chlorine pressure due to empty cylinders. Another system uses two pressure-reducing valves, each attached to its own source of chlorine and manifold on the downstream sides. The pressure settings of the two valves are adjusted so that the valves control at pressure approximately 5 psig apart. Since a pressure-reducing valve will not open until the downstream pressure is lower than its setting, the valve with the higher setting opens first, allowing gas to flow through the valve from its source. This process continues until the first source is depleted and the downstream pressure drops to the setting of the second valve, at which point it opens and chlorine flows from the standby source.

The recent development of small cylinder-mounted chlorinators has added more types of automatic changeover systems to the market place. It is not necessary to detail the operation of each, but merely to state that they meet the basic need of permitting continuous chlorine feed in a simple, inexpensive manner for even the smallest gas chlorination facilities.

8.7.7 Safety Considerations

(a) Only trained personnel should be permitted to handle chlorine cylinders and chlorinating equipment. They should be made aware of the hazards involved, the precautions to be observed and first aid to be rendered in emergencies. Rubber gloves, aprons and suitable gas masks should be provided. These should be housed in an easily accessible (unlocked) cupboard placed outside the chlorinator room. It is very important that the operating personnel are trained in the proper use of gas masks. A faulty gas mask is worse than none at all. Hence it is very important that these are tested frequently and the containers are changed at proper intervals.

(b) When a chlorine leak occurs, the mechanical ventilation system should be opened immediately before any person enters the chlorine room. It must be made a point that chlorine container valves are closed first before any investigation is started.

(c) Cylinders containing chlorine should be handled gently. They should not be bumped, dropped or rolled on the ground and no object should be allowed to strike them with force. The protective hoods over the valve should always be kept in place except when the cylinders are in use. Flames should never be applied to chlorine cylinders or their valves.

(d) Cylinders should not be stored in the open or in damp places. Empty cylinders should be stored away from full cylinders so that they do not get mixed up. It would be desirable to tag the empties as an additional precaution. Incidentally, this will ensure prompt return of used cylinders.

(e) In case the valve is found to be stuck, the cylinder should be immediately returned to the supplier. No attempt should be made to ease a stuck valve by hammer, as this is very dangerous.

(f) Only the spanners prescribed for use should be used as it is important not to put too much leverage on the valves.

(g) Cylinders as well as the chlorinators must be tested at the start and end of every shift period, for leaks, first by trying to detect the sharp irritating smell of chlorine, then by passing over each cylinder and around each valve and pipe connections a rod, with a small cotton-wool swab tied on the end, dipped in an aqueous solution of ammonia. Any leakage noticed anywhere must be attended immediately otherwise same is going to lead major trouble in the plant. If chlorine is present in the air, the swab will appear to 'smoke' due to the formation of white clouds of ammonium chloride. If the leak appears to be heavy, all persons not directly concerned should leave the area and the operator should put on his mask and make a thorough search for the leak. In tracing a leak, always work 'down stream' i.e. start at the cylinder and work down along the line of flow until the leak is found. It will save many valuable minutes over the practice of starting in the middle of the chlorinator and searching vaguely back and forth over the whole equipment.

(h) Water should never be applied to a chlorine leak to stop it as it will only make it. If the leak is in the chlorinator, the cylinder should be immediately shut off until the pressure has reduced. The joint or gasket should be repaired replacing with new packing, if necessary.

(i) Solvents such as petroleum, hydrocarbons or alcohols should not be used for cleaning parts which come in contact with chlorine. The safe solvents are chloroform and carbon tetrachloride. Grease should never be used where it can come in contact with chlorine as it forms a voluminous frothy substance on reaction with chlorine. Only special cements recommended by manufacturers should be used.

(j) No direct flame should be applied to a chlorine cylinder, when heating becomes necessary, as this is hazardous. A water bath controlled not to exceed 27° C should be used.

(k) Before disconnecting the flexible lids from containers to gas headers, the cylinder valves should be closed first and then the gas under pressure should be drawn from the header and flexible lids before the header valve is closed. The exhaust system should be turned on and operated while the cylinders are being disconnected or repairs being made.

8.7.8 Handling Emergencies

As soon as there is indication of a chlorine leak or other abnormal condition, corrective steps should be taken. Leaks never get better by themselves; they always get worse if not promptly and suitably repaired. Authorised trained personnel with suitable gas masks should investigate and all other persons should be kept away from the affected area. The ventilation system should be placed in operation immediately. Unconfined chlorine, being heavier than air, tends to lie close to ground levels (the characteristic must be kept in mind in designing chemical storage and use areas and appropriate natural or mechanical ventilation system). If leaks cannot be handled promptly, the chemical supplier or nearest office or plant of the producer should be called immediately for emergency assistance.

In case of fire, containers should be removed from the fire zone immediately. Portable tanks, tank cars, truck and barges should be disconnected and if possible, should be removed from the fire zone. If there are no leaks, water should be applied to a leaking chlorine container. Chlorine is only slightly soluble in water and the corrosive character of its reaction with water always will intensify the leak. In addition, the heat supplied by even cold water will increase the vaporization rate. Leaking chlorine containers similarly should not be thrown into a body of water because the leak will be aggravated and the container might float when still partially full, allowing uncontrolled gas evolution at the surface.

If a leak occurs in equipment or piping, the supply should be discontinued and the material under pressure at the leak should be disposed off. Leaks around container valve stems usually can be stopped by tightening the pack out or gland. If this action does not stop the leak, the container valve should be closed and material under pressure in the outlet piping should be disposed off. If the valve does not shut off tight, the outlet plug or cap should be applied. In the case of a leaking valve of a ton container, the container should be positioned so that the valves are in a vertical plane with the leaky valve on the top.

If one is confronted with other container leaks, one or more of the following procedures should be considered:

(i) Position cylinders or ton containers so that gas instead of liquid escapes. The containers may be insulated with sacks, earth etc to decrease absorption of heat and the discharge rate.

(ii) Apply appropriate emergency capping devices, if available.

(iii) Call the supplier or nearest producer for emergency assistance.

(iv) If practical, reduce pressure in the container by removing the gas to process or suitable disposal system. Caustic Soda, soda ash or other suitable alkali absorption system should be provided for disposing of chlorine from leaking cylinders and ton containers (100 kg of Cl_2 can be neutralized with 125 kg of caustic soda).

(v) In some cases it might be desirable and possible to move the container to an isolated spot where it will do the least harm.

8.7.9 Personnel Training

Safety in handling hazardous materials depends to a great extent, upon the effectiveness of employee education, proper safety instructions, intelligent supervision and the use of safe equipment. Training for both new and old employees should be conducted periodically to maintain a high degree of safety in handling procedures. Employees should be thoroughly informed of the hazards that may result from improper handling. They should be cautioned to prevent leaks and thoroughly instructed regarding proper action to take in case leaks do occur. Each employee should know what to do in an emergency and should be fully informed about first aid measures.

In addition, employee training should encompass the following:

(i) Instruction and periodic drill or quiz regarding the locations, purpose and use of emergency fire-fighting equipment, alarms and emergency crash shut-down equipment such as valves and switches.

(ii) Instruction and periodic drill or quiz regarding the locations, purpose and use of personnel protective equipment.

(iii) Instruction and periodic drill or quiz regarding the locations, purpose and use of safety showers, eye baths, bubblier drinking fountains and the closest source of water for use in emergencies.

(iv) Instruction and periodic drill or quiz of selected employees regarding the locations, purpose and use of respiratory first aid equipment.

(v) Instruction to avoid inhalation of toxic vapours and all direct contacts with corrosive liquids.

(vi) Instruction to report to the proper authority all leaks and equipment failures.

8.8 CHLORINE COMPOUNDS

Chlorine may also be applied in the form of compounds such as bleaching powder or as calcium or sodium hypochlorite which make the chlorine available when they come into contact with water. These are used for disinfection of small water supplies having capacities upto 0.5 mld.

(a) Bleaching Powder

Bleaching powder is a variable mixture of calcium hydroxide, calcium chloride and calcium hypochlorite. When it is mixed with water, the calcium hypochlorite decomposes into calcium chloride and chlorine. The action exerted by bleaching powder, is therefore, similar to that of gaseous chlorine in water. Bleaching powder is characterized by its content of available chlorine i.e. the chlorine which can be liberated by complete reaction with water. Commercial brands have an available chlorine of 20 to 30% i.e. 20 to 30 parts by weight of chlorine per 100 parts by weight of bleaching powder.

Bleaching powder is generally made into a thin slurry with the water and the supernatent, which contains the chlorine in solution, is applied to the water by a suitable feeding mechanism such as a float operated gravity box. In every installations, the solution may be applied through a dripfeed mechanism. Devices which can give constant feed can be easily fabricated. In the case of well supplies, bleaching powder solution may also be introduced at the suction side of the pump. An injector may be fitted on a bleed line on the pump discharge to suck the solution of the powder in proportion to the flow of water. A very simple method involving the use of porous earthenware cylinders which can be suspended in wells has been developed in the country. This method offers promise in the chlorination of rural water supplies where cost and technical skill have to be kept at a minimum.

Since bleaching powder contains only around 30% available chlorine, its use involves the extra expense of transporting and storing the inert material. The cost is further increased because the material is sold in nonreturnable drums which have no salvage value. Furthermore, bleaching powder is an unstable compound and loses its available chlorine on storage. All these considerations make its use uneconomical except in very small installations or for special cases such as disinfection of mains.

(b) Hypochlorites

The chemicals used are Sodium Hypochlorite and Calcium Hypochlorite. Specially fortified brands of Calcium Hypochlorite such as Perchloron and High Test Hypo (HTH) can have 60-70 per cent available chlorine. Calcium Hypochlorite can be fed either in the dry or solution form, while Sodium Hypochlorite is fed as solution. The solution form is usually preferred. Corrosion resistant materials such as ceramics, glass, plastic or special rubber should be used while handling hypochlorite solutions. Generally 1 to 2% chlorine solutions are prepared and fed directly through solution feeders. Usually constant head gravity devices with adjustable orifices are used to dose chlorine solution in the tanks. These can be fed through chemical proportioning pumps and can be injected under pressure into pressure pipe lines by venturi or orifice feeders.

(c) Chlorine Dioxide

Chlorine dioxide is an unstable gas. It is formed by reacting a strong solution of chlorine (7500 mg/l of Cl_2 at pH 3.5) with Sodium Chlorite.

$$Cl_2 + 2NaClO_2 \rightleftharpoons 2ClO_2 + 2NaCl$$

Theoretically ratio of Chlorine to Sodium Chlorite is 1: 2.6 . In practice, however, a large excess of chlorine is generally applied in order to avoid unreacted chlorite in the treated water. Chlorine dioxide is unstable and subject to explosion in gaseous form but aqueous solutions of the gas are stable and safe. It has been reported to be a good bactericide and its bactericidal efficiency is relatively unaffected by pH between 6 and 10. It is a good sporicide and a strong oxidant. It does not combine with ammonia and most organic impurities before oxidizing them. The common dosages of chlorine dioxide range from 0.2 to 0.3 mg/l.

Although chlorine dioxide is itself a disinfectant, the excess of chlorine used in its generation, rather than ClO_2 is generally counted upon to achieve disinfection. It can be effectively used for destruction of tastes and odours, particularly those which are caused by phenolic substances.

8.9 DISINFECTION METHODS OTHER THAN CHLORINATION

Chlorine and its compounds are most widely used for disinfection because they meet the requirements of a good disinfectant more fully than any other disinfectant. However, possible formation of carcinogenic byproducts by reaction of chlorine with some organics and enhancement of tastes and odours due to the reactions of chlorine with some water constituents have been reported.

Various other agents of disinfection are available and some of them such as ozone and ultraviolet rays are finding increasing usage in water treatment practice. Broadly, three main types of disinfectants are: (1) Physical agents including heat, (2) Chemical agents and (3) Radiations of various types such as Ultraviolet rays, Gamma rays and X-rays. Some of these disinfectants are discussed in subsequent sections.

8.9.1 HEAT

Boiling of water will disinfect it. This practice, however, cannot be used to disinfect municipal supplies for economical reasons. Boiling of water is applicable for disinfecting individual's drinking water in emergencies like accidental contamination of public water supply or during epidemic breakout. The thermal resistance of different microorganisms and viruses varies significantly with spores being upto 3,000,000 times more resistant than E. Coli, Viruses and bacteriophages. As no important water-borne disease is caused by spore forming bacteria or other heat resistant organisms, boiling of water can render the water safe for drinking purposes against diseases. Continuous-flow-water-pasteurises with flow rates of 1000 lph are also available.

8.9.2 CHEMICAL DISINFECTANTS

Chemical disinfectants are commonly grouped under following categories:

(i) Oxidizing chemicals including halogens, ozone and other oxidants such as potassium permanganate and hydrogen peroxide.

(ii) Metal ions.

(iii) Alkalies and Acids.

(iv) Surface active chemicals.

8.9.2.1 Halogens Other Than Chlorine

Halogens are oxidizing agents and include fluorine being the strongest and iodine the weakest oxidizing agents. However, disinfecting efficiency does not correlate directly with oxidizing capacity of a disinfectant. As fluorine can oxidize water, it cannot be used for disinfecting water.

Bromine is a heavy dark reddish-brown liquid which upon addition to water forms Hypobromous acid (HOBr), the dissociation of the acid resulting in formation of hypobromite ion (OBr). Bromine also reacts with ammonia in water to form monobromanine and dibromanine. No stable tribromanine is formed. Monobromanine is a strong bactericide almost as strong as free bromine in contrast to monochloramines. Bromine has been used for disinfection of swimming pool waters on a limited scale. However, because of its higher cost and less effectiveness, its use for public water supply has not found acceptance.

Iodine is a bluish black solid and its addition to water yields Hypoiodous acid (HOI) and Hypoiodite (IO). Iodine reacts less with organic matter compared to chlorine and is relatively stable in water. At pH = 7, the percentage of iodine, Hypoiodous acid and hypoiodite ion have been reported to be 52, 48 and 0 for a total iodine residual of 0.5 mg/l. Both iodine and Hypoiodous acids are equally good disinfectants. Iodine does not react with ammonia to form iodamines but oxidizes ammonia. It also oxidizes phenols. Because of these reasons, less iodine is required to obtain free iodine residual.

Iodine has been used for disinfection of swimming pool waters and small quantities of water in field. Iodine tablets (e.g. of tetraglycine hydroperiodide) have been used by the Army. Iodine is less dependent on pH, temperature, time of contact and nitrogenous impurities than chlorine and can also kill amoebic cysts which chlorine does not. It has the same disinfecting power as chlorine. Because of certain advantages over chlorine, iodine is better for post disinfection than chlorine providing longer lasting protection against pathogens and reduced offensive tastes and odours. However, it is more costly than chlorine.

8.9.2.2 Ozone

It is a faintly blue gas of pungent odour. Being unstable it breaks down to normal oxygen and nascent oxygen. This nascent oxygen is a powerful oxidizing and germicidal agent. Ozone is produced by the corona discharge of high voltage electricity into dry air. Ozone, being unstable, has to be produced onsite.

Ozone possesses more superior bactericidal properties than chlorine and is highly effective in removal of tastes, odours, colour, iron and manganese. As ozone reacts with chemical impurities prior to attacking the microorganisms, it produces essentially no disinfection unless ozone demand of water has been satisfied but much more rapid kills are achieved, once free ozone residuals are available. Studies have reported 99.99% kill of E.Coli

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within less than 100 seconds in the presence of only 10 bg/1 of free available ozone. Ozone is effective in killing some chlorine resistant pathogens like cysts and certain virus forms. Ozone, unlike chlorine, does not impart offensive tastes and odours to water, nor does it usually produce toxic substances such as chlorinated hydrocarbons. Further the efficiency of disinfection by Ozone is unaffected by pH or temperature of the water over a wide range.

Among the disadvantages of ozone treatment are:

(i) Its high cost of production.

(ii) Inability to provide residual protection against recontamination.

(iii) Its generation onsite due to unstability.

However, despite these disadvantages, ozone has been extensively used in Europe for disinfection of municipal water supplies.

8.9.2.3 Potassium Permanganate

Potassium permanganate is effective in removal of taste and odour and inorganic impurities such as iron, manganese and hydrogen sulphide besides possessing noticeable disinfecting properties. It is more expensive disinfectant than chlorine but has the advantages of not producing offensive taste, odour and/ or potential toxicity.

8.9.2.4 Metal Ions

Several metals including silver, copper, mercury, cobalt and nickel possess significant bactericidal properties. However, except silver, none has been found suitable for disinfecting drinking water supplies. Silver is relatively ineffective against viruses and cysts in acceptable concentrations. Long detention periods are required but very low concentrations of the order of 15bg/l are sufficient to destroy most organisms. Silver can be introduced in water either in the form of a silver salt or by immersing silver or silver-coated electrodes in the water and applying an electrical potential. Successful applications at 100 V have been reported. As the solubility of most silver salts is adequate, enough silver ion may dissolve which is considered sufficient for most disinfection purposes.

8.9.2.5 Acids And Bases

Addition of acids or bases resulting in pH values below approximately 3 and above approximately 11 creates toxicity and pathogens do not survive long in such waters. Increased acidity and basicity increases ionic strength and osmotic pressure which are suggested to be responsible for the destruction of cells.

Disinfection of water is achieved during Lime-soda softening process due to increased pH. However, explicit use of acids and alkalies for the sole purpose of disinfecting water is not commonly practiced. Changes in pH may have marked effect on the efficiency of disinfection by certain chemical disinfectants such as chlorine.

8.9.3 RADIATION

Several types of radiations including ultraviolet, gamma and X-rays and microwaves are cited in literature, for destruction of microorganisms. While high energy gamma rays of 1.10 and 1.30 million electron volts (Mev) are emitted by cobalt-60, X-rays are produced by electron bombardment of a heavy metal target in an evacuated X-ray tube. The wavelength of gamma rays corresponding to 1.10 Mev is 0.03366 A^0 compared to the wavelengths of 2537 A^0 of ultraviolet rays, making gamma rays one million times as powerful and giving them great penetrating properties. Gamma rays are highly effective even against spores and viruses. Death of cells results from ionization reactions within cell molecules and secondary radiation effects and chemical reactions due to production of unstable atoms, free radicals and other chemical species formed by the interactions of gamma rays with organic molecules present in water. A dosage of 100,000 to 150,000 rads (1 rad = 100 ergs per gm.) to the organisms is sufficient with the exception of Bacillus subtilis which is difficult to kill. X-rays also have been found to be suitable for killing bacteria. However, because of the high costs involved and other unfavourable properties of leaving no residual protection against recontamination, these have yet to find large scale practical applications in water treatment practice.

8.9.3.1 Ultraviolet Radiation

It was observed that exposure of water to sunlight and artificial light leads to destruction of organisms. These bactericidal effects of intense sunlight or artificial light are primarily due to ultraviolet rays. Ultraviolet radiation may kill a cell, retard its growth, change its heredity by genemutation. Depending upon the dose of radiation and the particular portion of the cell receiving radiation, one or several of the above mentioned three effects may occur. Wavelength region from 2500-2650 A^0 is recommended for maximum destruction of cells.

Ultraviolet rays are most commonly produced by a low pressure mercury lamp constructed of quartz or special glass which is transparent and produces a narrow band of radiation energy at 2537 A^0 emitted by the mercury-vapour arc. Efficient disinfection can be achieved if:

(i) Water is free from suspended and colloidal substances causing turbidity.

(ii) Water does not contain light absorbing substances such as phenols, ABS and other aromatic compounds.

(iii) Water is flowing in thin films or sheets and is well mixed.

(iv) Adequate intensity and time of exposure of UV-rays is applied.

About 2% of applied energy of ultraviolet rays may be reflected and some energy is absorbed by the impurities present leading to attenuation of radiant energy. Even distilled water will absorb about 8% of the applied energy for a water depth of 30mm, including surface reflection of 2%. Presence of iron even at low concentration of 1mg/l may drastically increase absorption by over 80%. Water depths of about 120 mm are recommended for efficient disinfection.

Intensity of ultraviolet rays is expressed in terms of germicidal unit which is an intensity of 100 mw per sq. cm. at wave length of 2537 A^0. It has been reported that Escherichia Coli kills of 99.99,99, and 90% can be achieved by ultraviolet rays of 3000, 1500 and 750 mW-sec per sq.cm. Typically a 30 watt lamp could achieve 99.9% kill for water flows of approximately 2.5 to 17.0 m^3/hr hr for water depths ranging from 125 to 880 mm approximately assuming 90% absorption of ultraviolet rays.

The advantages of ultraviolet radiation are that exposure is for short periods, no foreign matter is actually introduced and no taste and odour produced. Over exposure does not result in any harmful effects. The disadvantages are that no residual effect is available and there is lack for a rapid field test for assessing the treatment efficiency. Moreover, the apparatus needed is expensive.

CHAPTER 9
SPECIFIC TREATMENT PROCESSES

9.1 INTRODUCTION

Water treatment involves physical, chemical and biological changes that transform raw water into potable water. The treatment process used in any specific instance must depend on the quality and nature of the raw water. Quality requirements for industrial uses are frequently more stringent than for domestic supplies. Additional treatment may be required by the industry like demineralization of boiler feed water to prevent scale deposit.

Water treatment processes may be simple like sedimentation or may involve complex physico-chemical changes, as with coagulation. The specific treatment processes include control of algae, control of taste and odour in water, removal of colour, softening, removal of iron and manganese, defluoridation of water, demineralization of water and corrosion.

9.2 CONTROL OF ALGAE

9.2.1 GENERAL

Algae give rise to a variety of troubles in water supplies. They impart odours and tastes to the water. Synura causes a perceptible odour. Asterionella, Meridion and Tabellaria produce aromatic odour. Algae like Dinobryon, Peridinium, Uroglenopsis, Asterionella and Tabellaria produce fishy odour. Grassy odour is caused by Aphanizomenon,Anabaena, Gomphosphaeria, Cylindro-spermum and Rivularia. Septic odour is caused by Cladophora, Hydrodictyon, Ceratium, Aphanizomenon, Anabaena and Cylindrospermum. When algae like Microcystis, Anabaena and Aphanizomenon die en mass and decay, they produce foul odours.

Some algae impart sweet or bitter or sour tastes to water. Algae like Nitella, Geratium and Synura give rise to bitter taste, while algae such as Chara, Euglena, Aphanizomenon, Microcystis, Cryptomonas and Gomphosphaeria impart sweet taste to water.

Algae interfere in the process of flocculation and sedimentation. Algae like Asterionella and Synedra prevent floc formation. Water containing Gomphosphaeria and Anabaena need to be agitated for proper floc formation. They buoy up the flocs and carry into the filters. They choke the filters and as a result reduce the filter runs. Algae associated with filter clogging are Asterionella, Fragilaria, Navicula, Synedra, Cymbella, Diatom, Oscilatoria, Rivularia, Trachelomonas and Closterium. Algae like Synedra and Oscillatoria can pass through rapid sand filter. Algae such as Euglena, Phacus, Navicula, Nitzschia and Trachelomonas get through slow sand filter. These algae in distribution system cause biological corrosion.

Lyngbya, Anabaena, Cylindrospermum, Nodularia and Microcystis are some of the common toxic algae associated with fish and cattle mortality. Hay fever is caused by Anacystis and Lyngbya Contorta. The gastrointestinal disturbances are also said to be due to algal toxicity.

Algae may be killed by treating the water with suitable chemicals. However, the procedure of allowing the algae to establish themselves and then adopting algicidal measures has a number of disadvantages, viz. (i) the dose of chemical required is greater than that needed, if the treatment is adopted at the incipient stages of growth (ii) the dead algae decay and produce acute odour problems (iii) the dead algae provide a pabulum for a second crop which are generally more prolific than the first and also more resistant to the action of algicides. It is therefore, preferable to take all possible measures to discourage the growth of algae and to reserve the use of algicides as a final treatment.

9.2.2 Causative Factors for Growth

Algal growth is influenced by a number of factors such as nutrients in the water, the availability of sunlight, the character of the reservoir and temperature.

9.2.2.1 Nutrients in Water

Nutrients like nitrogen and phosphorus favour the growth of algae. Swamp water or water in contact with decaying vegetation as well as water polluted by sewage contain large amount of organic matters favouring certain types of algal growth. Among the various mineral compounds, those of nitrogen and phosphorus are particularly favourable and are generally brought in by agricultural return waters and some industrial wastes. Algicidal treatments have limited value when the water is rich in such nutrients because the conditions are favourable for the growth of succeeding crops of algae.

9.2.2.2 Eutrophication

Eutrophication is the process whereby lakes become enriched with nutrients that make the water undesirable for human use, both for water supplies and recreation. Limnologists categorize lakes according to their biological productivity. Oligotrophic lakes are nutrient poor. Typical examples are a cold-water mountain lake and a sand bottomed, spring-fed lake characterised by transparent water, very limited plant growth, and low fish production. A slight increase in fertility results in a mesotrophic lake with some aquatic plant growth, greenish water and moderate production of game fish. Eutrophic lakes are nutrient rich. Plant growth in the form of microscopic algae and rooted aquatic weeds produces a water quality undesirable for body-contact and non-body contact recreation.

9.2.2.3 Effects of Eutrophication

The process of eutrophication is directly related to the aquatic food chain. Algae use carbon dioxide, inorganic nitrogen, orthophosphate and trace nutrients for growth and reproduction. These plants serve as food for microscopic animals (Zooplankton). Small fishes feed on Zooplankton and large fishes consume small ones. Abundant nutrients unbalance the normal succession and promote blooms of blue-green algae that are not easily utilized as food by Zooplankton. Thus, the water becomes turbid. Floating masses of algae are windblown to the shore where they decompose producing malodours. Decaying algae also settle to the bottom, reducing dissolved oxygen.

Even a relatively mild algal bloom can result in accumulation of substantial decaying scum along the windward shoreline because of the lake's vast surface area. The most devastating aspect of eutrophication is that the process appears to be difficult to retard. Once a lake has become eutrophic it remains so, at any rate for a very long time, even if nutrients from point sources are reduced.

9.2.2.4 Sunlight

Algae require sunlight for their life processes and hence the growths are profuse in seasons of intense sunlight. Clear waters favour the growth of algae because they permit the penetration of sunlight to greater depths.

9.2.2.5 Characteristics of Reservoirs

Shallow reservoirs offer more favourable conditions than deep reservoirs because their dissolved nutrients closer to the surface may stimulate algal growth. Irregular margins and shallow areas encourage the growth of aquatic weeds which offer anchorage for the epithetic algae.

9.2.2.6 Temperature Effects

Temperature has a considerable influence on algal growth. The blue green and the green algae make their presence when the water temperatures reach 20-30^{0} C.

9.2.3 Remedial Measures

9.2.3.1 Preventive Measures

Preventive measures should, therefore, be based on control of those factors such as reduction of the food supply, change of the environment or exclusion of sunlight though they are not always practicable. Clear water reservoirs, service reservoirs and wells may be covered to exclude sunlight, but such a remedy is obviously inapplicable in the case of large reservoirs of raw waters. Turbid water prevents light penetration and thereby reduces algal population. Activated carbon (10.5 to 24.5 kg/hectare) reduces algal population by excluding sunlight but the disappearance of activated carbon in the water may support algal growth again. To a limited extent, the environmental conditions for the growth of algae may be made unfavourable by proper care in the construction and operation of reservoirs, as explained in 5.2.7.2 (f) and (g).

9.2.3.2 Control Measures-Algicidal Treatment

Algicidal measures may be adopted to control algae in reservoirs. As has been explained earlier, it is preferable to initiate the treatment in the early stages of algal growth.

(a) Microscopic Examination

To decide on the best time at which the water should be treated, it would obviously be necessary to have a regular programme of microscopic examination of the water. Such examination is especially necessary during the season in which algal invasions may be expected.

(b) Time for Treatment

Generally, the practice has been to apply algicides when the total count reaches or exceeds 300 areal units. Algae which are known to be particularly troublesome should be eradicated even though the total count is much less than 300 areal units. For example, algcidal treatment is indicated as soon as Synura, a type that causes severe smell troubles is encountered, irrespective of the total count.

(c) Types of Algicide

A large variety of algicides are available and a number of new algicides are being synthesized. Many of these are complex organic compounds and are credited with specific action against particular species. Chemicals such as ketones, aldehydes, organic acids, quaternary ammonium compounds, silver nitrate, ClO_2 and rosin amines have also been tried as algicides. However, these are costly and have not come into general use. The most widely used algicides are copper salts and chlorine and potassium permanganate in small scale water supplies. The chemical to be used as an algicide should be species selective, non-toxic to aquatic life particularly fish, harmless to human beings, have no adverse effect on water quality and inexpensive and easy to apply.

(1) Copper Salts

The most common algicide is copper sulphate. Its action is due to the copper ion which acts as a direct protoplasmic poison. The reaction is a function of the concentration of the chemical and the time of exposure of the algal cells to the action of the copper.

(i) Copper Sulphate

The copper sulphate reacts with the bicarbonates in the water to form a basic copper carbonate which further decomposes to form copper hydrate. The basic copper carbonate is somewhat soluble especially if the water is not very hard and if it contains carbon dioxide. The copper hydrate is almost insoluble in water. It remains in a colloidal form for sometime before it precipitates out. This reaction is retarded by low temperature and organic matter in the water, while temperature and suspended matter accelerate it. It, thus, follows that the efficacy of copper sulphate as an algicide is influenced by the temperature of the water, its hardness, its content of organic matter and suspended matter.

The added copper sulphate is rendered inactive in a short while. This is both an advantage and a disadvantage. It is an advantage because the content of the copper in the water rapidly gets reduced to levels below those at which copper is toxic to human beings by mere efflux of time and without the need for any elaborate treatment for removal of the excess copper. Tendency of copper to get out of the solution is a disadvantage because the algicidal effect is rendered purely temporary. With the disappearance of the copper from the field of action, another crop of algae can come up necessitating a repetition of the treatment.

(a) Dosage

Doses of copper sulphate required to kill algae are generally expressed in terms of concentration in mg/l of the salt $CuSO_4.5H_2O$. Some authorities express the doses in terms of the equivalent of copper. The values may be inter-converted on the basis of the following reaction :

100 parts of copper sulphate ($CuSO_4 . 5H_2O$) = 22.32 parts of copper

The quantity of copper sulphate required has to be calculated on the basis of the volume of water in the reservoir. A knowledge of the types of the algae present, quantity, period of multiplication and seasonal succession is necessary to decide on the dose to be applied. The lethal dose for different types of algae are given in Table 9.1. These apply for a temperature of 15°C and may be decreased by about 2.5 percent for each degree rise of temperature above 15°C.

TABLE 9.1

COPPER SULPHATE REQUIRED FOR CONTROL OF DIFFERENT ALGAE

Algae	Copper Sulphate concentration, mg/l
I. Cyanophyceae	
1. Anabaena	0.12 – 0.48
2. Aphanizomenon	0.12 – 0.50
3. Clathrocystis	0.12 – 0.25
4. Coelosphaerium	0.20 – 0.33
5. Microcystis	0.20
6. Oscillatoria	0.20 – 0.50
II. Chlorophyceae	
1. Cladophora	- 0.50
2. Clostrium	- 0.17
3. Coelastrum	0.05 - 0.33
4. Drapamaldia	- 0.33
5. Enteromorpha	- 0.50
6. Volvox	- 0.25
7. Hydrodictyon	- 0.10
8. Microspora	- 0.40
9. Scenedesmus	- 1.00
10. Spirogyra	- 0.12
11. Ulothrix	- 0.20
12. Zygnema	- 0.50
III Diatomaceae	
1. Asterionella	0.12 - 0.20
2. Fragilaria	- 0.25
3. Melosira	- 0.20
4. Tablellaria	0.12 - 0.50
5. Navicula	- 0.07
6. Synedra	0.36 - 0.50
7. Stephanodiscus	- 0.33

IV. Flagellates 1. Cryptomonas 2. Dinobryon 3. Euglena 4. Mallomonas 5. Synura	 0.50 0.10 0.50 0.50 0.12 – 0.25
IV. Miscellaneous 1. Chara 2. Nitella	 0.10 – 0.50 0.10 - 0.18

(b) Method of Application

Copper sulphate is generally applied by towing gunny bags containing the crystals tied to the bows of a motor-boat which is plied on the water. The crystals dissolve readily and mix with the body of the water. The rate of dissolution may be varied either by varying the speed of the motorboat or by varying the opening of the bags containing copper sulphate. The boat should traverse a predetermined zigzag path so as to get the best distribution of the chemicals.

It is generally preferable to choose a moderately windy day for the application of copper sulphate and also to ply the motorboat against the direction of the wind. Application of copper sulphate on a bright morning would be preferable as a large proportion of the algae would be at the top. If the algae are a type which remain scattered in the water, the application of copper sulphate in the evening may be preferable. To reach shallow bays, the finely powdered crystals may be scattered by hand or may be sprayed in the solution form with the aid of a suitable pump mounted on the boat.

Copper sulphate can also be applied on the basin walls, weir of settling tanks or filter troughs when the algae become a nuisance.

(c) Effects of Copper Sulphate

Copper sulphate is poisonous to fish. The doses, which are lethal to fish, are given in Table 9.2

TABLE 9.2
CONCENTRATION OF COPPER SULPHATE LETHAL TO FISH

Fish	mg/l
Tout	0.14
Carp	0.30
Suckers	0.30
Catfish	0.40
Pickerel	0.40
Goldfish	0.50

Fish	mg/l
Perch	0.75
Sunfish	1.20
Black bass	2.10

Comparison of these figures with those for the algae shows that the lethal doses for algae are very near the lethal doses for fish. Hence very careful control over the dose and the method of application is necessary to avoid large-scale mortalities of fish.

Despite care, it may be not be possible to avoid the death of a few fish which may have got into the regions of heavy local concentrations of copper sulphate. When a water which is heavily laden with algae is treated with copper sulphate, the dead and decaying algae may reduce the dissolved oxygen in the water to levels at which fish life cannot be sustained. The dead algae may also accumulate on the gills of the fish and smother them. Such death of fish due to smothering usually occurs on the second or third day following treatment by copper sulphate.

(d) Increase of Organic Matter and Bacteria

As a result of the death and decay of the algae, the organic matter in the water is increased and ample supplies of food become available for the saprophytic bacteria which begin to multiply at a very rapid rate with the consequent uptake of oxygen. A point is finally reached when the water cannot support this rapid growth of bacteria and then these gradually get reduced in number.

The decay of the algae also causes a temporary increase in the odour of the water. For these reasons it is necessary to close the supply for several days after treatment with copper sulphate. To permit such closure of supply, it is desirable that every reservoir consists of at least two separate compartments. Generally, the reservoir is ready for service after about two or three days after application of copper sulphate. Good practice requires that laboratory tests for water quality should be made before the water is allowed to pass on to the consumers. Such tests should also include the determination of copper to ensure that it is within the permissible limits given in Table 2.1.

(e) After Growths

The algicidal treatment of a reservoir with copper sulphate may be followed by a secondary growth of algae. This is particularly to be expected when conditions favourable for algal growth are present, viz., water rich in algal nutrients and plenty of sunshine. The secondary growth is generally more profuse than the first and consist of types which are more resistant to the action of copper sulphate than the original forms. Hence it would be necessary to repeat the treatment at intervals and also to employ higher doses of algicides or use more powerful algicides during the subsequent treatments.

(ii) Other Compounds

Attempts have been made to develop compounds of copper which can persist for longer periods. Treatment with copper citrate which can stay in solution for longer periods has been practiced but this chemical is much costlier than copper sulphate. By using a complex of copper with ammonia in conjunction with chlorine-cupri-chloramine process, the algicidal effect of copper is reinforced. The use of more persistent compounds of copper necessitate, however, a more rigid control over the treatment so as to ensure that the water is not supplied to the public until the copper content gets well below toxic limits.

(2) Chlorine

Chlorine is normally a bactericide but also used as an algicide. Whereas copper sulphate is more commonly applied to water in reservoirs, chlorine is generally added to the water as it passes the control point.

Chlorine has specific toxic effect and causes death and disintegration of some species of algae. The essential oils present in the algae are thus liberated and may cause tastes and odours. Occasionally these essential oils as well as the organic matter of the dead algae may combine with chlorine to form new or intensified odours and tastes. Such intensification of odours makes the control of algae by chlorine a problem which challenges the ingenuity of the operator.

(i) Dosage

The lethal doses of chlorine for the more common types of algae are given in Table 9.3

TABLE 9.3

AMOUNT OF CHLORINE REQUIRED TO DESTROY MICROSCOPIC ALGAE

Algae	Chlorine Dose mg/l
Aphanizomenon	0.85
Cyclotella	1.00
Melosira	2.00
Dinobryon	0.5
Uroglenopsis	0.5
Synura	0.3

(ii) Methods of Application

Chlorine may be applied either as a slurry of bleaching powder or as a strong solution of chlorine from a chlorinator. The latter is preferable.

Small reservoirs may be treated by applying a slurry of bleaching powder at the influent end or by towing bags containing the bleaching powder in the water. Chlorination for algal growth is more commonly adopted in the pretreatment part of the water works. The point of application is generally at the point of entry of raw water into the treatment plant or just ahead of the coagulant feed. Algal growths in raw water conduits can be got rid of by heavy doses of chlorine. Addition of chlorine along with coagulant is sometimes practiced, but this is to be discouraged since the turbulence would result in the dissipation and wastage of chlorine.

(iii) Microstrainer

A special process known as microstraining is being used in some water treatment plants. The microstrainer is an open drum. The water is passed through a finely woven fabric of stainless steel. The size of the openings in the mesh determines the size of the plankton removed from the water.

9.2.3.3 Relative Merits of Chlorine and Copper Sulphate Treatment

Each plant should conduct experiments and decide on the type and dose of the algicide on the basis of local conditions. To a certain extent the method will depend on the facilities available for dosing the water with chemicals, the general arrangements of the system as well on the costs. There are, however, certain special conditions where the use of copper sulphate is not possible and chlorination has to be preferred. For example, when the point of application is too near the point of entry into a pipe, copper sulphate cannot be used as the copper will plate out on the metal and become inactive. Similarly when the problem is to prevent algal growth in a coagulant basin, copper sulphate cannot be used as it will be thrown out of solution almost immediately. In cases where the supply cannot be shut off for periods sufficient to cause a reduction of the copper content to permissible limits, chlorination has to be preferred.

When chlorination causes an intensification of the algal tastes and odours, application of heavy doses of chlorine followed by removal of the excess usually overcomes the difficulty.

The growth of plankton in reservoirs may be controlled by copper sulphate treatment. Generally satisfactory results have been secured, but this chemical has not always been effective as an algicide. The doses required for this purpose differ with each organism, so, economy in the use of copper sulphate and its distribution in reservoirs warrants microscopic examination of appropriate samples collected at significant locations, to determine the types of organisms and their relative numbers. The recommended doses are 0.3 mg/l or less, so this dose may be used in the absence of laboratory control. On the other hand, many troublesome organisms may be killed with doses of 0.12 mg/l, indicating the economy possible when microscopic examinations can be made.

The required dose is influenced by temperature, alkalinity and carbon dioxide content of waters.

Effective control of microorganisms throughout the year is facilitated by the continuous application of copper sulphate to the water entering reservoirs, the microorganisms are thus controlled before heavy growths occur, avoiding the necessity of periodic treatment. Furthermore, the prevention of growths obviates the subsequent destruction of large quantities of organisms, which would result in the reduction of the dissolved oxygen content of the water, and hence protects fish life because the lowering of the oxygen content is frequently responsible for fish-kills, erroneously blamed on copper sulphate. The continuous application of copper sulphate, however, reduces the available supply of food for fish, so continuous treatment should be restricted to those reservoirs used exclusively as sources of public water supply.

Copper sulphate may be continuously applied with either commercial chemical feeders or homemade equipment of a solution tank and a constant-head tank with an orifice or control valve. Another simple device consists of a perforated box so supported in the water of the entering stream that the depth of submergence may be varied at will. Lumps of copper sulphate are placed in the box, to be dissolved by the water flowing through the box. The rate of solution may be controlled by raising or lowering the box, which should be kept filled to a point above the water level. Doses of 0.12 mg/l have been found effective in raw-water reservoirs, and doses as low as 0.03 mg/l have been used to treat filtered water before it enters open storage reservoirs.

Algae in water treatment plant may be removed by the application of chlorination, ozone, chlorine dioxide or activated carbon. Prechlorination will kill many of the algae and facilitate their settling. Prechlorination will prevent the growth of algae in basin walls and will aid in the removal of algae by coagulation and sedimentation, because the dead cells of these organisms are more readily coagulated. The chlorine in the settled waters will also destroy slim organisms on the filter sand and thus prolong filter runs and facilitate filter washing. Doses required for this purpose may have to be over 5.0 mg/l to meet the chlorine demand of water, oxidize free ammonia, etc. and leave 0.2 to 0.5 mg/l free residual chlorine in the settled water.

Most economical results are secured with the use of prechlorination for initial disinfection by free residual chlorine and post chlorination with chlorine dioxide. Chlorine dioxide doses sufficient to give an apparent content of 0.2 to 0.3 mg/l free residual chlorine in the filtered water are adequate, this amount of chlorine dioxide being equivalent in oxidizing power to 0.5 to 0.75 mg/l free residual chlorine.

Ozone is very effective in the destruction and gives more uniform, predictable results and is a very active oxidizing agent. Ozone is only slightly soluble in water and hence persists in the treated water for periods upto about 30 minutes. Control is through the use of special equipment indicating the concentration of ozone in the treated water, or by the orthotolidine test. The latter indicates the presence of 0.1 mg/l ozone when the reagent colour is equivalent to 0.15 mg/l of residual chlorine.

9.3 CONTROL OF TASTE AND ODOUR IN WATER

9.3.1 General

Taste and odour in water are subjective phenomena and are difficult to quantify exactly. The problems of taste and odour (one co-exists with the other) are more intensive and more frequent in surface water sources as these are more subject to contamination by natural and man made wastes. Taste and odour are caused by dissolved gases like hydrogen sulphide, mercaptans, methane, organic matter derived from certain dead or living micro-organisms (blue and green algae), decomposing organic matter, industrial liquid wastes containing phenols, cresols, ammonia, agricultural chemicals, high residual chlorine and chloro-phenols. It is possible that some of the dissolved gases may be found in well water also. Odour can be classified as aromatic, earthy, swampy, septic or chemical.

Biological organisms are one of the most common causes of taste and odour in water. Diatomaceae with Asterionella and Synedra, actinomycetes and free swimming nematodes are the principal offenders causing earthy or musty odour. Apart from algae, decomposing leaves, weeds or grasses also cause odour. Vegetation that grow in the low-water areas in the reservoir subsequently get submerged and decompose resulting in odour. Chemical and refinery effluents have the greatest potential for odour, followed by domestic sewage. Odour tests indicate that only a few mg/l of these materials are needed to produce a detectable odour.

In short, taste and odour producing materials in water are chemical compounds of many varieties with different physical and chemical characteristics, present in water because of direct pollution or biological activity. Most of these compounds are in solution and some exist in the form of particulate and colloidal compounds. Those in solution are comparatively more difficult to remove.

9.3.2 CONTROL OF TASTE AND ODOUR

Preventive and corrective treatment of raw and processed water is necessary for control and elimination of taste and odour problems. Wherever possible, preventive steps like control of microorganisms are to be undertaken where the source of raw water supply is from rivers, reservoirs or lakes and control of effluent discharges. Special treatment is given to water in the treatment plant for the removal of odours, by aeration, oxidation by chemicals or adsorption by activated carbon.

9.3.2.1 Preventive Measures

Reservoir preparation and management are fundamental for an effective tackling of the problem. For construction and management of reservoirs, 5.2.7.2 (f) and (g) may be consulted. Control of algae for the mitigation of this problem has been discussed in detail in 9.2.

9.3.2.2 Corrective Measures

Odours can be removed by mechanical aeration, oxidation by chemicals like chlorine or its compounds or ozone or permanganate and adsorption of odour by agents such as activated carbon, floc or clays.

For removing dissolved gases like hydrogen sulphide and volatile matter, aeration can be practised at the start of water treatment. Free available residual chlorination at prechlorination or postchlorination stage can bring about complete elimination of taste and odour. Inadequate chlorination will only intensify the odour of water containing phenolic compounds and tannin and lignin imparting a medical taste. Even with breakpoint chlorination it may not be possible to remove taste and odour from water in certain cases. Such compounds can be removed by super-chlorination. Super-chlorination is normally done at the lake outlet or the plant inlet in order to bring the maximum chlorine concentration and the maximum contact time together to effect oxidation. This should invariably be followed by dechlorination using sulphur dioxide or sodium sulphate to reduce the residual chlorine to acceptable limits. Using ammonia with chlorine in combined residual chlorination can partly mask or delay chlorophenol tastes in water.

Chlorine Dioxide which is 2.5 times more powerful than chlorine as an oxidizing agent has been found extensively efficient and the general dosage values range from 0.2 to 2.0 mg/l. This is a specialized form of chlorine treatment used for taste and odour control where large doses of chlorine are to be avoided. Chlorine dioxide gas is released in water on site by the inter-action of a solution of sodium chlorite ($NaClO_2$) with a strong chlorine solution of 6000-7500 mg/l

$$2NaClO_2 + Cl_2 \longrightarrow 2ClO_2 + 2NaCl$$

Through the theoretical ratio of chlorine to sodium chlorite is 1 : 2.6, values between 1:2 and 1:1 are employed in practice. Chlorine dioxide is more expensive and is used for taste and odour control only. It is applied at the first stages of the treatment plant. Thereafter, the final desirable residual chlorine may be adjusted by simple chlorination after filtration. Ozone at dosages of 1.0 mg/l has also produced good results. Chloramination is useful in the removal of phenol tastes.

Preferred method of treatment for taste and odour removal is activated carbon. Activated carbon is made from hydrocarbon or carbohydrate sources, the principal requirement being that the carbon residue left after destructive distillation has a porous structure. Odour producing substances which cannot be removed by oxidation are physically adsorbed on to the surface. This treatment is usually applied before filtration. The contact time varies from 10 to 60 minutes. Activated carbon performs well at lower pH values. A bed of carbon or suspension kept in circulation could be used. The active surface must be preserved from coating by other chemicals. Application of carbon can be before sedimentation if taste and odour is severe and frequent and in certain cases after sedimentation. The approximate dosage for routine, continuous application as suspension is 2 to 8 mg/l, for emergency treatment 20 to 100 mg/l. Carbon beds are generally 1.5 to 3 m deep with the sizes 0.2-0.4 mm with loadings of about 4.8 $m^3/hr/m^3$ of bed. Filtration rates range from 7.2 to 15 $m^3/m^2/hr$ with expected efficiencies of about 90%. As many variables are involved, pilot plant tests are indicated. Carbon can also be used as a polishing agent to remove residual odours after other treatment.

Variables such as pH, temperature, quantity and type of organic matter in the influent water and detention time have a marked effect on the efficiency of removal of odorous materials.

9.4 REMOVAL OF COLOUR

9.4.1 CAUSES OF COLOUR

Colour in water may be due to natural causes or as a result of human activity. Waters occurring in peaty soils acquire colour because of the presence of colloidal organic matter. Colour is also due to mineral matter in solutions, as a colloid or in suspensions as in the case of ground water in certain areas. Waters containing oxidized iron and manganese impart characteristic reddish or black colour. Heavy growths of algae may also impart colour to the water. Discharge of industrial wastes or heavy sewage pollution may also bring in colour.

9.4.2 Colour Removal

The appropriate treatment for the removal of colour from a water has to be determined for each individual case on a consideration of the causative factors and on the basis of local trials.

9.4.2.1 Colour due to Iron and Manganese

Colour due to iron and manganese may be removed by specific treatment for the removal of these constituents as discussed in 9.6.

9.4.2.2 Colour due to Algae

A water which is coloured because of the growth of algae, has to be treated to eliminate the source by control of the algae as discussed in 9.2 or to remove them by processes such as micro-straining. Microstrainers are cylindrical drums 3 m diameter x 3 m long with stainless steel screens having openings of 0.025 mm. The proprietary units have sizes varying from 0.75 m dia x 0.6 m long to 3 m dia x 3 m long. The latter size will handle 10-30 lpm of water with a power consumption of about 3 kW. The headloss is only about 150 mm with unit capacities of 7.2 to 12 m^3/hr per m^2 of strainer, the capacity depending upon the concentration of microorganisms in the raw water. The coatings of microorganisms are continuously washed down by a jet of water, with the volume of wash water varying from 1 to 3% of the volume of water strained.

9.4.2.3 Colour due to Colloidal Organic Matter

Coagulation at low pH range by chemicals such as alum or ferric salts is used for removing colour due to colloidal organic matter. Ferric coagulants are generally superior to alum. After removal of the colour colloids the pH of the water will have to be corrected by treatment with lime. The colour colloids are often stabilized at high pH value and hence the addition of lime to aid coagulation is fraught with danger in the case of waters which are coloured. It is essential that laboratory tests should be conducted to determine the most suitable chemical and its optimum dosage in the given conditions.

9.4.2.4 Colour due to Industrial Wastes

Colour due to industrial wastes may be removed by the use of bleaching powder or chlorine or by activated carbon. Since removals are more efficient when the concentrations are high, it is advisable to treat the effluents for colour at the factory site itself before discharge into the water bodies.

9.4.2.5 Oxidation of Colour

In some cases, colour is not removed by coagulants and it will be necessary to oxidize the coloring matter. Application of heavy doses of chlorine is one of the methods commonly adopted. When the colour is not destroyed by such treatment, the water may have to be treated with strong oxidizing agents like chlorine dioxide. Refer 9.3.2.2.

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9.4.2.6 Treatment by Activated Carbon

Treatment with activated carbon is effective against most problems of colour in waters. Carbon removes the coloring matter by adsorption. Application has already been discussed in 9.3.2.2.

9.5 SOFTENING

9.5.1 GENERAL

Water is said to be hard when it does not form lather readily with soap. The hardness of water is due to the presence of calcium and magnesium ions in most cases.

Bicarbonates, sulphates and chlorides are the anions associated with the hardness. The purpose of softening is to remove these salts from the hard water, to reduce the soap consuming properties and to ensure longer life to washed fabrics, mitigate its scale forming tendencies, and improve palatability.

Usually a total hardness of 75 to 100 mg/l (as $CaCO_3$), would meet these requirements. The magnesium hardness should not exceed 40 mg/l to minimize the possibility of magnesium hydroxide scale in domestic hot water heaters. Calcium and magnesium associated with bicarbonates are responsible for carbonate hardness and that with the sulphates, chlorides and nitrates contribute to non-carbonate hardness.

Normally, the alkalinity measures the carbonate hardness unless it contains sodium alkalinity. The non-carbonate hardness is measured by the difference between the total hardness and the carbonate hardness. Carbonates and bicarbonates of sodium are described as negative carbonate hardness.

A summary of the more common salts and the problems they cause in water are presented below:

Alkaline causing		Saline (neutral causing)		Acidity causing
Na and K Alkalinity only	CO_3 Hardness	Non CO_3 Hardness	Salinity only	
$KHCO_3$	$Ca(HCO_3)_2$	$CaSO_4$	K_2SO_4	Mineral acids and
K_2CO_3	$CaCO_3$	$CaCl_2$	KCl	acid salts are
$NaHCO_3$	$Mg(HCO_3)_2$	$MgSO_4$	KNO_3	restricted to acid
Na_2CO_3	$MgCO_3$	$MgCl_2$	Na_2SO_4	mine wastes and
				rare mineral water
			NaCl	
			$NaNO_3$	$FeSO_4$

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Water is classified with regard to its hardness as follows :

Classification	Total hardness as mg/l of $CaCO_3$
Soft	50
Moderately Hard	50-150
Hard	150-300
Very hard	300

When hardness is less than 150 mg/l, softening for domestic purposes is not usually justified.

9.5.2 Method of Softening

The two methods ordinarily used are lime and lime-soda softening and ion-exchange softening.

9.5.2.1 Lime and Lime-Soda Softening

Softening with these chemicals is used particularly for water with high initial hardness (greater than 500 mg/l) and suitable for waters containing turbidity, colour and iron salts because these have a tendency to inactivate the ion-exchange bed, by a coating on the granules. Lime-soda softening cannot, however, reduce the hardness to values less than 40 mg/l while ion-exchange softening can produce a zero-hardness water.

(a) Lime Soda

(i) Chemical Reactions

When lime and soda are added to water containing calcium and magnesium salts, the following reactions take place.

$$CaO + H_2O \longrightarrow Ca(OH)_2 \qquad (1)$$

(Quicklime Slaking)

$$Ca(OH)_2 \rightleftharpoons Ca^{++} + 2OH^- \qquad (2)$$

$$CO_2 + OH^- \rightleftharpoons HCO_3^- \qquad (3)$$

(Removal of CO_2)

$$HCO_3^- + OH^- \rightleftharpoons CO_3^{2-} + H_2O \qquad (4)$$

$$Ca^{++} + CO_3^{--} \rightleftharpoons CaCO_3 \qquad (5)$$

Solid (Removal of Calcium hardness)

$$Mg^{++} + 2OH^- \rightleftharpoons Mg(OH)_2 \qquad (6)$$

Solid (Removal of Magnesium hardness)

Reaction (3), (4) and (5) indicate that carbon dioxide, which is present in water, is converted by hydroxyl ions in lime to CO_3^{2-} which combines with Ca^{++} to form relatively insoluble $CaCO_3$. The alkalinity present as bicarbonate furnishes the necessary CO_3^{2-} by reaction with added OH^- (equation 4) for equation (5) to be completed. Magnesium ions will have to be removed as $Mg(OH)_2$, according to equation (6) since $MgCO_3$ is fairly soluble. The removal of Mg^{++} is effective around pH ranges of 10 to 10.5 for which additional OH^- ions in the form of lime have to be made available. No reduction of hardness takes place by removal of magnesium by the addition of lime since an equivalent amount of Ca^{++} ion is put back into the solution from the lime added. When all the alkalinity is used by OH to form CO_3^{2} any further CO_3^{2} needed, has to be added to the water. This is the case when non-carbonate hardness is present in the water and the needed CO_3^{2-}; can be added in the form Na_2CO_3 (Soda ash).

For calculating the theoretical amount of lime and soda required for softening an analysis of the following constituents is necessary, viz., free CO_2,bicarbonate (usually the total) alkalinity, total hardness and magnesium. Appendix 9.1 gives illustrative example to compute chemical dosage required in water softening using lime-soda process. Chemical requirements (mg/l) are computed by the sum of the following factors.

On a 100% purity basis, the dosage of lime as CaO required for softening as obtained from the chemical equations is as follows :

1.	For every gram of CO_2 to be removed	1.27 g
2	For every gram of CO_3 hardness as $CaCO_3$ to be removed	0.56 g
3.	For every gram of Mg as Mg to be removed	2.33 g
4.	Additional lime required for raising the pH to the range of 10 to 10.5 for precipitation of $Mg(OH)_2$	30 50 mg/l
5.	Soda ash requirements as Na_2CO_3 to remove one gram of non-carbonate hardness as $CaCO_3$	1. 06 g
6.	Additional soda required to neutralize every gm of excess lime	1. 89 g

Plant conditions like temperature, time of detention and agitation influence the completion of reactions and the dosages of chemicals may have to be increased to provide for the inadequacies.

Alternatively caustic soda can be used instead of lime. The reactions are:

$$NaOH \rightleftharpoons Na^+ + OH^-$$

$$OH^- + HCO_3^- \rightleftharpoons CO_3^{2-} + H_2O$$

$$Ca^{++} + CO_3^{2-} \rightleftharpoons CaCO_3$$

$$Mg^{++} + 2OH^- \rightleftharpoons Mg(OH)_2$$

Liquid caustic soda can be used since it can be handled and fed easily. The amount of calcium carbonate sludge formed in this case is theoretically half that formed by the use of lime. However, caustic soda is costlier than soda ash which is more expensive than lime.

Waters higher in carbonate than in noncarbonate hardness will require relatively more lime than soda ash for their treatment. The sludge settling out carries with it a large portion of the turbidity, iron, manganese, silica, colour producing matter and bacteria in the water. The softened water may have a higher pH due to neutralization of CO_2 with lime, thus reducing its corrosive nature.

(2) Process Equipment

Lime soda softening plants include chemical feeders, rapid mix, flocculation and sedimentation basins and rapid sand filters. The process design must ensure the promotion of the chemical reactions necessary to remove the hardness from water by converting them into insoluble precipitates and then settle these precipitates and filter the partially clarified water.

(i) Chemical Feeders

The solubility of calcium hydroxide (slaked lime) is low being of the order of 1600 to 1800 mg/l in cold water. It is common practice, therefore, to feed a 5% slurry of lime so that unusually large solution tanks are avoided. Soda ash is added in solution form. Types of chemical feeders used are discussed in 7.3.

(ii) Rapid Mix and Flocculating Basins

Rapid mixing ensures a thorough mixing of the slaked lime (lime slurry) and soda ash with the water. A mixing period of 5 to 10 minutes is necessary due to the low solubility of lime. Peripheral speeds of paddles for rapid mixing basins should be 45 to 60 cm/ sec with water velocities of 15 to 30 cm/sec. The addition of lime and soda ash results in a supersaturated solution of calcium carbonate and magnesium hydroxide after which some precipitation of the two compounds starts immediately. These are kept in suspension to further the growth of these precipitates. Some of the settled sludge deposited, from the settling basin is often recirculated to the rapid mix basin for aiding in flocculation. This may ensure promotion of chemical reactions with the previously precipitated solids and the formation of more readily settleable precipitates. The flocculation basin promotes precipitation and floc growth after the chemical reactions have been completed.

The removal of the reaction products takes place by precipitation on the surface of the previously formed particles which enhances the reaction rate. The flocculation time required varies from 40 to 60 minutes. Mechanical flocculators are preferred to baffled ones. Tapered flocculation is generally preferable. The precipitates are formed on the paddles in rapid mixing and flocculation basins, which may require periodic cleaning.

(iii) Sedimentation Basins

They may be circular or rectangular and the detention time varies from 2 to 3 hours depending on the character of the flocs. The limiting horizontal velocity is usually about 0.3 m/min. A large volume of sludge is produced since every mg/l of hardness produces approximately 2 mg/l of dry sludge and therefore an extra basin capacity must be provided. Continuous mechanical sludge collection and removal is almost always used.

Another type of softener is the sludge blanket type which is used for flows larger than 2 MLD and where compactness is an essential requirement. The treated water is passed and sieved upward through a suspended sludge blanket composed of previously formed precipitates. Some of the hydrated lime in suspension may be carried down by sludge. In this type, the upward flow or sieving through the blanket offers full scope for a complete solution and utilization of the added lime. The intimate contact of the treated water with a large mass of solids which serve as nuclei prevents supersaturation and hence overcome problems of after precipitation. The need for separate filtration can also be dispensed with. The upflow velocity at the sludge blanket zone should be equal to one half the settling velocity of the floc in order to have a good blanket of flocs. Overflow rates of 50 to 100 $m^3/d/m^2$ could be used. Experience has shown that use of precipitated calcium carbonate granules or other suitable catalysts of 0.3 to 0.6 mm grain size in these sludge blanket units could reduce the detention times to one quarter that of the regular units without loss of efficiency.

(3) Sludge Disposal

The disposal of sludge is usually carried out by lagooning it. Plenty of land area is required as the sludge does not dry up rapidly. Lime can be reclaimed from the sludge and it has to be free from magnesium if it is to be reused. One of the common methods (Hoover Process) is to precipitate the calcium sludge in the first stage by adding just enough chemicals and then precipitating the magnesium sludge in the second stage. The sludge from the first stage, which is devoid of magnesium, is dried and calcined. The resulting calcium oxide is reused for softening and the carbon dioxide utilized for recarbonation. Another method (Lykken-Estabrook Process) is to apply the total chemicals required for the precipitation of calcium of about 12% of the water and precipitating the calcium and magnesium sludge which is wasted. The water from this portion carrying excess chemicals is then mixed with the rest precipitating only calcium sludge which can be calcined and reused. The reclamation processes also reduce the quantity of sludge to be handled.

(4) Filters

(a) Conventional softening units are followed by rapid sand filters but incrustation of filter sand due to precipitation of super-saturated calcium carbonate has to be guarded against. The application of 0.5 mg/l or more of polyphosphate or metal phosphate has been found to prevent effectively not only the sand incrustation but also coatings of calcium carbonate on the filter walls and wash water troughs. Surface wash devices are used to remove loose deposits. Another method of checking after precipitation is to adjust the pH value of the softened water to about 8.3 by recarbonation. Carbon Dioxide is applied to the effluent from the settling tank. The gas can be obtained by burning coke, oil or gas in excess of air and then scrubbed to remove the other gases. In small plants, dry ice or liquid CO_2 can be used. The carbonation tank must provide 15 to 30 minutes retention time with a depth of water equal to 3 to 4.5 m and the whole plant should be well ventilated.

(b) Excess Lime Treatment

Plain lime or lime soda treatment is used when the bulk of hardness is due to calcium and magnesium is not significant. However, when the water contains more than 40 mg/l of magnesium warranting its removal, excess lime treatment has to be resorted to since magnesium has to be removed as magnesium hydroxide whose solubility decreases with increasing pH values. The water treated thus is highly caustic and has to be neutralized following precipitation. This is done either by recarbonation or by split treatment. In the latter case, the total flow is divided into two parts, one part being treated with excess lime and the settled effluent mixed with unsoftened water. The final residual hardness in the water will depend upon the percentage flow bypassed and the levels of hardness in both the portions.

(c) Hot Lime Soda Process

The hot lime-soda process is used for boiler feed water treatment. This is similar to cold process except that the raw water is heated to about 95° to 100° C before it is taken to the reaction tank. The reactions take place rapidly, the decreased viscosity hastening the settling of the precipitates. A greater degree of softening is accomplished than that in the conventional cold processes.

9.5.2.2 Ion Exchange Softening

The ion exchange process is the reversible inter-change of ions between a exchange medium and a solution and this process is used extensively in water softening. The hardness producing ions preferentially replace the cations in the exchangers and hence this process is also known as base exchange softening. The ion exchange can produce a water of zero hardness. There is only a temporary change in the structure of the exchange material which can be restored by regeneration. The ion exchanger can work on the hydrogen or sodium cycle, the hydrogen ions being released into the water in the former case and the sodium ions in the latter. The regenerants are an acid and sodium chloride respectively. In general the ion exchange materials used in softening, also called zeolites, are hydrated silicates of sodium and aluminium having the formula $xNa_2O.yAl_2O_3.zSiO_2$. The reaction can be depicted as follows.

$$Na_2Z + \frac{Ca^{++}}{Mg^{++}} + 2A^- \longleftrightarrow \frac{Ca}{MgZ} + 2Na^+ + 2A^-$$

Where A^- represents the relevant anions of bicarbonates, sulphates or chlorides and Z represents anionic part of the zeolite.

Treatment with zeolite thus increases the dissolved solids in the ratio of 46 : 40 of the hardness removed. The reverse equation operates during the regeneration resulting in a strong solution of calcium and magnesium salts, which is run to waste.

(a) Inorganic Zeolites

The two common inorganic zeolites are the natural and synthetic types. The natural zeolite is available as green sand while the synthetic or gel type is obtained by the reaction of either sodium aluminate or aluminium sulphate with sodium silicate which after, drying is graded to suitable sizes by screening. A cubic metre of green sand weighs 1600 kg with a specific gravity of 2.1 to 2.4 and for regeneration it requires 3.5 to 7 kg of salt for every kg of hardness removed. The synthetic inorganic zeolite weights 900 to 1100 kg per cubic metre. The relevant exchange capacities and regenerant requirements are given in 9.5.2.2 (d).

(b) Organic Zeolites

They are lighter than the inorganic zeolites weighing 500 to 800 kg/m^3. These consist of sulphonated carbonaceous material and sulphonated styrene type resins which have excellent cation exchange properties, requiring for regeneration 2-4 kg of salt for every kg of hardness removed. These are resistant to attack by acid solutions and hence can be regenerated with acid also. They can be used for waters with a wide pH range. The loss due to attrition is negligible compared to the synthetic inorganic zeolites.

(c) Raw Water characteristics

For application to ion exchangers, the raw water should be relatively free from turbidity, as otherwise the exchange material gets a coating which affects the exchange capacity of the bed. The desirability of using filters prior to zeolite beds or resorting to more frequent regeneration would depend upon the level of turbidity. Metal ions like iron and manganese, if present are likely to be oxidized and can coat the zeolites, thus deteriorating the exchange capacity steadily since the regenerant cannot remove these coatings. Oxidizing chemicals like chlorine and carbon dioxide as well as low pH in the water will have a tendency to attack the exchange material particularly the inorganic types, the effect being more pronounced on the synthetic inorganic zeolites. Waters low in silica content are likely to pick up silica from synthetic inorganic zeolites, which has to be avoided in boiler feed water. The organic zeolites operating on brine regeneration cycle do not add any silica to the water and consequently are ideally suited for boiler feed water.

(d) Design Criteria

The design criteria for a softening system is based upon (i) the required flow rate, (ii) the influent water quality, (iii) desired effluent water quality, (iv) exchange capacity and hydraulic characteristics of the exchanger, (v) period between regenerations, (vi) type of operation, (vii) number of units required, (viii) rate, time of contact, uniformity and concentration of brine application, (ix) rate and volume of rinse and (x) quality of regenerant. A softening unit is similar to a rapid sand filter unit regarding the hydraulics and equipment.

Volume of exchange material to be used in cubic metres (E) is calculated by the formula :

$$E = \frac{QH}{1000G}$$

Where,

Q = Volume of water to be treated between regeneration, in m^3

H =Hardness of water in mg/l

G = Exchange capacity of the material kg/m^3

Generally, ion exchange beds are encased in shells, shell diameter and bed depth being adjusted to maintain a rinse rate of flow in the range of 0.15 to 0.30 m/min. The vertical units are 0.2 m to 3 metres in diameter while the horizontal ones are 3 m in diameter and 8 to 9 m long. The ion exchange bed has a depth of 0.6 m usually and is placed over supporting gravel (size depending upon composition of the exchange material but with similar specification as those for rapid gravity sand filters) of 0.30 to 0.45 m depth with an underdrain system at the bottom for collecting softened water. After the softening cycle, the softener should be backwashed for 3 to 5 minutes to loosen the exchange resin and remove particulate matter. The rate of backwash should ensure atleast 50% bed expansion. Then regeneration of the bed is carried out with brine solution. The brine distribution manifold is placed immediately above the softener bed.

Exchange capacities and the common salt requirements of Cation exchangers are presented in Table 9.4

TABLE 9.4

EXCHANGE CAPACITIES AND COMMON SALT REQUIREMENTS OF CATION EXCHANGERS

Cation Exchanger	Capacity Kg/m^3	Common Salt kg/kg exchanged
Green sand	7 - 14	3.5 – 7
Synthetic Siliceous Zeolite (inorganic)	14-37	2.5 – 3.5
Synthetic Organic	-----	-----
Sulphonated Coal	12-70	2 – 4
Resin, Polystyrene	25 – 100	2 – 4

The optimum concentration of brine for restoration of maximum exchange capacity in any resin is about 10 to 15% and the contact time for regeneration varies from 20 to 45 minutes. A dosage of salt of 15 kg/ min/m^3 of resin using 10% brine solution is usually applied at a rate of about 150 lpm/m^3 of exchanger. For sea water, about 200 to 400 lpm/m^3 of exchanger is necessary.

The total rinse water requirement is 3 to 10 m^3/m^3 of material and applied at a rate of 9 to 18 $m^3/h/m^3$ in the slow and 30 $m^3/h/m^3$ in the fast types. The rinse water is introduced through the brine distribution network or by simply flooding the unit through a hose.

The salt or brine storage tank should provide for a capacity of 24 hours or 3 successive regenerations, whichever is greater.

(e) Disposal of Spent Brine

The total waste flow from a softening unit may vary depending upon the capacity of the exchanger and the hardness removed in each cycle. The waste flow consists of a mixture of salts of sodium, calcium and magnesium with concentration as high as 9000 to 12,000 mg/l. The disposal of spent salt or spent brine poses a problem. Methods like controlled dilution, evaporation ponds, disposal at sea or brine wells can be adopted.

9.5.2.3 Combination of Lime and Zeolite Softening

For waters which contain a large carbonate hardness, a combination of lime and zeolite softening can be practised. The lime treatment, which is applied first, removes by precipitation a large part of the carbonate hardness, simultaneously decreasing the amount of dissolved solids in the water. After leaving the lime reaction tank, the water is settled and filtered and then passed through the zeolite softeners which by base-exchange remove the residual carbonate hardness and all the non-carbonate hardness.

The combination of lime and zeolite offers the following advantages :

(a) It gives a water with a lower hardness than can be obtained by lime and soda ash treatment.

(b) It reduces the amount of total dissolved solids which the zeolite treatment alone would not do.

(c) It gives a lower cost of chemicals than with lime and soda ash and possibly lower than with zeolite alone, depending on the relative costs of salt and lime.

Surveys carried out in other countries have clearly brought out the fact that the benefits of savings of soap alone justify the expenses of softening on municipal scale. There are other benefits like good public relations that add to the attractiveness of the proposition. The practice has not, however, caught up with this trend even in those countries. With greater demands of higher quality water, water softening may have to be carried out on a municipal scale also.

9.6 REMOVAL OF IRON AND MANGANESE

Appreciable amounts of iron and manganese in water impart a bitter characteristic, metallic taste and the oxidized precipitates can cause. Coloration of water which may be yellowish brown to black and renders the water objectionable or unsuitable for domestic and many industrial processes. In addition, staining of plumbing fixtures and laundered materials can also result. Carrying capacity of pipelines in the distribution system is reduced due to the deposition of iron oxide and bacterial slimes as a result of the growth of microorganisms (iron bacteria) in iron bearing water. Concentration of iron in excess of 0.2 to 0.3 mg/l may cause nuisance, even though its presence does not affect the hygienic quality of water.

9.6.1 SOURCES AND NATURE

Iron and manganese occur in certain underground waters and springs, alone or in association with organic matter, iron being generally predominant, when they are together. They could also be found in surface waters occasionally. Iron and Manganese are found in solution in water derived near the bottom of deep lakes, where reducing conditions develop. These are usually seasonal. The presence of iron can also result due to the discharge of certain industrial wastes or mine drainage.

Iron and manganese in ground waters are attributed to the solution of rocks and minerals chiefly oxides, sulphides, carbonates and silicates of these metals. This dissolution is enhanced by the presence of carbon dioxide present in groundwater.

Iron exists in water in two levels of oxidation (i) as the bivalent, ferrous iron (Fe^{++}) and (ii) as the trivalent, ferric iron (Fe^{+++}), the latter occurring generally in the precipitated form. Therefore in clear ground waters, the iron, if present, is all ferrous iron. Manganese is also found in water naturally in two oxidation states, bivalent and quadrivalent, the latter being very sparingly soluble.

Iron forms complexes of hydroxides and other inorganic complexes in solution with substantial amounts of bicarbonate, sulphate, phosphate, cyanide, or halides. Presence of organic substances induces the formation of organic complexes and chelates which increase the solubility of iron and manganese.

The terminology of organic iron and manganese is used when difficulties in oxidation are encountered. There are no analytical techniques for determination of organic iron or manganese.

Waters of high alkalinity have lower iron and manganese contents than waters of low alkalinity. If water contains significant amounts of hydrogen sulphide, little or no iron or manganese is found in solution as most of it is precipitated.

9.6.2 REMOVAL METHODS

Chemical analysis of water alone may not always provide a clue to the removal method to be adopted. Hence it is advisable that laboratory and pilot plant studies are made before any particular method is used. Oxidation by aeration or use of chemicals like chlorine, chlorine dioxide or potassium permanganate followed by filtration alone or by settling and filtration can bring about the precipitation of iron and manganese and their removal. Use of zeolites as well as catalytic oxidation also serve the purpose.

9.6.2.1 Precipitation

Iron or manganese in water in reduced form is converted to soluble ferric and manganic compounds by oxidation and these are removed by filtration alone or by sedimentation and filtration. The reaction period is about 5 minutes or less at a pH of 7 to 7.5 and 0. 14 mg of oxygen is needed to convert 1 mg ferrous iron to ferric hydroxide as indicated below.

$$4\,Fe^{2+} + O_2 + 10H_2O \Rightarrow 4\,Fe(OH)_3 \downarrow + 8H^+$$

$$4 \times 56 \text{ mg } Fe^{2+} = 2 \times 16 \text{ mg } O_2$$

$1\ mg\ Fe^{2+} \equiv 0.14\ mg\ O_2$

The rate of oxidation of ferrous iron by aeration is slow under conditions of low pH, increasing 100% for every unit rise of pH. Increased aeration time would be necessary for stripping the carbon dioxide, hydrogen sulphide etc. Addition of lime can also remove the carbon dioxide or in case where there is mineral acidity it can accomplish the raising of pH. Rates of precipitation and flocculation are accelerated in practice by contact and catalysis. Water is allowed to trickle over coke or crushed stone. The deposition of hydrated oxides of iron and manganese and bacteria on the contact media is believed to act as catalysts which accelerate the oxidation of iron.

The contact beds for deferrisation are normally 2 to 3 m deep operating at a surface loading of 40 to 70 $m^3/d/m^2$ with the contact medium of sizes 50 to 150 mm. Accumulation of iron and manganese are flushed out by rapid drainage after filling the bed to near overflow level. Sedimentation before filtration will be necessary when the iron content exceeds 10 mg/l. A settling period of two to three hours is adequate. The water has to pass through filters (gravity or pressure type) with 75 cm depth of sand or sand and anthracite. Filter rates are usually of 6 to 9 $m^3/h/m^2$

Oxidation of iron can be inhibited possibly due to the binding of ferrous iron by organic substances and ammonia which behave in a manner similar to tannic, gallic or ascorbic acids. All the organic material has to be oxidized before any perceptible oxidation of iron can be effected. Chlorination of many iron bearing waters can bring about the oxidation of the organic matter and other reducing agents facilitating the oxidation of ferrous iron. Deeper filter beds upto 2 to 2.5 m with sand size of 0.6 mm have also been used with good results. In many waters, especially containing organics, prechlorination ahead of coagulation, sedimentation and, filtration at pH values between 6.7 and 8.4 usually will ensure removal to acceptable limits.

By addition of lime to raw or pre-aerated waters, carbon dioxide content can be brought down to zero and the resulting high pH value will promote the flocculation of iron and manganese. The plants will require washing of filter medium. Necessity of washing is ascertained as and when there is overflow through the overflow pipe provided in the filter compartment of the units. The interval between successive washing varies and depends on the initial turbidity and iron content. Experience indicates a closer interval of one week for turbidity around 40 mg/l and 1-2 months for waters with low turbidities (less than 10 mg/l). Washing of filter medium involves removal of top 5 to 10 cms filter medium and washing it manually with water to free it from sediment and replace the same in position. The coke medium needs washing/replacement once in 6 to 24 months depending on the iron content in raw water.

Iron removal is also concomitant at the high pH value reached in municipal softening plants using lime.

Manganese removal requires a pH adjustment upto 9.4 to 9.6. 0.29 mg of oxygen is needed to convert 1 mg manganese

$$6\ Mn^{2} + O_2 + 6H_2O \rightleftharpoons 2\ Mn_3O_4 + 12\ H^{+}$$

$$2\,Mn_3O_4 + 2O_2 \rightleftharpoons 6\,MnO_2 \text{ (black)}$$

$$6\,Mn^{2+} + 3O_2 + 6\,H_2O \rightleftharpoons 6\,MnO_2 + 12\,H^+$$

$$1 \text{ mg } Mn^{2+} \equiv 0.29 \text{ mg. } O_2$$

Prechlorination to free residual values upto 0.7 to 1.0 mg/l will effect the oxidation and precipitation of manganese.

9.6.2.2 Contact Beds

The purpose of contact beds is to facilitate oxidation of iron or manganese through the catalytic action of previously precipitated oxides of these minerals on the gravel or ore. Superior results are claimed for the manganese ore, pyrolusite, which is an oxide of manganese. Usually upward flow at rates upto 9.6 m/h is preferred, but a lower rate may be used. Bed depth should be 1.8 m or any greater depth found necessary by pilot-plant studies. Provision must be made for the rapid draining of the beds, so as to wash excess oxides from the gravel or ore, and for the use of a hose stream for periodic cleansing of the gravel or ore. The beds are regenerated by backwashing with potassium permanganate solution when permanganate is not applied continuously to the raw water.

Contact beds of pyrolusite ore, for manganese removal without lime or potassium permanganate treatment, must be in closed structures to prevent the entrance of air. Upward flow, at rates established by pilot-plant tests, should be provided. A trial rate of 4.8 m/h with a bed depth of 1.8 m is suggested giving a contact period of 9 minutes, with a usual void volume of 40%. The effluent from such beds should be aerated in a downward flow contact led -aerator constructed to facilitate passage of air. Final filtration is needed as discussed further.

Manganese zeolite, formed by treating sodium zeolite with a solution of potassium permanganate, is an effective contact material that will remove by oxidation about 1.63 kg manganese per cubic meter of zeolite per cycle. Re-oxidation or regeneration of all material in each cycle is secured by backwashing with a solution of potassium permanganate containing about 3.26 kg of this chemical for each cubic meter of zeolite. Incomplete re-oxidation will result in the passage of manganese through the contact bed. The need to regenerate may be anticipated by computing the volume of the raw water which contains 1.63 kg manganese per cubic metre of zeolite. For example, a water with a content of 1 mg/l manganese will contain 1 kg/m^3. Then a contact bed with a volume of say, 4 m^3 would treat 4 x 1.63 x 1.0 = 6.52 m^3 of this water before regeneration is necessary. A solution of potassium permanganate such as 10 kg/m^3, would be used to re-oxidize the bed. Then 1.3 m^3 of this solution would be needed per cycle: (3.26/ 10) x 4 = 1.3.

Chlorine dioxide and potassium permanganate which are strong oxidants are employed chiefly for manganese bearing waters. The pH range at which manganese dioxide is oxidized rapidly is quite broad.

9.6.2.3 Zeolite

The method is applicable if the iron is present in the reduced state and in a soluble form in the raw water. Such waters are encountered from the bottom strata of deep reservoirs or ground waters. It is usual to limit the application of this process to water having not more than 1 mg of iron or manganese for every 30 mg of hardness upto a maximum of 10mg/l of iron or manganese.

The process consists of percolation of the water through the bed of the zeolite which takes up the iron and manganese by a process of ion exchange. The base exchanger may be of the siliceous, carbonaceous or synthetic resin type. Air should be excluded from the system to prevent deposition of colloidal oxides on the ion exchange material. Therefore, air lifts, open tanks or pneumatic tanks should not be used preceding the ion exchanger. If the water is to be softened also, then the zeolite process offers a very simple method of iron and manganese removal as it can be carried out under pressure and therefore usually obviates the necessity for double pumping such as is required in most other processes. In fact, many zeolite plants have been installed principally for iron and manganese removal, the softening being of secondary importance. The removal of iron and manganese is almost complete, the exhausted bed of ion exchange material being regenerated with salt solution.

9.6.2.4 Catalytic Method

This method is of limited application but is of value if the content of iron and manganese is low and if it is desirable to treat the water under pressure. It is applicable in the case of clear deep well waters where the iron is held in solution by the carbon dioxide. In municipal use, it is usual practice to restrict the use of this method to waters whose content of iron or manganese is not greater than 1 mg/l. For household use or for the rather smaller plants, it may be used with waters containing upto 10 mg/l iron or manganese. The removal of iron and manganese is accomplished without affecting the hardness of the water as this process is entirely one of oxidation and filtration and does not involve base exchange. The method consists of percolating the water through suitable contact materials which oxidize the iron and the manganese. These contact materials, which are sold under various names, are made by treating a siliceous base exchange material successively with solutions of manganese chloride and potassium permanganate. They may be housed in a separate filter or a layer of this material may be sandwiched in the sand bed of a pressure filter, By percolation through this bed, iron and manganese are oxidized and also filtered out. At intervals, the filter has to be backwashed to remove the deposits. The backwash rates are generally of the order of 21 $m^3/h/m^2$. When the bed loses its capacity for oxidation of iron and manganese, it can be regenerated by treatment with potassium permanganate solution.

9.6.3 Simple Techniques for Iron Removal in Rural Areas for Small Communities

For small communities in rural areas, where the density of population is low, piped distribution is costly and trained personnel for the operation and maintenance not available; a simple and inexpensive treatment unit for the removal of iron is suggested so that the difficulties of operation and maintenance can also be minimized. In a rural water supply scheme to treat raw water containing free carbon dioxide and dissolved iron, the units include, hand-pumps, tray-aerators, sedimentation basin and sand filters.

Where the source is a well or a sump and the water consumption rate is in the order of 40 lpcd and where hand pump is used, a tray type aerator with two trays operated at an aeration rate of 1.26 $m^3/m^2/hr$ are employed and the water aerated. Then the water is settled in a sedimentation basin having a detention period of 3 hours and the clarified water passed through a rapid sand filter having a depth of 0.3 m supported by gravel 3-6 mm in size and 0.1 m deep. The effective size of sand is 0.30-0.45 mm and its uniformity coefficient 2-3. Sand is cleaned by manual scraping. Provision could also be made for adding sodium carbonate wherever essential. Reference may be made to the type design for iron removal plant given in Appendix 9.2

9.6.3.1 Package Iron Removal Plants For Hand Pump

NEERI has designed package iron removal plants having different capacities of 0.5 m^3/hr, 1.0 m^3/hr 1.5 m^3/hr and 2 m^3/hr depending upon requirement of treated water and discharge of hand pump. The plants are designed in rectangular/circular shapes having aeration chamber, collection chamber, settling chamber and filter. The settling chamber is provided with plate settling device to enhance settling and reduce the detention time thereby reducing the dimension of settling chamber. The aeration chamber contains media of size 2.0-5.0 cm gravel/ stone chips to increase the surface area of air-water interface. The iron contaminated water trickles over aeration media through spraying device. The aerated water flows through pores over baffle plate to collection chamber to settling chamber to filter. The filter bed of 20 cm depth contains sand media of size 0.8-1.4 mm, supported by 5 cm depth gravel of size 0.8-1.0 cm. The treated water is taken out from tap attached to it. The sedimentation chamber having a detention period of one hour and is provided with two plate settlers at 45° angles as shown in drawing of 1 m^3/hr plant (Fig. 9.1). The filter is cleaned by making backwash connection with hand pump, scraping sand bed manually and opening the sludge scouring valve. The hand pump is operated with possible high speed till clean water has accumulated over the sand bed.

9.6.4 Iron Removal Plants for Large Communities

When the question of iron removal is under consideration for community water supply, it is important to decide and cover what other treatment of the water, if any, is necessary or desirable. Considerable free carbon dioxide and toxic substances are usually present in ferruginous waters. Hence, it is not advisable to remove iron alone leaving the free carbon dioxide which may cause corrosions of mains and pipes. The means by which iron, free carbon dioxide and other toxic substances are removed from water in community systems consists substantially of their oxidation and removal of free carbon dioxide, followed by precipitation and its separation by sedimentation and/ or filtration. Aeration may suffice for the preliminary precipitation but may not be adequate when concentrations are high and pH correction may be required by addition of lime. The community water supply scheme makes provision to meet these requirements and comprises raw water storage tank, cascade tray aerators, chemical dosers, sedimentation basin, filtration and disinfection.

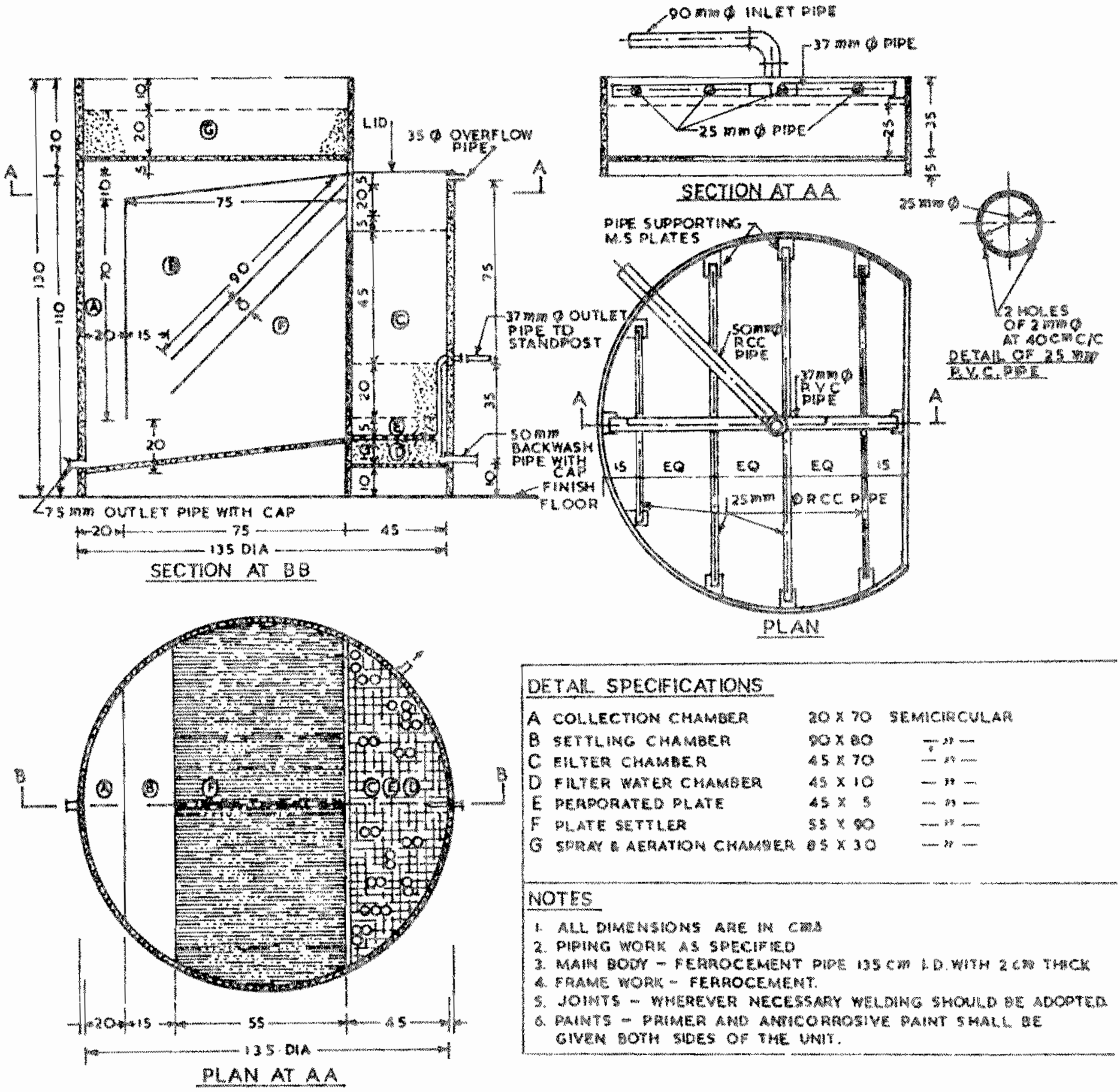

FIG. 9.1 IRON REMOVAL PLANT FOR 1 M^3/HY

Tray aerators are commonly used for aerating water. The trays are designed for an aeration rate of 1.26 $m^3/m^2/hr$ and spaced at intervals of 1 m. Then the water is settled in a sedimentation basin having a detention period of 2.5 hours. The clarified water is filtered through a rapid sand filter having sand of effective size 0.6-0.8 mm and uniformity coefficient 1.3 with an effective depth of 1.2 m. The head of water above sand is 1.35 m and the rate of filtration 5 $m^3/m^2/h$, The minimum backwash rate is 35 $m^3/m^2/h$ and the total head required for filter wash is 12 m.

Type designs for iron removal plants for 5, 10 and 15 m^3/hr of flow are given in Appendix 9.3 along with drawings.

The sand is supported over a gravel layer of depth 0.39 - 0.62 m, and it is arranged as follows :

size	depth
65 - 38 mm	13 – 20 cm
38-20 mm	8 – 13 cm
20-12 mm	8 – 13 cm
12-5 mm	5 - 8 cm
5-2 mm	5 – 8 cm

Power shut-downs are frequent and rarely more than two hours supply is available in the morning and evening in rural areas. Hence raw water pumping hours can be assumed to be 2 hours in the morning and 2 hours in the evening. During these 4 hours of pumping the total daily requirements of water are to be pumped to the raw water elevated storage tank. The treatment plant has therefore to be designed to operate under gravity from the raw water storage tank taking these facts into account. To avoid extra cost for additional over-head tank for filtered water, the filtered water from the pump-well could be directly pumped for the distribution. The distribution of treated water would follow the same time schedule as for pumping raw water. Backwashing of the sand filter would be carried out by using raw water from the overhead tank.

9.7 DEFLUORIDATION OF WATER

Excessive fluorides in drinking water may cause mottling of teeth or dental fluorosis, a condition resulting in the discoloration of the enamel, with chipping of the teeth in severe cases, particularly in children. In Indian conditions where the temperatures are high, the occurrence and severity of mottling increases when the fluoride levels exceed 1.0 mg/l. With higher levels, skeletal or bone fluorosis with its crippling effects are observed. The chief sources of fluorides in nature are (i) fluorapatite (phosphate rock), (ii) fluorspar, (iii) cryolite and (iv) igneous rocks containing fluorosilicates. Fluorides are present mostly in ground waters and high concentrations have been found in parts of Andhra Pradesh, Bihar, Gujarat, Haryana, Karnataka, Kerala, Madhya Pradesh, Maharashtra, Punjab, Rajasthan and Tamil Nadu in the country. While majority of values range from 1.5 to 6 mg/l some values as high as 16 to 18 mg/l and in one solitary instance, even 36 mg/l have been reported.

9.7.1 REMOVAL METHODS

The removal of excessive fluorides from public water supplies or individual water supplies is justifiable solely on public health grounds. This is a problem particularly in rural areas and hence the accent has to be on simplicity of operation, cheapness and applicability to small water supplies. The methods use fluoride exchangers like tricalcium phosphate or bone meal, anion exchangers, activated carbon, magnesium salts or aluminium salts.

9.7.1.1 Fluoride Exchangers

Degreased and alkali treated bones possess the ability to remove fluorides but have not been used on a plant scale. Bone charcoal prepared by controlled combustion of bones under limited supply of air in the presence of catalysts when treated with alkali or phosphate has been found to be useful.. One cubic metre of bone charcoal is capable of removing 1.1 kg of fluoride from a water with fluoride content upto 6.0 mg/l. The spent material can be regenerated with mono or trisodium phosphate. Tricalcium phosphate in powdered form can also be used but it has a lesser capacity of 0.7 kg of fluoride/m^3. The spent material is regenerated by treatment with 1% alkali solution and rinsed with dilute hydrochloric acid.

9.7.1.2 Anion Exchangers

Fluorides can also be removed by anion exchange resins-strongly basic formaldehyde resin quaternary ammonium type-in hydroxide or chloride form. But their efficiency is lowered in the presence of other anions like bicarbonates, hydroxides and sulphates in the water.

9.7.1.3 Activated Carbon

Activated carbons have also been known to have the capacity for removal of fluorides. An activated carbon for fluoride removal has been developed in India by carbonising paddy husk or sawdust, digesting under pressure with alkali and quenching it in a 2% alum solution. This could remove 320 mg of fluoride per kilogram of the dry material. The spent material could be regenerated by soaking it in a 2% alum solution for 14 hours. The attrition and hydraulic properties of the carbon are however poor.

A granular ion-exchange material Defluoron 2, which is a sulphonated coal operating on the aluminium cycle has been developed in the country. The capacity of the material is estimated to be 500 gm of fluorides/m^3 with test water containing 5 mg F/l and 150 mg/l alkalinity. The regeneration is carried out by means of a 2.5% alum solution, with replacement of two bed volumes. A flow rate of 4.8 m^3/ m^2/hr of bed area is adopted. The rinse water requirements after regeneration are 9-12 m^3/ m^2/hr for a maximum duration of 10 minutes. The medium has a life of three years.

High alkalinity of the water considerably lowers the capacity as well as the efficiency of the bed. Hydroxyl alkalinity beyond 5 mg/l has a deleterious effect on the removal efficiency of the medium. The efficiency of the medium falls down by 30% when hydroxyl alkalinity becomes 25 mg/l.

Treatment cost using Defluoron-2 varies from Rs. 1.0 to Rs. 5.0 per 1000 litres of water treated, depending upon the initial fluoride concentration and the alkalinity of water.

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9.7.1.4 Magnesium Salts

Excess lime treatment for softening effects removal of fluoride due to its adsorption by the magnesium hydroxide floc. The fluoride reduction is given by the following expression :

$$\text{Fluoride reduction} = 7\%\ \text{initial fluoride conc.} \times \sqrt{\text{magnesium removed}} \qquad (9.1)$$

Sizeable fluoride removals are possible only when magnesium is present in large quantities which may not always be the case and magnesium have to be supplemented in the form of salts. The process is suitable only when the water is being softened.

Magnesia and calcinated magnetite have also been used for removal of fluoride from water. The study established the following empirical relationships for amounts of MgO which are required to obtain 1 or 2 mg F/l in treated water.

(a) MgO required to obtain 1 mg F/l in treated water (F_0 > 2 mg F/l)

$$= 1.71(13.33)\left(1 - \frac{1}{F_o}\right)^2 + 160 \text{ x basicity of raw water (me/l)} \qquad (9.2)$$

(b) MgO required to obtain 2 mg F/l in treated water (F_O > 3 mg F/l)

$$= 1.71(13.33)\left(1 - \frac{1}{F_O}\right)^2 + 120 \text{ x basicity of raw water (me/l)} \qquad (9.3)$$

F_O represents the fluoride concentration in the raw water. The pH of the treated water was always beyond 10 and its correction by acidification was essential, adding to the complexity of operations and control.

9.7.1.5 Aluminium Salts

Aluminium salts like filter alum and activated aluminium and alum treated cation exchangers have shown beneficial effects. Filter alum during coagulation brings about some removal of fluorides from water. The removal efficiency is improved when used along with a coagulant aid-like activated silica and clay. 300 to 500 mg/l of alum is required to bring down fluoride from 4.0 mg/l to 1.0 mg/l while with coagulant aid, the fluorides were reported to be reduced from 6.0 mg/l to 1.0 mg/l with alum dose of only 100 mg/l.

Alum treated polystryene cation exchangers and sulphonated coals have also been used successfully. A cation exchanger prepared from extract of Avaram bark and formaldehyde when soaked in alum solution has been found to have good fluoride removal capacity (800 mg/kg).

Calcinated or activated alumina in granular form can be used for fluoride removal and the spent material regenerated with alkali, acid or by both alternately (removal efficiently 1.2 kg of fluoride/m^3). A dilute solution of aluminium sulphate used as the regenerate for the spent material makes the alumina four times more efficient.

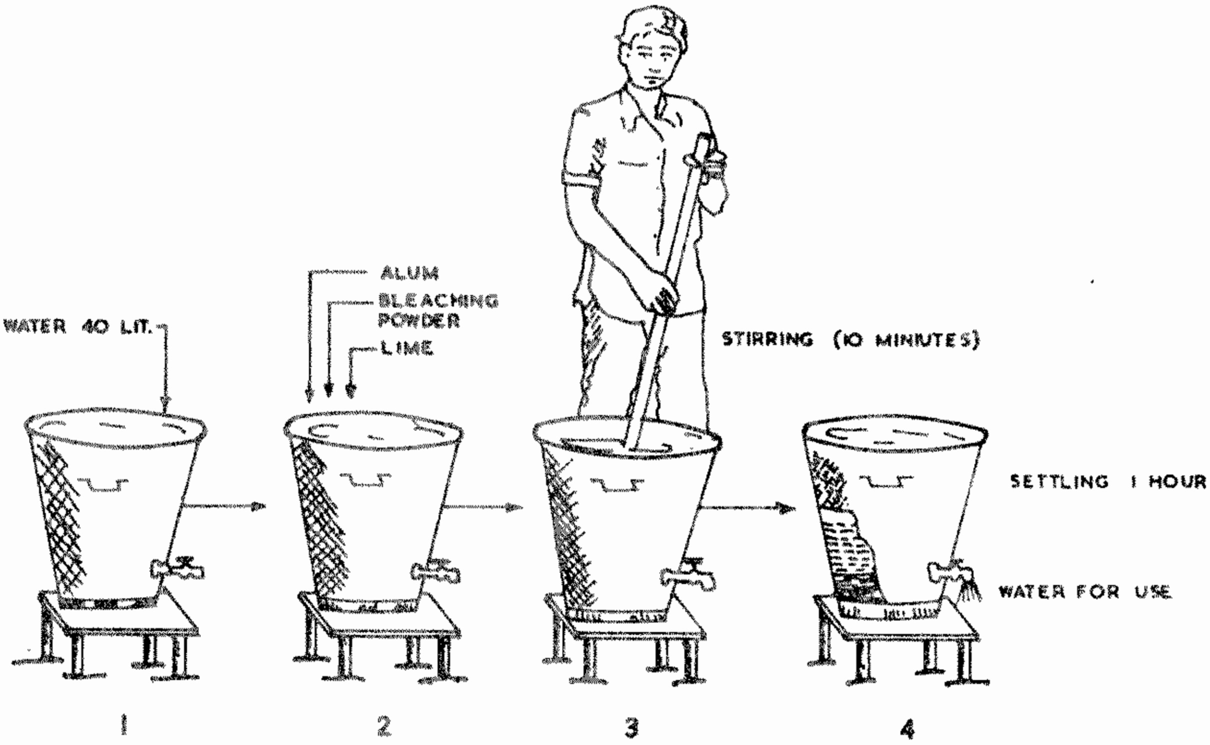

FIGURE 9.2: DEFLUORIDATION AT DOMESTIC LEVEL

9.7.2 Simple Method of Defluoridation

Defluoridation is achieved either by fixed bed media which could be regenerated or by the process of precipitation and formation of complexes. A simple method of defluoridation is employed in the Nalgonda Technique. It involves the use of aluminium salts for the removal of fluoride. The Nalgonda Technique employs either the sequence of precipitation, settling and filtration or precipitation, floatation and filtration and can be used for domestic as well as community water supply schemes.

(i) Domestic Treatment-Precipitation, Settling and Filtration

Treatment can be carried out in a container (bucket) of 40 l capacity with a tap 3-5 cm above the bottom of the container for the withdrawal of treated water after precipitation and settling (Fig. 9.2). The raw water taken in the container, is mixed with adequate amount of lime or sodium carbonate, bleaching powder and aluminium sulphate solution, depending upon its alkalinity and fluoride content. Lime or sodium carbonate solution is added first and mixed well with water.

Alum solution is then added and the water stirred slowly for 10 minutes and allowed to settle for nearly one hour. The supernatant which contains permissible amount of fluoride is withdrawn through the tap for consumption. The settled sludge is discarded. The amount of alum in ml to be added in 40 litres of water at various alkalinity and fluoride levels is given in Table 9.5.

TABLE 9.5

ALUM DOSE FOR DIFFERENT FLUORIDES AND ALKALINITY LEVELS

Test water Fluoride	Test water Alkalinity, mg $CaCO_3$/l							
mg F/l	125	200	300	400	500	600	800	1000
2	60	90	110	120	140	160	190	210
3	90	120	140	160	205	210	235	310
4		160	165	190	225	240	275	375
5			205	240	275	290	355	405
6			245	285	315	375	425	485
8					395	450	520	570
10							605	675

(ii) Fill and Draw Type for small community

This is also a batch method for communities upto 200 population. The plant comprises a hopper bottom cylindrical tank with a depth of 2 m equipped with a hand operated or power driven stirring mechanism (Fig. 9.3). Raw water is pumped or poured into the tank and the required amounts of bleaching powder, lime or sodium carbonate and alum added with stirring. The contents are stirred slowly for ten minute and allow to settle for two hours. The defluoridated supernatant water is withdrawn to be supplied through standposts and the settled sludge is discarded.

The notable features are:

(a) With a pump of adequate capacity the entire operation is completed in 2-3 hours and a number of batches of defluoridated water can be obtained in a day.

(b) The accessories needed are few and these are easily available (these include 16 1 buckets for dissolving alum, preparation of lime slurry or sodium carbonate solution, bleaching powder and a weighing balance).

(c) The plant can be located in the open with precautions to cover the motor.

(d) Semi-skilled labour can perform the function independently.

(iii) Fill and draw type (electrically operated)

The Fill and Draw type vertical unit comprises cylindrical tank of 10 m^3 capacity with dished bottom, inlet, outlet and sludge drain. The cylindrical tank will have sturdy railings, etc. Each tank is fitted with an agitator assembly consisting of (i) 5 HP drip proof electric motor; 3 phase; 50 Hz; 1440 RPM with 415 V ±6% voltage fluctuation, and (ii) gear box for 1440 RPM input speed with reduction ratio 60:1 to attain an output speed of 24 RPM, complete with downward shift to hold the agitator paddles. The agitator is fixed to the bottom of the vessel by sturdy, suitable stainless steel supporting bushings.

The scheme comprises tanks of 10 m^3 capacity each, a sump well and an overhead reservoir. Typical layout for system with two units in parallel for treating water for 1500 population at 40 lpcd is shown in Fig. 9.4. Raw water is pumped into the units and treated by Nalgonda Technique. The treated water collected in a sump is pumped to an overhead tank, from where the water is supplied through stand posts.

Approximate alum doses (mg/l) required to obtain permissible limit (1 mg F/l) of fluoride in water at various alkalinity and fluoride levels are given in Table 9.6.

TABLE 9.6

ALUM DOSE FOR DIFFERENT FLUORIDES AND ALKALINITY LEVELS

Test water Fluoride	**Test water Alkalinity, mg $CaCO_3$/l**							
mg F/l	125	200	300	400	500	600	800	1000
2	143	221	273	312	351	403	468	520
3	221	229	351	403	507	520	585	767
4		403	416	468	559	598	689	936
5			507	598	689	715	884	1010
6			611	715	780	936	1066	1209
8					988	1118	1300	1430
10							1508	1690

Note : *To be treated after increasing the alkalinity with lime or sodium carbonate.

(iv) Precipitation, Floatation and Filtration

Domestic treatment is achieved using a 100 l capacity batch type dissolved air floatation cell with hand operated pressure pump. The pump and cell form a compact dissolved air floatation defluoridation system.

Raw water in the cell is mixed with alkali and aluminium salts. A small quantity of air-water mix from the pressure pump is allowed into the cell. The precipitate with fluoride lifts to the top and floats. The treated water is collected in a bucket filtered through a sand filter. Using this cell, 100 l water is available for use in 20 minutes (Fig. 9.5.)

The same principle of floatation is extended to a 500 l capacity dissolved air floatation cell to obtain nearly 1 m^3 treated water per hour for small communities.

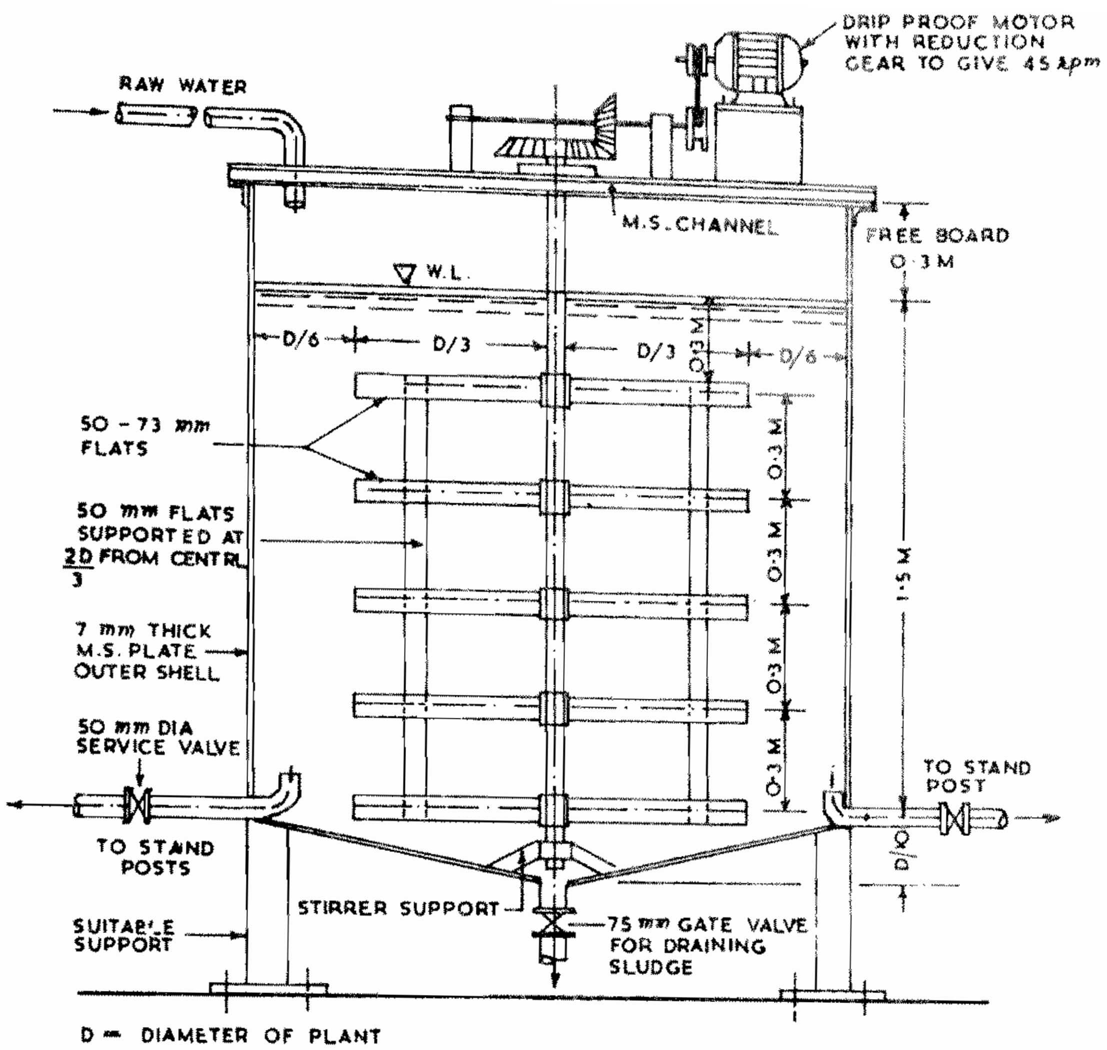

FIGURE 9.3: FILL AND DRAW TYPE DEFLUORIDATION PLANT FOR POPULATION UPTO 200 @ 40 lpcd

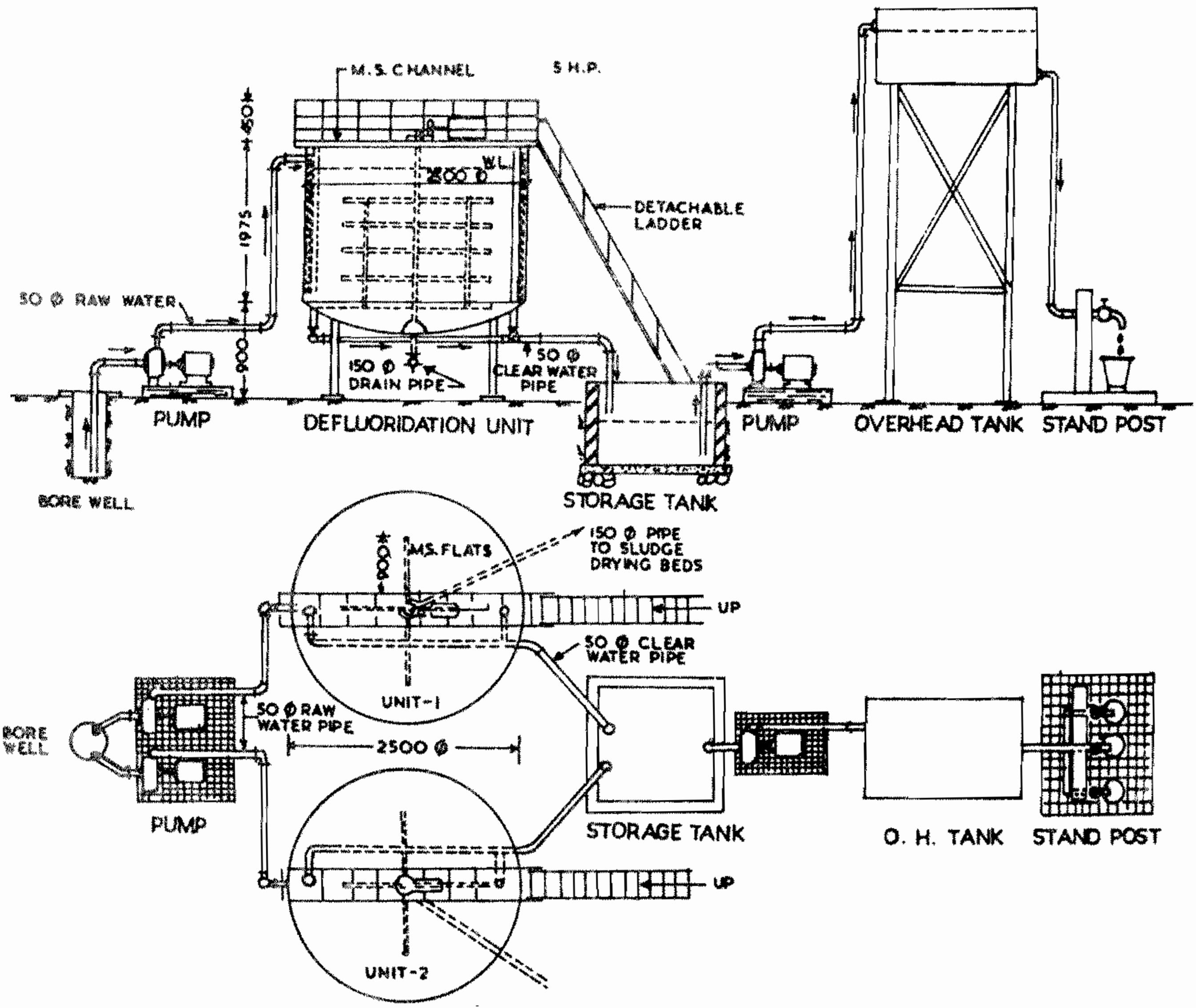

FIG. 9.4 FILL AND DRAW TYPE DEFLUORIDATION SYSTEM FOR RURAL WATER SUPPLY

9.7.2.1 Mechanism Of Defluoridation by Nalgond a Technique

The chemical reactions involving fluorides and aluminium species are complex. It is a combination of polyhydroxy aluminium species complexation with fluoride and their adsorption on polymeric alumino hydroxides flocs. Besides fluorides, turbidity, colour, odour, pesticides and organics are also removed. The bacterial load is also reduced significantly. All these occur by adsorption on the floc surface. Lime or sodium carbonate insures adequate alkalinity for effective hydrolysis of aluminium salts so that residual aluminium does not remain in the treated water. Simultaneously disinfection is achieved with bleaching powder and this keeps the systems free from undesirable biological growth.

9.7.2.2 Rural Water Supply Using Precipitation, Settling, Filtration Scheme Of Nalgonda Technique-Continuous Operation

This scheme intends to treat the raw water for villages and includes channel mixer, pebble bed flocculation, sedimentation tank and constant rate sand filters. The designs of entire water facilities are available for 500, 1000, 2000 and 5000 populations. The scheme is gravity operated except the filling of the overhead tank and delivery from treated water sump. Channel mixer is provided for mixing lime slurry or sodium carbonate solution and aluminium salts with the raw water. Pebble bed flocculation is used in place of conventional flocculation in order to avoid the dependence on electric power supply. The scheme envisages power supply for 2 hours each during morning and evening for filling the over head tank and for supply of treated water. The basis of design of various units are given below:

(i)	*Water consumption*	70 lpcd
(ii)	*Flash mixing-detention period, velocity to be maintained*	30 secs.
(iii)	*Pebble bed flocculator*	
	detention period (considering 50 % voids)	30 minutes
	size of media	20-40mm
	depth of media	1.2m
	rate of backwash	0.5m/min
(iv)	*Sedimentation*	
	liquid depth	3m
	weir loading rate	$< 300\ m^3/m/d$
	surface overloading rate	$< 20\ m^3/\ m^2/d$
(v)	*Sand gravity filter*	
	depth of water over sand	2m
	rate of filtration	$5\ m^3/\ m^2/h$
	head required for backwashing filter	12m
	minimum backwash rate	36m/h
	gravel depth	0.45m
	effective size of sand	0.6mm to 0. 8mm

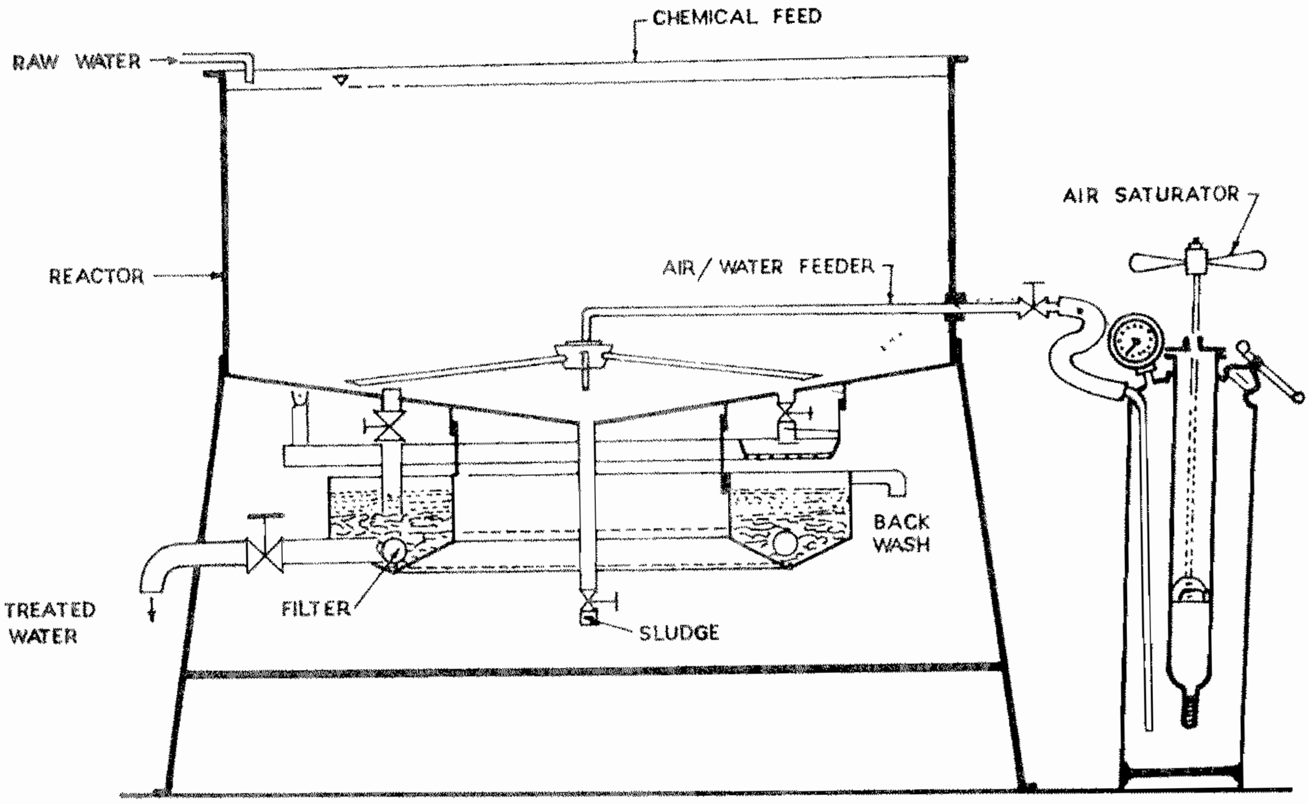

FIGURE 9.5: MUSCLE-POWER DISSOLVED AIR FLOATATION SYSTEM FOR WATER TREATEMENT

The size of all units, viz., overhead tank, channel mixer, pebble bed flocculator, sedimentation tank, sand filter and underground treated water storage tank are based on these design considerations for populations 500,1000,2000 and 5000. Layout plan and sectional elevation for treatment plant of Nalgonda Technique are given in Fig. 9.6

Nalgonda Technique has several advantages over the fixed bed ion-exchange processes. It does not involve regeneration of media and employs chemicals which are readily available and easy to operate and maintain using local skills. Colour, odour, turbidity, bacteria and organic contaminants also get removed simultaneously. The sludge generated is convertible to alum for use in removal of excess turbidity of surface waters.

9.8 DEMINERALISATION OF WATER

Conventional methods of water treatment do not materially change the mineral content of water. Base exchange softening merely converts the calcium and magnesium salts to the corresponding sodium salts. Lime softening causes a slight decrease in the contents of total solids but does not bring about any decrease in the content of sodium chloride or sulphate. Hence these methods are not effective in converting brackish water into a potable one. For providing a potable supply in brackish water area, the least mineralized water source could be prospected. When potable water is unavailable some method of treatment has to be adopted. Thus ships on the high seas as well as lifeboats are provided with stills for manufacturing distilled water. Distillation of seawater has also been adopted during the war in isolated atolls which had to be occupied.

9.8.1 DISTILLATION

Of the processes of removing water from saline solutions, distillation is the oldest and in terms of established plants, the most productive. It differs from the other processes by its passage of water through the vapour phase. The plant design is directed to tapping the most economic source of heat energy and exploiting the most efficient processes of heat transfer,

While relatively small quantities of water are to be distilled, straight or single-effect distillation is preferred because of the simplicity of operation and the lower capital cost of the installation. With larger outputs improvement in efficiency acquires much greater importance because of the much higher rates of evaporation involved and the need for the highly efficient heat transfer systems. Problems of scale formation also play a significant role.

Performance of an evaporator plant is measured by the specific heat consumption, i.e. the number of kilocalories required to produce one kilogram of distillate. Distillation plants are generally better for lower values of specific heat consumption. The introduction of the flash evaporator has helped in better economics of heat recoveries and more efficient plants can be built more cheaply. It is only in such situations where natural gas or fuel is available cheaply that low thermal performance evaporators can be used with the resultant saving in capital cost.

9.8.1.1 Solar Stills

Solar energy can be harnessed by the use of a system of mirrors following the path of the sun to focus the sunlight on sheets of water. In one of the popular methods, the salt water trickles down to trays mounted on an inclined compartment provided with glass sides and a heat insulated back which screens the condensing chamber from the sun. Since the focussing mirrors form an important element in the cost of the stills, the development of cheaper non-focussing types of mirrors and use of inexpensive materials of construction have been resorted to. In basin solar stills, a commonly used design, salt water tanks, filled either by gravity or by stainless steel impeller pumps, feed the solar still whose cover is at a shallow angle of 10° to 18° with the glass panes tightly sealed to the holding frame and the joints between the still cover and the vertical walls perfectly tight. The rate of feed to still should be such that for each 7.6 litres of salt water, 3.7 litres of fresh water is obtained and 3.7 litres of brine is discarded. The collecting troughs at the foot of the still cover must be constructed so that water will drain freely to the pipe which carries the distillate to the fresh water tank but preventing the entry of any contaminated water either from the roof or the ground in which it is constructed. In addition to the fresh water tank, it is good practice to construct additional distilled water storage so as to balance out the fluctuations between production and demand.

By their very nature, still covers are ideal for collection of run off of rain water and every advantage should be taken of the available rainfall by diverting in to the fresh water tank after disinfection. Such an addition can be substantial in areas, as for example, where annual rainfall is of the order of 30 cm and a still is so arranged as to recover 70% of it. The increase

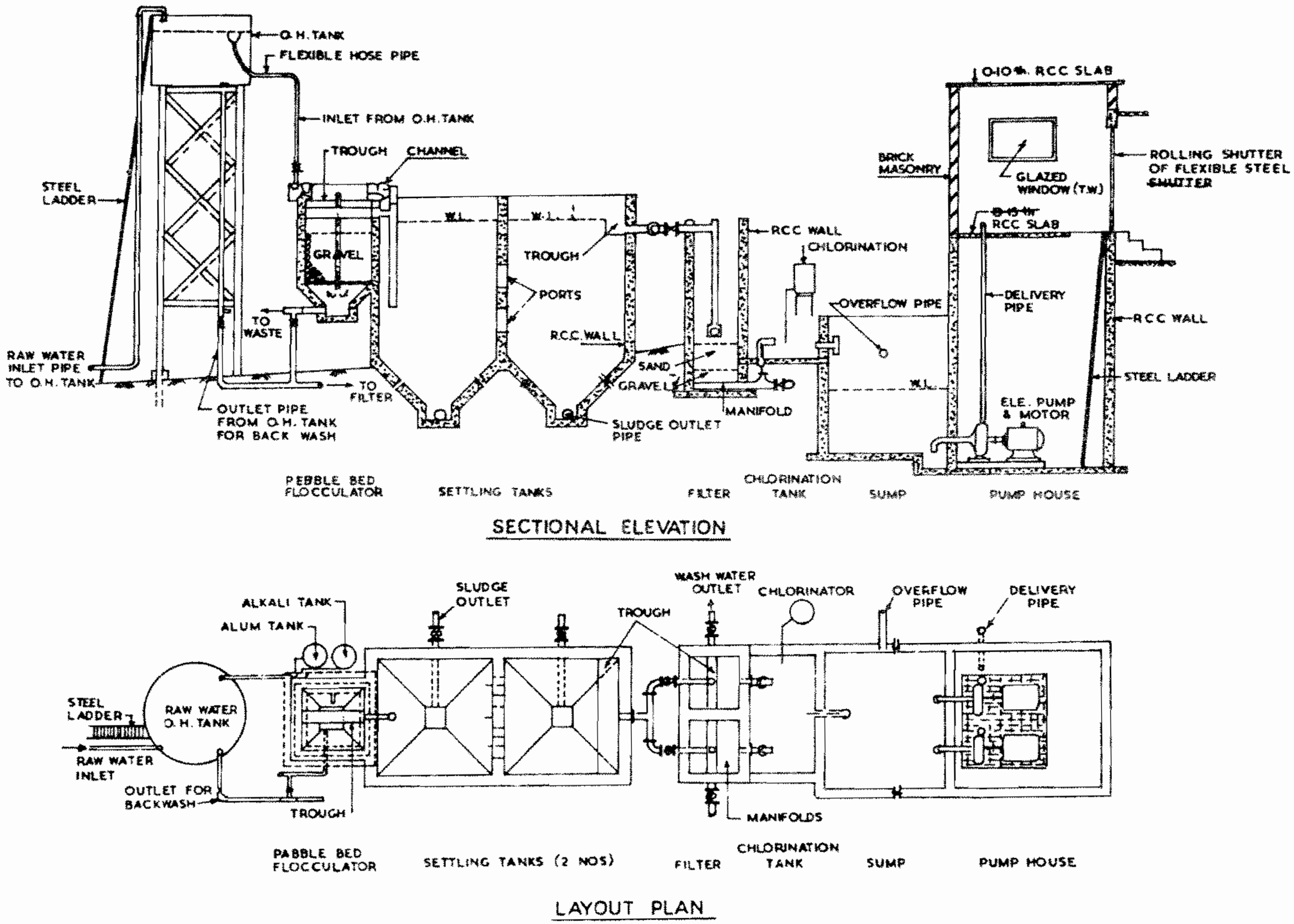

FIG. 9.6 DEFLUORIDATION PLANT USING NALGONDA TECHNIQUE

per square metre of still area is about 200 liters per year. The efficiency of a solar distiller is the condensed water actually produced divided by the water which could theoretically be evaporated by all the solar energy reaching the outer cover.

In general, wherever skies are generally clear, solar distillation is feasible upto 40° latitude, where 1000 kW/ m^2 of energy from the sun in each year can be available, the solar radiation being more important than the mean ambient temperatures and the wind factors being negligible except as they relate to stresses upon solar distillation structures. The production of water by still varies from month to month and even day to day depending upon the solar radiation available. The size of still is often to be designed on the basis of the least productive month. Yields of about 1 m^3/ m^2/year have been adopted for some of the bigger stills constructed and used successfully. The still area needed is given by the expression:

$$Q = 6.008 \times 10^{-3} \times S \qquad (9.4)$$

Where

Q = Output per square metre of still area in lpd and

S = Insolation or solar radiation in calories/ cm^2/day

Values typical for India for various latitudes are given in Appendix 9.4

The best situations for the use of solar distillation are the isolated areas and certain regions where fresh water is unobtainable, solar intensities are high, fuel resources are meager and industrial development is poor.

9.8.1.2 Single-Effect Distillation

The sea water is boiled in a vessel, using steam as the heating medium. The vapour is condensed by heat extraction to a cooling supply of sea water, part of which forms the feed to the plant.

It is not useful to install liquid/liquid heat exchangers to recover heat from the exit brine and exit distillate. The vapor produced has to be condensed. Any recovery of heat could only be used to heat the feed water, and if this were done, the circulating water supply to the condenser would need to be increased.

9.8.1.3 Multiple-Effect Evaporation

Each component unit of a multiple-effect evaporator is maintained in series at slightly lower pressure and temperature in order to permit the steam produced in one effect to serve as the source of heat in the next. Weight for weight, the amount of product water then approximates the number of effects. It has been computed that the quantity of water that can be evaporated by one kg of steam in single double and triple effect evaporations are in the ratio of 0.9, 1.7 and 2.5 respectively.

(a) Multi-Stage Flash Evaporation

This is also accomplished at successively lower pressure and temperatures. The multistage flash systems is logically related to the multiple effect system by extending the preheaters to full condensation duties and omitting all evaporation heating surface entirely, so that all vapour is obtained by flashing. The incoming water is warmed by the heat of condensation and only a small amount of heat energy is required to flash the preheated water in the reduced-pressure stage into steam. Specific heat consumption values as high as 110 are possible.

(b) Low Temperature Flash Evaporation

This method has for its object, the exploration of the possibility of utilizing the energy in streams of warm water from power plant, oil refineries and industrial plants as well as from naturally occurring sources. The studies show that this method for warm saline waters is theoretically sound and technically feasible.

(c) Vapour Compression Process

This process relies on mechanical compression of the vapour to boost its temperature high enough to supply through its own condensation the heat necessary to evaporate the feed water. Once started, this process does not draw upon further heat energy but only upon mechanical energy.

Steam at 100^0 C is compressed so that its temperature is raised to about 105 °C and this compressed steam is used to raise the temperature of the feed water to the boiling point. Vapour compression distillation improves the efficiency of the reuse of the latent heat of steam. Heat is required only for the initial production of vapour. Thereafter the heat derived from the mechanical energy developed by the motor that drives the compressor may supply all the needs of energy. This method has been found to be remarkably efficient. Heat transfer coefficients can further be improved 4 to 6 times by making a thin film of the water pass rapidly over a rotating surface. The rotor surface showed no scale or corrosion and the mechanism appears to be self-cleaning.

Because of high cost of the compressor, the expected over-all benefit of vapour compression as far as cost is concerned is not good. However, there are many special applications, particularly in small capacity plants, where considerations other than cost determine that the vapour compression process is most suitable and convenient.

(d) Critical Pressure Distillation

The principle of this method is that by operating at pressures in excess of 250 kg/ m^2 and temperatures greater than 370°C, the density difference between the liquid and vapour phases is made relatively small so that the size of the vapour handling equipment can be greatly reduced. The main difficulties in this process are the rapid building up of scale and the need for developing materials of construction which can withstand these elevated temperatures and pressures.

(e) Vapour Reheat Distillation

This process is similar, in several respects, to multi-stage flash evaporation. In this system deaerated sea water enters the system and passes through a heat exchanger counter-current to hot fresh water. The temperature is then raised with heat from an external source (the prime energy supply). The hot sea water then cascades through a series of flash chambers counter current to a stream of fresh water flowing in open channels. In each stage, some sea water flashes to form steam, which condenses in the stream of fresh water. As a result, sea water is cooled and fresh water is heated. Hot fresh water leaving the highest pressure stage is used to heat in coming sea water. Part of the cooled fresh water is recycled to the lowest pressure stage; the rest is product.

In most processes involving sea water distillation, scaling limits the maximum temperature in the systems. In the vapour reheat system, the absence of heat-transfer surfaces and reduction of scaling problems removes this limitation.

9.8.2 FREEZING

Water can be transposed from saline water to the solid phase as ice. The fact that the latent heat of fusion, viz., 80 Kcal/kg is small compared to the latent heat of vaporization is taken advantage of in this process. However, even though the ice crystals formed constitute essentially pure water, the yield of product water is decreased because some of it is used to wash salt from the ice surfaces and heat is required to melt the ice crystals. As in distillation, countercurrent operation conserves heat energy in this system also. By cooling the feed water to the freezing point before a refrigerant is evaporated in direct contact with the feed and by countercurrent washing and melting of the ice crystals, maximum economy is effected.

(a) Contact Freezing

This makes use of two heat-transfer circuits of recycling hydrocarbons. The first circuit absorbs heat from the incoming salt water, transfers it in part to the fresh water and loses it in part to the waste brine. The second circuit vaporizes the liquid hydrocarbon in contact with the salt water to freeze it; the vapour is then compressed and the heat energy released is used to melt the ice. The vapour separating from the fresh water is repumped through the freeze chamber.

(b) Eutectic Freezing

This operates at the eutectic temperature of the incoming water. Down to the eutectic point only ice is formed. At the eutectic point, ice crystals nucleate and grow independently of salt crystals and other substances in the water, thus permitting separation. Further removal of heat does not continue to lower the temperature.

9.8.3 SOLVENT EXTRACTION

Organic solvents partially miscible with water can be used to extract the fresh water leaving behind a more concentrated salt solution. The solvent fresh water phase can be separated out from the concentrated salt solution and distilled to yield fresh water.

9.8.4 OSMOSIS

Certain natural and synthetic membranes have the property of permitting the solvent (water) to get through them but not the solute. Such semipermeable membranes permit the separation of solute from solvent. This phenomenon is known as Osmosis.

(a) Reverse Osmosis (RO)

Reverse Osmosis is a membrane permeation process for separating relatively pure water (or other solvent) from a less pure solution. The solution is passed over the surface of an appropriate semi permeable membrane at a pressure in excess of the effective osmotic pressure of the feed solution. The permeating liquid is collected as the product and the concentrated feed solution is generally discarded. The membrane must be highly permeable to water, highly impermeable to solutes, and capable of withstanding the applied pressure without failure. Because of its simplicity in concept and execution, reverse osmosis appears to have considerable potential for wide application in water and waste water treatment.

(b) Electrodialysis (ED)

Unaided osmosis is a relatively slow process and hence attempts have been made to combine this with electrolysis. Application of an external electromotive force can draw the ions away from the salt solution towards the electrodes so that the solution is impoverished of its salt content. The reunion of the ions by diffusion can be prevented by using suitable membranes to separate the cathode and anode chambers and also by continuously removing the relatively concentrated solution of the electrolytes from the electrode chambers. To obtain purification of sufficient magnitude a number of electrolytic cells have to be used in series. In essence the apparatus would consist of a number of electrolytic cells each of which is composed of 3 compartments separated from each other by suitable membranes. The saline water circulates in series through the middle compartments of the cells and undergoes progressive purification. The number of cells and the rate of flow may be adjusted to give the degree of purification required. A direct current of 110 to 220 volts is employed. The electrodes are continuously washed with the treated water. One of the main disadvantages of the electrodialysis process is that the membranes get badly damaged as a result of corrosion and scale formation. Another disadvantage is that the cost goes up steeply as total solids content of the finished water decreases. Power loss is minimized if the water is demineralised only partially to final concentrations of less than 500 mg/l in a multi-compartment cell. Average power requirements are 1 kWh/ m^3 of water/1000 mg/ 1 of TDS removed for waters with initial TDS values of 10,000 and less. Since power requirements rise sharply with higher initial values in this method compared to distillation and freezing, this process is adopted only for waters containing less than 10,000 mg/l of dissolved solids.

(c) Osmionic Process

This process is based on the principle of osmosis through ion-selective membranes which pass only anions or cations preventing the passage of the other ions. The concentration gradient between the solutions supplies the potential required to drive the ions through the ion-selective membranes unlike in the case of reverse osmosis where pressure is applied to force the water but not the salts through the membranes.

9.8.5 Ion-Exchange Process

When a salt solution is percolated through a cation exchange resin treated with acids the effluent contains equivalent amounts of the corresponding acids as shown below:

$$H_2Zc + M^{++} + 2A^- \rightleftharpoons MZc + 2H^+ + 2A^-$$

Where M^{++} is Ca^{++} or Mg^{++} . The same equation can also be written for monovalent ions like Na^+ or K^+.

When this acidic effluent is passed through an anion exchange resin which has been treated with alkali so that it contains replaceable hydroxyl ions, the anions are exchanged for the hydroxyl ion with the result that the effluent is rendered free from salts as illustrated as follows:

$$Z_AOH + H^+ + A^- \longrightarrow Z_A A + H_2O$$

Thus it is possible to remove salts from brackish water by a process requiring no more technical skill than that involved in the use of percolation columns. The beds could be regenerated and used repeatedly without appreciable loss in capacity.

High capacity cation exchange materials have been discussed in 9.5.2.2. (b). The anion exchange materials have been prepared by condensing substituted aromatic amines with formaldehyde. These ion exchange resins have come to stay in the field of treatment of water for industries and especially in the production of make up water for high pressure boilers. They have also a place in the treatment of brackish water for the production of potable water.

9.8.6 PERFORMANCE OF RO AND ED PLANTS

Based on evaluation studies conducted by NEERI on the working of desalination plants employing Reverse Osmosis and Electrodialysis principles, following information emerges:

(a) Recurring cost of desalination by Reverse Osmosis (RO) and Electrodialysis (ED) ranges from Rs. 9 to Rs. 31 and Rs. 8 to Rs. 24 respectively per m^3 (1987). Including depreciation and interest on capital, the cost works out as Rs. 40 to Rs. 131 for RO and Rs. 28 to Rs. 85 in case of ED (1987).

(b) Quality of product water in RO is consistent while it is generally not so in ED.

(c) In spite of elaborate pre-treatment, operation and maintenance, the plants could not yield consistent quality of product water within permissible limits. Whenever such consistency in quality was attempted, the product water recovery decreased considerably, thereby raising the cost of treatment of desalinated water. The reject water quantity correspondingly increased.

(d) In the RO plants evaluated, rated capacity of product water was rarely achieved. In the plants Studied by NEERI, only one produced at 100% capacity, while others functioned at 30,50 and 72% of the rated capacity, associated with problems during operation.

(e) Membrane life indicated by various firms for RO plants varied from 1 to 3 years. A membrane life of upto 5 years is claimed for ED. These claims, however, need validation as all plants evaluated operated on an average for 5-8 hours/day only and the frequency of membrane changes was higher.

(f) Pressure pumps maintenance pose several problems during operation; non-availability of spare parts at site can seriously affect their maintenance.

(g) Due to frequent deposition of salts on membrane that needed acid-wash more frequently, the maintenance of ED plants became more difficult.

(h) Scaling is a potential problem and large quantities of acid are used to prevent its formation. General practice has been to use the Langelier saturation Index of the concentrate to calculate acid requirements. Stiff and Davis Stability Index is recommended which results in a significant reduction in acid use.

(i) Energy costs are typically 40-60% of the total operating costs of Reverse Osmosis. The production of 1 m^3 of water requires 4-6 kWh of energy, compared with 12-18 Kwh for distillation process. However, the requirement can be reduced if energy recovery turbines are used, wherever feasible.

(j) Membrane replacements, during the life of an RO plant, are typically estimated to account for 25-35% of the operating costs. There is plenty of scope for reducing the frequency of membrane replacements.

There is no one 'best' method of desalination. Generally, Distillation and Reverse Osmosis are recommended for seawater desalination, while Reverse Osmosis and Electrodialysis are used for brackish water desalination. However, the selection and use of these processes should be very site specific, they must be selected very carefully, especially in rural areas.

One of the major considerations in the selection of a desalination process should be its cost and maintenance. However, despite the substantial costs involved, the availability of desalinated water in arid-zones can be a boon to that area. Where the water is salty, alternative water for consumption is often transported over long distances by truck or animal. When the water is sold, its unit price often exceeds that of desalinated water. Therefore, the economic conditions to support desalination already exit in many water-short areas.

9.9 CORROSION

Corrosion is the phenomenon of the interaction of a material with the environment (water, soil or air) resulting in its deterioration. In water supply corrosion causes significant loss in the hydraulic carrying capacity of pipes and fittings, poor quality of water transported and possible structural failures. Corrosion of metal due to soil electrolyte and stray currents are termed as 'underground corrosion' while that due to water flowing or contained in the pipes or containers is denoted as 'internal corrosion' or underwater corrosion .

9.9.1 MECHANISM OF CORROSION

When a metal is in contact with an electrolyte it has a tendency to ionize and go into solution. The driving force for this process is called the solution potential.

$$M + H^{+} \rightleftharpoons M^{+} + H \qquad (1)$$

The hydrogen ion required for this reaction comes from the ionization of water.

$$H_20 \rightleftharpoons H^{+} + OH^{-} \qquad (2)$$

The hydrogen ion liberated on the metal surface has to be taken away for the ionization to continue according to equation (1). Otherwise, it will cover the metal surface preventing further reaction. The hydrogen atoms can be removed according to the following reactions.

$$2H + O \longrightarrow H_2O \qquad (3)$$

$$2H \longrightarrow H_2 \qquad (4)$$

Reaction (3) is quite significant in water supplies since dissolved oxygen is always present. Reaction (4) requires low pH or a second metal which can serve as an outlet for the hydrogen (depolarizes). In water supplies such low pH conditions are not possible. Where contact with another metal is available galvanic corrosion occurs.

9.9.2 TYPES OF CORROSION

The major types of corrosion are galvanic, concentration cell, stress, stray current electrolysis and bacteria (biochemical).

9.9.2.1 Galvanic Corrosion

When a metal is kept in an electrolyte, it forms a half cell or electrode and the potential associated with it is called half cell potential or electrode potential. In a galvanic cell anodic metal goes into solution while metal is deposited on the cathode. The metal that is placed higher in the galvanic series (electrode potential) will form anode and will be corroded. The Galvanic Series of metals and alloys given as under;

GALVANIC SERIES

Least noble Most Corroded ↑ Decrease in nobility ↑	*Corroded End* Magnesium Magnesium alloys Zinc Aluminium 2S Cadmium Aluminium 17ST Steel or iron Cast iron Chromium iron (active Stainless type 410) Nickel-Resist cast iron 18-8 Chromium-nickel iron (active) Stainless type 304 18-8-3 chromium-nickel molybdenum-iron (active) Stainless type 316 Lead-tin solders Lead Tin Nickel (active) Inconel nickel-chromium (active)

	Hastelloy alloy C (active)
	Brass
Zero	Hydrogen
	Copper
	Bronze
	Copper-nickel alloys
	Monel nickel-copper alloy
	Silver solder
	Nickel (passive)
	Inconel nickel-chromium alloy (passive)
	Chromium-iron (passive)
Increase in nobility	Stainless type 410
	Titanium
	18-8 Chromium-nickel-molybdenum iron
	Hastelloy alloy C (passive)
	Silver
	Graphite
	Gold
	Platinum
Most noble, Least Corroded	*Protected End*

Galvanized iron (zinc-coated) is more serviceable than steel alone, because the iron exposed at joints is protected at the expense of the zinc.

9.9.2.2 Concentration Cell Corrosion

This type of corrosion is most prevalent and occurs when there are differences in the metal ion concentration, anion concentration, hydrogen ion concentration, temperature, or dissolved oxygen level which cause a difference in the solution potential of the same metal thereby promoting corrosion.

In water containing dissolved oxygen, the oxidation of iron from ferrous to ferric state with subsequent hydrolysis results in the increase of hydrogen ion concentration. The increase in the hydrogen ion concentration in contact with hydrogen results in a hydrogen ion concentration cell at this point thus accelerating the rate of corrosion. Similarly an oxygen concentration cell is established due to the difference in the dissolved oxygen content near the anode and cathode areas. This also increases the rate of corrosion at the anode where there is little or no oxygen. In the case of buried pipes, the nature of the soil plays an important role in the availability of oxygen. For example, lime and sandy soils have different permeability for air penetration to the surface of the buried pipelines and local cells form between various parts of the pipeline.

The porous ferric hydroxide deposit acts as a protective coating and retards the corrosion. The accumulation of hydroxide ions near the cathode which reduces the free movement of electrons also retards the corrosion reaction.

9.9.2.3 Stray Current Corrosion

Stray current corrosion is a complex process of metal disintegration under the combined action of soil and stray currents whose usual source is electrified railway track and earthing of electrical fittings. The flow of stray current depends on the distribution of potentials in the track circuit. All metals have greater conductivity than the surrounding environment and hence the current will stay with the metal until there is discontinuity of the metal conductor. Excess of electrons will leave the metal at the points where the environment is highly conductive receptor for the current. Corrosion takes place at the anode, the points where the current leaves the metal and returns to the power source.

Of paramount importance is the simple, reliable and efficient method of measuring the densities of leakage current flowing off the metal in underground pipelines which lie in the field of action of stray currents. This stray current corrosion can be alleviated by making the interfacial resistance of the pipe significantly higher than the surrounding soil, e.g. coating of the pipe. In addition, cathodic protection can be given.

9.9.2.4 Stress Corrosion

Potential difference between different parts of the same metal is due to various factors such as non-homogeneity of surface and non-uniformity of pressure. A smooth surface is less susceptible to corrosion than a rough surface. In fact, the grain size of a metal is important since the solubility of very small grains is greater and hence it is likely to be corroded easily. Metal under stress is easily corroded because the stressed areas become anodic. Therefore, metals exposed to different stresses and strain like points of bolts and nuts in pipe supports are more corroded compared to plain pipes. When a freshly forged metal is used in machinery along with parts made of the same metal but which has been in service for sometime and in which the strain has been relaxed, more rapid corrosion of the new piece of metal is noticed. Residual stress may be relieved by annealing the metal at suitable temperature. Cycles of alternate stresses and strains which induce fatigue also tend to increase the rate of corrosion.

9.9.2.5 Bacterial (Biochemical) Corrosion

Several bacteria like the sulphate reducing bacteria, iron fixing bacteria and other micro-organisms that enter into electrolytic or ionic reactions are responsible for bacterial corrosion. Stagnation of water as in the dead ends gives scope for the development of anaerobic conditions with the production of sulphide from sulphate present in the water. The sulphide thus formed will attack the pipe metal forming black deposits of the metal sulphides which are noticed when the dead ends are flushed. Iron bacteria like Erenothrix and Leptothrix grow utilizing the energy available in the oxidation of metallic iron to the oxide thus corroding the metal. The characteristic stingy masses that come out of handpump tubewells are the result of such growths.

9.9.3 Physical and Chemical Factors of Water Affecting Corrosion

Velocity and temperature of water in pipes affect the rate of corrosion. For aggressive waters, high velocities more than 1 mps are conducive for rapid corrosion. With adequate inhibitor concentration referred to in 9.9.6.3 (b) (1), higher velocities normally prevent metal corrosion. At low velocities the protective properties of water containing inhibitors are not utilized to their best advantage, since the slow movement does not aid the effective diffusion of the protective ingredients to the metal surface. For example at velocities below 0.6 mps corrosion is significant even in the presence of inhibitors.

In general corrosion increases with temperature. This is due to the increase of polarization and diffusion as temperature increases. Even a passive metal may become highly active at elevated temperature. If the heating products have chloride, at high temperature it may form HCl and consequently produce more corrosion. The chloride ion of alkali and alkali earth metals enhance the corrosion of many metals since the chloride ion destroys the passive film on the metals. Some anions like silicate form an insoluble product that gets deposited on the metal as a protective layer, thus acting as inhibitor of corrosion. The nature of cations present will also influence the corrosion rate. Traces of copper and other noble metals will accelerate the rate of corrosion of iron pipes. Iron and several other metals corrode more readily in ammonium salt solutions than in sodium salt solutions of the same concentration. Some inhibitors which protect iron increase the corrosion of zinc, copper and nickel because of the formation of complex cations with the metals.

Concrete constructions will be attacked by salts present in the ground water. Formation of calcium sulphate from sulphate and calcium carbonate from the concrete is responsible for the latter's corrosion. Water with 200 to 600 mg/l of sulphate and 100 to 300 mg/l of magnesium is considered to be slightly aggressive and water with 600 to 2500 mg/ l of sulphate and 300 to 1500 mg/l of magnesium are aggressive to concrete.

9.9.4 Soil Nature and Corrosion

The corrosion current will depend on the conductance of the medium which is an important factor in the corrosion of buried pipelines and structures. Dry sandy soil has low conductance but in moist clay and mineral areas it is too high. This difference in the conductivity of the soil permits its classification into cathodic and anodic sections. Stray currents from power leaks will be more dangerous to metal structures in soil. Sea water has high conductance which is a significant factor in its corrosive nature. Therefore, investigations for conductivity should form an essential part of the soil analysis particularly for large and lengthy buried pipelines apart from the routine tests of pH, redox potential, chemical analysis for calcium carbonate, sulphate, sulphide, pyrites, free carbon, moisture content, organic content and grain size analysis.

9.9.5 Corrosion Testing

Corrosion rates are often expressed as loss in weight from clean metal per unit surface area (g/cm^2) during a specified period of time (hour, day, month or year). If pits are caused by the corrosion, then the intensity of corrosion is expressed as the depth of the pit during a specified period (mm/year).

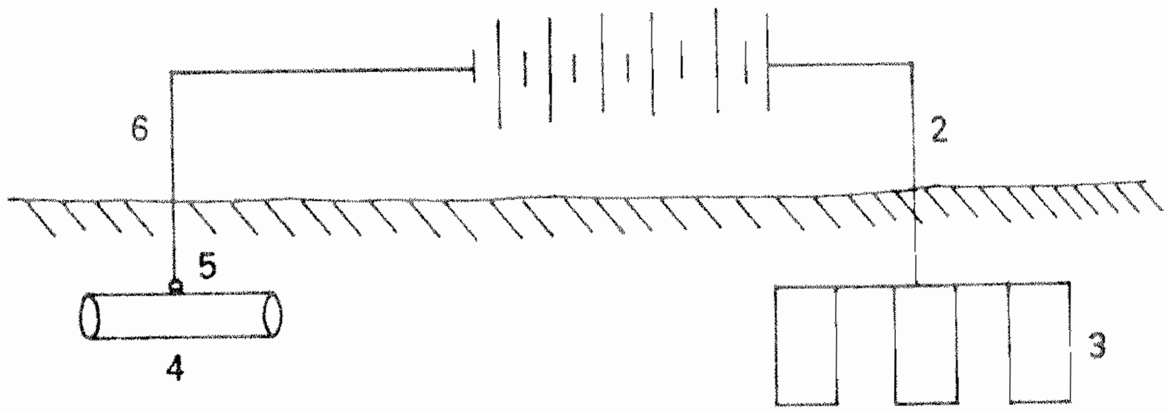

FIGURE 9.7: CATHODIC PROTECTION ASSEMBLY

Corrosion testing can be carried out either in the field or in the laboratory under controlled environment simulating field conditions. Corrosion testing is done using coupons or resistance probes. Coupons are made up of the same material as the structure and normally insulated from the main structure. The coupons are cleaned before and after insertion and the weight loss is expressed as $g/cm^2/year$ which is the measure of corrosion rate. Resistance probes are metallic rods or plates inserted at elbows using a tee in the mainstream of water or in a bypass. They operate on the principle that when a thin wire or foil corrodes its electrical resistance increases due to the decrease of its cross sectional area. The resistance measurements are converted to corrosion rate. Other field tests use thickness detectors for measuring the metal remaining in the corroded pipes or visual examination as a crude method. All these tests are not completely satisfactory by themselves.

Investigation of groundwater level and characteristics of water along with the results of laboratory or field test can be used to predict the possible corrosivity of the soil in which pipes are laid. Correlation between the soil resistivity or conductivity and corrosion is given in Table 9.7.

TABLE 9.7

CORRELATION BETWEEN THE RESISTIVITY AND CORROSION

Resistivity (ohms/cm)	Corrosion
Upto 500	Very strong
500-1500	Strong
1500-2500	Moderate
above 3000	Feeble or none

Mud, muck, clay, tidal marsh and organic soils in high water tables fall under the category of strong to very strongly corrosive. Sands, sandy loam, porous and clay loam in low water tables are moderately corrosive. Even soils with good or feeble corrosion may contain pockets of low resistivity. It is at the junction of such soils that corrosion is seen maximum. A pipeline passing from a high resistance soil to a low resistance soil will corrode in the latter because of difference in pipe to soil potentials of the two area. The current flows from the pipe through the bad soil to the good soil and then back to the pipe.

9.9.6 CORROSION CONTROL

9.9.6.1 Cathodic Protection

Cathodic protection is the application of electricity from an external power supply or the use of galvanic methods for combating electrochemical corrosion. Cathodic protection should be used as a supplement and not as an alternative technique to other methods of protection. It may be a more suitable and expeditious method for control of external corrosion of pipelines.

(a) Basic Principle

The basic principle is to make the entire surface of the equipment cathodic thus affording protection since corrosion takes place only at the anodic surface. This can be achieved by connecting it to a D.C. source. In this case, the anode consists of specially earthed electrodes. The general arrangement in a cathodic protection assembly is shown in the Fig. 9.7.

The current from the positive pole of the D.C. source flows through the conductor 2 into the earthed anode 3 and then into the soil. From the soil the current flows to the surface of the pipe 4 to be protected and flows along the pipe to the drainage junction point 5, the conductor 6 and back to the negative terminal of the current source. Thus the entire surface of the underground pipe or equipment becomes cathodic and is protected from corrosion while the earthed anode gets corroded. The anode is usually a scrap metal e.g. old tubes, rails etc. Other metals which are resistant to attack by surrounding soil like special alloys or graphite are also used. The conductivity of the protective coating has a direct influence on the length of the protected section of the pipe. The required power increased with increasing conductivity of the coating.

(b) Preliminary Investigations

The existing pipeline has to be inspected to ascertain the sections which require protection. Other basic information required are :

(1) Plan and details of the pipelines (showing branch connections, diameter, length and wall thickness) and

(2) Location plan of the section to be protected along with :

   (i) Data on soil resistance along the section to be protected at the intervals of at least 100 m as well as the earthing points.

   (ii) Information on the availability of sources of electricity, amperage, voltage, AC/DC (phase) in the vicinity and spaces for housing current supply and controls.

(iii) Data on the conductivity or resistivity of the existing protective insulation; and

(iv) Condition of the pipeline, if it is already in use.

(c) Power Requirements

With the above data, minimum current density and maximum protection potential can be worked out. The capacity of the current source for a cathodic protection system depends on (1) length of the section to be protected (2) type and state of the coating of the pipeline (3) diameter of the pipe (4) wall thickness of the pipe (5) conductivity of the soil and (6) design of anode earthing. The power requirements vary from 0.4 to 10 kilowatts in most cases. The possible current sources are DC generator, converter-rectifier, storage batteries of dry or acid type. The pipeline should be at least 0.3 V negative to the soil.

(d) Anodes

The main power loss occurs in the anode earthing. The earthing can be carried out by any metal (pure or scrap) of any shape and also carbon forms like coke or graphite. When tubes are used the earthing can be either horizontal or vertical. Near the earthing zone, soil treatment can be done to reduce soil resistance by adding salts like sodium chloride, calcium chloride or moistening the soil, the former being better and long lasting. Carbon or graphite electrodes have longer durability than metal electrodes.

(e) Other Facilities

A cathodic protection station should provide space for housing the equipment, installation of current sources, supply and distribution zones, equipment for check measurements, construction of earthing structures and facilities for carrying out operational tests.

9.6.6.2 Protection by Sacrificial Anode

Sacrificial anodes serve the same purpose as the cathodic protection system but does not require electric power supply. The required current is supplied by an artificial galvanic couple in which the parts to be protected, usually iron or steel, is made as the cathode by choosing the other metal, having the higher galvanic potential, as the anode. Zinc, aluminium and magnesium (with sufficient purity) or their alloys which are higher up in the galvanic series must be used for this purpose. Sheets of zinc suspended in a coagulation basin is an example. A single protector anode will not be sufficient and it will be necessary to install a number of such anodes generally spaced at 4 to 6 m in the pipeline or the structure to be protected.

The performance and service life of anodes depend mostly on the nature of soil or water surrounding them. Use of fill materials in the soil such as clay and gypsum powder results in low resistance of anode earthing and yields a high current. The costs of protection by galvanic anode would be preciously higher in the case of pipeline networks in big towns since it would be necessary to suppress incidental contacts. For the application of galvanic protection the resistance of the soil should be less than 12 ohm-m. A higher resistance of the circuit can neither achieve the required current density nor reduction of the pipe to soil potential. In such cases, cathodic protection by means of external power supply offers better protection.

9.9.6.3 Control Of Internal Corrosion

(a) Associated Factors

Corrosion of the interior surfaces of waterpipes results in reduced carrying capacity, redwater and taste and odour problems. Experience in the country has shown that the 'C' value of cast iron pipes have gone down to as low as 45 in 30 years of service due to corrosion.

The mineral content, dissolved oxygen level and pH of water influence the corrosion rate of mild steel. The effect of dissolved oxygen is decreased by increase in pH. With no minerals the pH of water in mild steel containers adjusts itself to about 8.4 and corrosion becomes negligible. Pitting can occur at joints and welds (stressed areas) if pH is around 8.4 but not above that required for complete protection.

In the absence of carbonate minerals, increasing concentration of other minerals such as chloride and sulphate salts increases the corrosion rate at all pH values below the pitting range of pH. Increasing temperature accelerates both general corrosion and pitting.

Bicarbonates inhibit corrosion. In the absence of calcium, the inhibitory effect of bicarbonate is maximum at pH 6.5 to 7.0 when its concentrations are 5 to 10 times above the chlorides and sulphates. It is minimum at pH 8 to 9.

When dissolved oxygen is absent, the type of minerals present have insignificant effect on mild steel.

The method of controlling corrosion by deposition of calcium carbonate was first suggested by Langelier. According to him the tendency of deposition of calcium carbonate depends upon carbon dioxide and calcium carbonate balance in water.

Langelier index, $I = pH - pHs$, where pHs is associated with calcium carbonate equilibrium (determined by marble test or by calculation from dissolved solids) and pH is the actual pH of the water in the pipeline. When $I = 0$ neither deposition nor dissolution of calcium carbonate takes place. A positive value indicates that the water is oversaturated with calcium carbonate (or lacking in free carbon dioxide) and will tend to deposit calcium carbonate. A negative value indicates that the water is undersaturated (or has an excess of free carbon dioxide) and will tend to dissolve existing deposits of calcium carbonate.

The Langelier's saturation index is not quantitative but shows only the directional tendency. Hence other indices of evaluating the scale-forming or dissolving properties of water have been developed.

An index was proposed by Ryznar, using the empirical expression 2 pHs-pH, which is known as Ryznar Stability Index to differentiate it from the saturation index. Values of the stability index greater than about 7.0 indicate a corrosive water, while, values less than 7.0 indicate a scale forming water. This index is of particular interest in evaluating waters of widely different composition.

A minimum alkalinity of 50 to 100 mg/l as $CaCO_3$ and a minimum of about 50 mg/l calcium (as $CaCO_3$) must be present at normal temperature (0° to 70°C) for giving some degree of protection. The protective action is decreased by increasing proportions of chlorides and sulphates. This limitation is less significant in the absence of dissolved oxygen.

Excessive residual chlorine may increase the corrosiveness of water. High temperatures will tend to increase the rate of corrosion while other factors remaining constant. Soft waters are generally more corrosive than hard waters.

Carbon dioxide acidity or mineral acidity will increase corrosion of iron or destroy the protective coatings. High pH will decrease corrosion, but very high pH levels may be destructive to galvanized or other protective coatings thereby actually increasing the corrosion.

(b) Inhibitors

An inhibitor is a chemical which when added to the corrosive environment will effectively decrease the corrosion rate. It can be considered as the opposite of a catalyst. It retards or stops the corrosion reaction. Inhibitors may be organic or inorganic in nature. Most of the inorganic inhibitors such as silicate, chromate, phosphate, borate etc. control the rate of corrosion by acting on the anode. The use of some of these is not advisable under all conditions. For example the improper use of chromate may accelerate the rate of corrosion rather than inhibiting. If hydrogen polarization is present, the addition of chromate or any other oxidizing agent will cause depolarization and thus increase the flow of corrosion current. Again if insufficient amount of inhibitor is used to provide a complete film over the anode, the small area left exposed will corrode with increased rapidity thereby causing pitting. Sodium silicate is used as a good inhibitor. Alkaline sodium nitrate alone or in conjunction with other inorganic inhibitors such as phosphate is good inhibitor. Sodium benzoate with sodium nitrate is a good inhibitor for iron.

Organic inhibitors may act in a variety of ways. Organic colloids form protective layers by adsorption. Organic bases form positive ions containing hydrophobic groups. These positive cations attach themselves through nitrogen to the cathodic surface. Their effectiveness as inhibitors depend on the size of the hydrocarbon. A few parts per million of tertiary amine gives almost complete protection. High molecular weight amines derived from rosin are good inhibitors. The protective action of the inhibitors increase with temperature.

Vapour phase inhibitors (VPI) vaporize readily and form an inhibiting (or) protective layer. These inhibitors are used to protect steel or iron in presence of moisture and SO_2 Metal parts may be wrapped in papers impregnated with VPI instead of using layers of grease or oil as rust protective substances. Dicyclohexyl ammonium nitrate and chlorohexyl amine carbonate are used as good VPI.

Some of these inhibitors may not be suitable for community water supply unless they are proved to be harmless for consumption. But they are suitable for industrial water systems.

(c) Methods

(i) Deposition of Protective Coatings

A thin film of calcium carbonate can be deposited by the water on the inner surface of pipes by adjusting pH and alkalinity of the water to keep the Langelier Saturation Index 'I' to a slightly positive value. Lime or soda ash or both can be used to raise pH and alkalinity.

Small amount of sodium silicate can deposit dense, adherent but slightly permeable film. A dose of 12 to 16 mg/l is maintained in the beginning and gradually reduced to 3 to 4 mg/l. Organic coatings such as enamels, tar or bituminous coating are effective only to the extent of their coverage and durability. Epoxy coatings hold promise but their toxic effects due to leaching are not fully established. For cast iron and steel pipes, cement lining of the interior surface is satisfactory. Insertion of plastic pipe into an existing partly corroded pipe is also useful. For controlling corrosion of reinforcing steel and preventing disintegration of concrete in RC dome covers of overhead tanks, the concrete cover of such domes may be adequately protected (IS No. 456 : 1978). Protective coating to reinforcement is also suggested.

Some polyphosphates are reported to inhibit corrosion by forming protective films on the cathodic area. They also function as inhibitors for precipitation of calcium, magnesium and iron. Red water problem has been minimized in certain cases because oxidation and precipitation of iron is prevented. Sodium hexametaphosphate (Calgon) is the most widely used polyphosphate. The effectiveness of polyphosphates is progressively greater at increasing turbulent velocities and at increasing concentrations. The initial dose may be as high as to 6 to 12 mg/l and then reduced to 1 to 2 mg/l. This can prevent the formation of rough deposits and remove sharp projections from the existing rough films.

(ii) Treatment of Water

Treatment of water such as adjustment of pH, removal of carbon dioxide, increase in calcium or carbonate ion concentration or addition of inhibitors can overcome to a large extent the corrosive nature of water. Chemical treatment can be effective as only a supplement to other methods like protective coatings and is limited by the cost.

Iron bacteria problems in tubewells can be overcome by treating the well with concentrated bleaching powder solution dose of 50 mg/l (as chlorine) and a contact period of 6 hours. It is necessary to periodically flush out the dead ends so that stagnation for more than a month does not take place. After flushing, these dead ends have to be disinfected by chlorine. De-oxygenation or deactivation of water is the essence of reducing corrosive nature of water and is accomplished by passing over heated scraps of iron or by deoxygenation under vacuum. These methods, however, are not practised in community water supply systems because of cost considerations but are eminently suitable for industrial water systems.

CHAPTER 10
DISTRIBUTION SYSTEM

10.1 GENERAL

The purpose of the distribution system is to convey wholesome water to the consumer at adequate residual pressure in sufficient quantity at convenient points. Water distribution usually accounts for 40 to 70% of the capital cost of the water supply project. As such, proper design and layout of the system is of great importance. Metering is recommended for all cities as indicated in section 17.4.2.

10.2 BASIC REQUIREMENTS

The requirements for the distribution system may be classified as functional and hydraulic. The geometrical configuration of pipes, reservoirs and boosters, selection and proper location of valves, specials, etc., for efficient operation and maintenance and overall economy in cost constitute some of the functional aspects. Adequate residual pressure at the maximum demand depends upon the hydraulic characteristics of the system.

10.2.1 CONTINUOUS VERSUS INTERMITTENT SYSTEM OF SUPPLY

In the continuous system of supply, water is made available to consumer all the twenty-four hours a day, whereas in the intermittent system, the consumer gets supply only for certain fixed hours (a few hours in the morning and a few hours in the evening).

The intermittent system suffers from several disadvantages. The distribution system is usually designed as a continuous system but often operated as an intermittent one. There is always a constant doubt about the supply in the minds of the consumers. This leads to limited use of water supplied, which does not promote personal hygiene. The water is stored during non-supply hours in all sorts of vessels which might contaminate it and once the supply is resumed, this water is wasted and fresh supply stored. During non-supply hours, polluted water might reach the water mains through leaky joints and thus could pollute the protected water. There will be difficulty in finding sufficient water for fire fighting purposes also during these hours. The taps are always kept open in such system leading to wastage when supply is resumed. This system does not promote hygiene and hence, wherever possible, intermittent supply should be discouraged.

10.2.2 SYSTEM PATTERN

For efficient and equitable distribution of water, a grid pattern, where the different mains are interconnected keeping dead ends to a minimum, is recommended. The system facilitates any one point being fed at least from two different directions. For small water supplies, the tree or branch system with smaller mains branching off from a single trunk main may be adequate.

10.2.3 ZONING

Zoning in the distribution system ensures equalization of supply of water throughout the area. The zoning depends upon (a) density of population (b) type of locality (c) topography and (d) facility for isolating for assessment of waste and leak detection. If there is an average elevation difference of 15 to 25m between zones, then each zone should be served by a separate system. The neighboring zones may be interconnected to provide emergency supplies. The valves between the zones, however, should normally be kept closed and not partially opened. The layout should be such that the difference in pressure between different areas of the same zone or same system does not exceed 3 to 5m.

10.2.4 SYSTEM OF SUPPLY

In selecting a source of water supply for a town, the mode of conveyance of water from the source to the town is a factor for consideration. Water could be conveyed by gravity alone, or by pumping, or by gravity-cum-pumping. Any of these three modes could be selected based mainly on the elevation of the source of supply with respect to the town. Efforts should be made to minimize the cost of transmission by considering the various alternatives and their suitability for the given situation.

10.2.5 LOCATION OF SERVICE RESERVOIRS

The location of service reservoirs is of importance for regulation of pressures in the distribution system as well as for coping up with fluctuating demands. In a distribution system fed by a single reservoir, the ideal location is a central place in the distribution system, which effects maximum economy on pipe sizes. Where the system is fed by direct pumping as well as through reservoirs, the location of the reservoirs may be at the tail end of the system. If topography permits, ground level reservoirs may be located taking full advantage of differences in elevation. Even when the system is fed by a central reservoir, it may be desirable to have tail end reservoirs for the more distant districts. These tail end reservoirs may be fed by direct supply during lean hours or booster facilities may be provided.

10.3 GENERAL DESIGN GUIDE LINES

10.3.1 PEAK FACTOR

The per capita rate of water supply indicates only the average consumption of water per day per person over a period of one year. In the design of water supply distribution system, it is to be recognized that consumption varies with the season, month, day and hour. As far as the design of distribution system is concerned, it is the hourly variation in consumption that matters. The fluctuation in consumption is accounted for, by considering the peak rate of consumption (which is equal to average rate multiplied by a peak factor) as rate of flow in the design of distribution system.

The variation in the demand will be more pronounced in the case of smaller population and will gradually even out with the increase in population. This is so because in a large population different habits and customs of several groups tend to minimize the variation in the demand pattern.

The following peak factors are recommended for various population figures:

For population less than 50,000	3.0
For a population range of 50,000 to 2,00,000	2.5
For population above 2,00,000	2.0
For Small Water Supply Schemes (Where supply is effected through standposts for only 6 hours)	3.0

10.3.2 FIRE DEMAND

Fire demand can be assessed as per the norms given in section 2.2.8.3. Reference can also be made to IS 9668-1980

10.3.3 RESIDUAL PRESSURE

Distribution system should be designed for the following minimum residual pressures at ferrule points:

Single storey building ⇒ 7 m

Two storey building ⇒ 12 m

Three storey building ⇒ 17m

Distribution system should not ordinarily be designed for residual pressures exceeding 22 meters. Multistoreyed buildings needing higher pressure should be provided with boosters.

10.3.4 MINIMUM PIPE SIZES

Minimum Pipe sizes of 100 mm for towns having population upto 50,000 and 150 mm for those above 50,000 are recommended. For dead ends, less than 100 mm can be considered. If it is a grid, less than 100 mm can be used in situations where no further expansion is contemplated.

10.3.5 LAYOUT

The distribution layout should be such as to facilitate hydraulic isolation of sections, metering for assessment and control of leakage and wastage.

10.3.6 ELEVATION OF RESERVOIR

The elevation of the service reservoir should be such as to maintain the minimum residual pressure in the distribution system consistent with its cost effectiveness. The hydraulic gradient in the pipe should normally be between 1 and 4 per thousand at peak flow.

A suitable combination of pipe sizes and staging height has to be determined for optimization of the system. The staging height of service reservoirs is normally kept as 15-20m.

10.3.7 Boosting

For distant localities, boosters may be provided instead of increasing the size of mains or height of the reservoir unduly for maintaining the required pressure.

10.3.8 Location of Mains

For roads wider than 25 meters, the distribution pipes should be provided on both sides of the road, by running rider mains suitably linked with trunk mains.

10.3.9 Valves

(a) Sluice Valves

Sluice valves shall be located on at least three sides of every cross-junction and at every kilometre on long mains. The size of the sluice valve shall be the same as the size of the main up to 300 mm diameter and at least two-thirds the size of main for larger diameters.

(b) Air Valves

These have been discussed in 6.16.3.

(c) Scour or Blow Off Valves

The scour or blow off valves have been discussed in 6.16.2.

(d) Flow Dividing Valves

These specially devised and constructed valves are used in distribution and other mains at the branch point to ensure that the assigned flow in a distribution main is always maintained. These are based on the principle that the diaphragm or the other arrangement in valves opens proportionally depending upon the upstream pressure allowing the regulation of flow, irrespective of the pressure conditions obtained in the distribution main.

(e) Maximum Demand Controllers

The maximum demand controller permits all flows upto a preset value and automatically assumes control when the flow just exceeds this predetermined rate, thus preventing excess withdrawals. This form of controller finds considerable use both in municipal and industrial installations, where two or more users taking water from a common source, are to be prevented from consuming more than a set quantity.

10.4 SERVICE RESERVOIRS

10.4.1 Function

The service reservoirs provide a suitable reserve of treated water with minimum interruptions of supply due to failure of mains, pumps etc. They also enable meeting the widely fluctuating demands when the supply is by intermittent pumping. They are also helpful in reducing the size of the mains which would otherwise be necessary to meet the peak rates of demand. They can serve as an alternative to partial duplication of an existing feeder main as the load on the main increases.

10.4.2 Capacity

The capacity of the service reservoir to be provided depends upon the better economic alternatives amongst various options. A system supplied by pumps with 100% standby will require less storage capacity than that with less standby provision. Similarly a system divided into interconnected zones will require less storage capacity for all the zones except for the zones at higher elevations.

However, the minimum service or balancing capacity depends on the hours and rate of pumping in a day, the probable variation of demand or consumption over a day, the hours of supply can be calculated from a mass diagram or by a demand and pumping budget. The variation of demand in a day for a town which depends on the supply hours may have to be assumed or known from similar towns or determined based on household survey.

Typical example on estimation of storage capacity is given in Appendix 10.1.

10.4.3 Structure

The ground level reservoir is generally preferred as storage reservoir which is circular or square or rectangular in shape. If it is circular, it is usually constructed of RCC and in the case of other shapes it is constructed either of RCC or masonry. The elevated reservoirs are used principally as distributing reservoirs and can have shapes like circular, square, rectangular and conical or may be of Intze type. They are generally made of RCC or prestressed concrete. Small capacity tanks can be fabricated with steel or PVC or HDPE. Circular shapes are generally preferable as the length of the wall for a given capacity is a minimum and further the wall itself is self-supporting and does not require counterfort. Reservoirs of one compartment are generally square and those of two or three compartments may be rectangular with length equal to one and half times the breadth. The economical water depth for reservoirs with flat bottom upto 1000m^3 capacity is between 3 and 5.5m. The service reservoirs should be covered to avoid contamination and prevent algal growths. Suitable provision should be made for manholes, mosquito-proof ventilation, access ladders, scour and overflow arrangements, water level indicator, and if found necessary, lightning arresters.

10.4.4 Inlets and Outlets

The draw pipe should be placed 15 centimenters above the floor and is usually provided with a strainer of perforated cast iron. The reservoirs filled by gravity are provided with ball valves of the equilibrium or other type which close when water reaches full tank level. The overflow and scour main should be of sufficient size to take away by gravity the maximum flow that can be delivered through the reservoir. The outlet of the scour and overflow mains should be protected against the entry of vermin and from other sources of contamination. The inlet or outlet of reservoir should be such that no water stagnates. When there are two or more compartments, each compartment should have separate inlet and outlet arrangements, while the scour and overflow from each compartment may be connected to a

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single line. To avoid waste of energy, it is advantageous to form the opening of the outlet with a configuration identical to the surface. This could be achieved by providing a bell mouth at the opening of the outlet pipe. The details of the bell mouth for different sizes of openings are given in Appendix 10.2.

10.5 BALANCING RESERVOIRS

The tank is said to be "floating on the line" when connected by a single pipe to the source and the distribution system. When the rate of supply exceeds the demand, water flows into the tank. When demand exceeds supply, water flows through the same pipe from the tank. The relation between rate of supply, rate of demand and tank capacity is based on a study of the service required as in case of service reservoirs.

When the balancing tank floating on the line is designed for the full service storage based on a study of the hydrograph of demand, its location and altitude is governed by the same conditions as are applicable to the service reservoir. Where the distribution system is designed for direct pumping into the system, it is advantageous to provide a balancing tank at the end of the system with a nominal capacity (1 or 2 hours) to provide pressure relief and improve the tail end distribution. The balancing reservoir has the advantage of minimum of pipe work and operational maintenance.

10.6 HYDRAULIC NETWORK ANALYSIS

10.6.1 PRINCIPLES

Hydraulic analysis of the pipe network is the building block for the design of water distribution system and essentially involves determination of flow conditions associated with specified pipe sizes, the location and size of reservoir and capacity of pumps.

Irrespective of the methods used, the hydraulic analysis of pipe network is based on fundamental laws, viz., $\sum Q = 0$ at a junction, $\sum H = 0$ around a loop or a circuit and $h = kQ^n$, which is the exponential friction flow equation relating the head loss to the flow in pipe.

The problem of hydraulic network balancing is one of finding either the distribution of flows in the pipes given a set of nodal inflows and outflows, or the distribution of pipe head losses given a set of some nodal water elevations, subject to Kirchoff's laws. From the layout, a proper skeletonizing of the network is done and pipe lengths are determined. On the basis of pipe sizes chosen by the engineer from experience the network balancing reduces to a problem of solving a set of non-linear simultaneous equations in the pipe flows and pipe head losses. Either of the two methods, viz.;

(a) Balancing head losses around loops by correcting assumed flows, or

(b) Balancing flows at junctions by correcting assumed head losses in pipes,

is applicable. Notable among the several methods applicable are those developed by (i) Hardy Cross, (ii) the electrical analogy method developed by Mcllroy (iii) the graph theory approach and more recently, (iv) iterative procedures such as Newton-Raphson method using digital computers. The second method belongs to analog category and the rest belong

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to digital category. These methods have virtually replaced other earlier ones largely on account of their accuracy and efficiency. The Hardy Cross method is a relaxation technique, which, through successive iterations, applies a series of linearly approximated correction to either assumed flows or head losses of all the pipes of the network.

10.6.2 METHODS OF BALANCING

(a) Hardy Cross Method

(i) Balancing Heads

In this method, from the knowledge of system inflows and outflows, the flows in all the pipes of the network are distributed so as to meet continuity constraints at all the nodes. When inflows and outflows are explicitly known, this will involve assigning as many flows as there are primary loops in the system. The requirement that the sum of head losses around all primary loops should equal zero gives rise to a system of as many equations. Solution of the exactly determined system of non linear equations is effected by a systematic relaxation in the Hardy-Cross method. In the Hardy Cross method of balancing heads, which is a trial and error process, the correction factor for assumed flows (necessary formulae are made algebraically consistent by arbitrarily assigning positive signs to clockwise flows and associated head losses and negative signs to anti-clockwise flows and associated head losses) ΔQ in a circuit is calculated by the formula:

$$\Delta Q = \frac{\sum H}{n.\sum \frac{H}{Q}}$$

Where Q = Quantity of flow

H = Head loss

n = Constant, 1.85 for Hazen William's formula

The assumed flows are corrected accordingly and the procedure repeated until the required degree of precision is reached. This is essentially a repetitive procedure. The sequential steps are presented below:

(i) Assume suitable values of flow Q in each pipeline such that the flows coming into each junction of the loop are equal to flows leaving the junction,

(ii) Assign positive sign to all clockwise flows and negative sign to all anti-clockwise flow,

(iii) Compute the head loss H in each pipe by use of the friction formula with the help of chart or monogram giving the same sign as for the flows,

(iv) Compute $\sum H$ (i.e. algebraic sum of the head losses) around each loop and if this is nearly equal to zero in all loops (within allowable limits of $\pm$ 0.15 m), the assumed flows are correct,

(v) Otherwise, if $\sum H$ is not equal to 0 for any loop, compute the error in flow

$$\Delta Q = -0.54 \frac{\sum H}{\sum \frac{H}{Q}}$$

and the correction factor is of the opposite sign. Add the correction factor to the assumed flows with due regard to the sign of flows.

(vi) Pipes operating in more than one circuit draw corrections from each circuit. However, the second correction is of the opposite sign as applied to the first circuit,

(vii) Repeat the cycle, till $\sum H$ (around each loop) is nearly equal to zero within the allowable limits. Then the final values of flows are the actual values in the pipelines,

(viii) If during the correction process, the head difference in an element becomes zero, the pipe should be omitted from the particular balancing operation in which this occurs.

A computer program for solution of the head balance problem could be written.

In setting up the program, the following guidelines will be helpful.

(i) Each primary loop is first numbered, (i) serially starting from 1 (i= 1, 2 . . . . N),

(ii) The pipes in each loop are then numbered, (i,j) with the loop number first and pipe number second, serially starting from (i, 1) (j =1, 2, ... N_p),

(iii) Flows in the clockwise direction in the pipe of any loop is considered positive, anti-clockwise negative. This applies to correction ΔQ_i also. The sign of head loss $H_{i,j}$ is the same as that of $Q_{i,j}$. The ratio H/Q or Q/H is thus always positive,

(iv) Successive corrections to flows (ΔQ_i) are calculated from Equation

$$\Delta Q_i = -\frac{\sum_{j=1}^{n} H_{i,j}}{n \sum_{j=1}^{n} \frac{H_{i,j}}{Q_{i,j}}} \tag{10.1}$$

Here n is the exponent of Q in the simplified pipe flow formula $H_{i,j} = K_{i,j}Q^n_{i,j}$. These corrections are applied to $Q_{i,j}$ by the computer and the balancing operation repeated until a desired tolerance for either ΔQ_i, or $|\sum H_i|$ is obtained, at which the program terminates. Specification on this criterion is a nontrivial problem reflecting the desired accuracy.

(v) Pipes common to two loops i and k receive flow corrections from both with due regard to signs. When the pipe is being considered in loop i, corrected $Q_{i,j} = (Q_{i,j} + \Delta Q_i - \Delta Q_k)$ whereas when being considered under loop k as pipe (k,l), corrected $Q_{k,l} = (Q_{k,l} + \Delta Q_k - \Delta Q_i)$.

In case of smaller networks, the calculations could be made manually as well.

A typical problem of balancing head loss by correcting assumed flows by hand computation is presented in Appendix 10.3.

(ii) Balancing Flows

When using the method of balancing flows at junctions or nodes of the system, pressures at nodes are assumed on the basis of given pressure surface elevations at some nodes (e.g. fixed elevation reservoirs) and the flows in the pipes are estimated.

In the 'method of balancing flows' (modification of original Hardy Cross Method), which is applicable to junctions and nodes, the flows at each junction are made to balance for the assumed heads at the junctions and the corresponding head losses in the pipes. The correction factor for assumed head losses in the pipes (H) is calculated using the formula :

$$\Delta H = +1.85 \frac{\sum Q}{\sum\left(\frac{Q}{H}\right)}$$

The steps in the computation are as under :

(i) Assume heads at all the free junctions such that the sum of the head losses in clockwise direction equals the sum of the head losses in the anti-clockwise direction in all the loops,

(ii) Assign positive sign to head losses for flows towards the junction and negative sign to those away from the junction,

(iii) Compute the flows in each pipe by use of the friction formula with the help of chart or monogram giving same signs as for the head losses,

(iv) Compute ΔQ (i.e. algebraic sum of the flows) at each free junction and if this is nearly equal to zero at all junctions (within allowable limits of $\pm$ 2%), the assumed head losses are correct,

(v) Otherwise, if $\sum Q$ is not equal to zero at any junction, compute the error in head loss

$$\Delta H = -1.85 \frac{\sum Q}{\sum \frac{Q}{H}}$$

The correction factor is of the opposite sign. Add the correction factor to the assumed head losses with due regard to the sign of head losses,

(vi) Pipes common to more than one loop receive corrections from each loop. However, corrections to the companion circuit is of the opposite sign to that of the first circuit,

(vii) Repeat the cycle till $\sum Q \approx 0$ at each node or junction when the final corrected values of H are obtained.

Although the Hardy Cross Method is rational and mathematically correct, drastic skeletonising of the network because of the complexity, the time consuming nature and the tedium of calculations, particularly for the large size networks and the uncertainty of convergence of values impose serious limitations on this method.

In setting up a computer program, the following guidelines will be helpful:

(i) Each junction of the system is numbered serially, starting from 1, except those with an unknown inflow or take off, where usually a fixed water elevation is specified ($i = 1, 2 \ldots N$)

(ii) All pipes joining node i are numbered (i,j),j denoting the pipe number at junction i ($j = 1, 2 \ldots N_{pi}$)

(iii) Heads and flows towards the node are considered positive; away from it, negative. The same applies to correction ΔH_i also. H/Q and Q/H are always positive

(iv) At each node, i, a test of $\sum Q_i$, is then made to see whether it is zero. If not, the head correction ΔH_i to be applied to all the head losses $H_{i,j}$ in pipes (i,j) meeting at junction i is calculated from equation

$$\Delta H_{i,j} = \frac{n \sum_{j=1}^{n} Q_{ij}}{\sum_{j=1}^{n} \frac{Q_{ij}}{H_{ij}}} \tag{10.2}$$

and applied. The process is repeated until either

$$\underset{i}{Max}|\Delta H_i| \quad or \quad \underset{i}{Max}\left|\sum_{j} Q i, j\right|$$

is less than the prescribed limit.

(v) Pipes common to more than one junction receive ΔH correction from both with due regard to signs, as stated before.

It is pointed out here that any network balancing problem can be solved by either of the two methods-head or flow balance. Where there are two or more reservoirs with fixed water elevations in the system, synthetic or artificial loops can be introduced between them to introduce exactly as many additional equations as necessary to make the system exactly determined. Although, the Hardy-Cross method can be used to solve network problems to any desired degree of accuracy, it is highly time-consuming for large and complicated networks. More powerful rapidly converging methods are now available.

(b) Newton-Raphson Method : Balancing Heads

Network balancing using Newton-Raphson method is again an iterative process but the method seems to be faster and convergence much more rapid from a reasonably good start. The principle of this method is explained most simply by reference to solution of a single equation f (p) = 0. According to Newton's rule, if p is an approximation to a root of f(p), then ($p + \Delta p$) is a better approximation where;

$$\Delta p = -\frac{f(p)}{f'(p)} \tag{10.3}$$

The nature of this result can be recognized from the Taylor series expansion of f(p+Δp), viz.

$$f(p + \Delta p) = f(p) + (\Delta p. f'(p) + \frac{(\Delta p)^2}{2!}.f''(p) + ... + \tag{10.4}$$

terms involving higher powers of (Δp).

f(p + Δp) is equal to zero if (p + Δp) is in reality a solution to f (p)= 0. If, in the above equation, the terms involving powers of Δp higher than the first are neglected, one obtains Newton's rule. The method can be extended to the solution of n simultaneous equations with n variables.

In setting up a water distribution network for balancing heads by Newton-Raphson method on the computer, it is useful to note the following steps and observations; Flows in the pipes are assumed so as to meet all the continuity constraints. The flows in all pipes of loop i are assumed to be in error by ΔQ_i, correction from both loops, the one coming from the loop under consideration being algebraically added, the other being algebraically deducted.

Equations to balance head losses around loops are then framed in terms of corrected flows.

In general, the arranged loop head loss equations take the following form:

$$\left(\sum_j \frac{H}{Q}\right)_i \Delta Q_i + \sum\left[-\left(\frac{H}{Q}\right)_{i,j} Q_k\right] = -\frac{\left(\sum H\right)_i}{n} \tag{10.5}$$

Where the second summation on the L.H.S extends only for the common pipes of loop i. The number of equations in the system is the same as the number of primary loops in the system. For the i^{th} loop on the L.H.S $\left(\sum_j \frac{H}{Q}\right)$ for all pipes of the loop forms the coefficient of ΔQ_i, the correction for all pipes of the loop. The other non-zero terms are of the form $\left(-\frac{H}{Q}\right)_{i,j}$ ΔQ_k, where ΔQ_k is the correction for loop k which has a pipe in common with loop i. The common pipe is called (i,j) in loop i, and by some other name like (k,l) in loop k. If loop t has no pipe in common with loop i, the coefficient of ΔQt in the equation for loop i, will be zero. On the R.H.S of the equation, we have the unbalanced head in loop i with a negative sign, multiplied by the inverse of exponent n in the pipe flow formula chosen.

A general Fortran Program for network head balance according to Newton-Raphson Method could be written to compute $H_{i,j}$ from input $Q_{i,j}$ values and set up the coefficient matrix A for solution for 'ΔQ_i's. The set of linear simultaneous equations could be solved by calling appropriate library subroutines. The computed ΔQ_i are applied to all pipes of the network as explained under Hardy Cross method giving due consideration to common pipes between loops and the iteration proceeds. The program terminates at the allowable head tolerance or when iterations exceed a certain prescribed limit.

The success of the Newton-Raphson technique lies in the selection of a good starting approximation. If the approximation is poor, it can result in the divergence of the solution. Computer programmes are readily available for the Newton-Raphson technique.

(c) Linear Graph Theory

The analysis of water distribution network requires that the node and loop continuity equations be satisfied. Linear graph theoretic approach differs from other methods in a fundamental way. While in other methods, it is customary to change the value of either the assumed flow or head loss using one set of continuity equations and satisfying the other set as constraints, this method depends on the simultaneous utilisation of both sets of equations (node equations and loop equations).

In the graph theory approach, the water supply distribution pipe network is treated as a linear graph (consisting of points or vertices and lines or edges). By the properties of graph theory and matrices, the system equations involving the three physical laws of fluid flow, i.e., Kirchoff node law, Kirchoff loop law and pipe flow formula are combined to form a single set of non-linear equations involving one set of variables i.e., either head loss variables or flow variables. These non-linear equations are then solved by iterative methods. After one set of variables are obtained, the other set of variables are calculated from the pipe flow formula.

In this approach, by dividing the variables as primary and secondary variables according to 'tree' and 'co-tree' pipes, the decision variables are confined to only the primary variables. The application of the Graph theory helps considerably in formulating the hydraulic equation and also in deriving a good starting approximation to ensure fast convergence.

(d) Linear Theory Method

This method, proposed by Wood and Charles is useful for network balancing through "balancing heads by correcting assumed flows". This is also an iterative method, said to converge faster than the Hardy Cross method.

In the methods of balancing described earlier, it is necessary to assume certain values for the variables to start the iterative procedure. Naturally, therefore, the number of iterations depend upon the initial guess. No such initialization is needed in the linear theory method.

The linear theory transforms the loop head loss non-linear relationships into linear relationships by approximating the head loss in each pipe by

$$h_p = (rQ^n)_p = (rQ^{n-1})_p Q_p = (r'Q)_p \qquad (10.6)$$

in which Q_p is the assumed flow in pipe p. Thus the pipe resistance constant r_p is replaced by $(r')_p$ so that, $(r')_p = (rQ^{n-1})_p$

All the nonlinear loop head loss relationships become linear. These linear equations and the node flow continuity linear equations are solved simultaneously to obtain all Q_P values. The solution, however, will not be correct as the obtained Q_P values will not be the same as assumed Q_P values. However, it is claimed that by repeating the process several times, the obtained and the assumed values will be found to be identical, thus giving the correct solution.

In the linear theory, for the first iteration, all the Q_p values are taken as i giving (r')p = rp. (This amounts to assuming the flow to be laminar for the first iteration). It will be observed that this method, if used just as suggested earlier, yields pipe flows which tend to oscillate about the final solution. To obviate this, Wood and Charles have suggested that after two iterative solutions, for all the iterations thereafter, the initial flow rates to be used in the computations should be the average of the flow rates obtained from the past two iterations. Thus, for the i^{th} iteration,

$$Q_P = \frac{(i-1)\,Q_p + (i-2)\,Q_P}{2} \tag{10.7}$$

in which the subscript i, i-1 and i-2 denote the i^{th}, $(i-1)^{th}$, and $(i-2)^{th}$ iterations respectively.

(e) Use Of Models For Analysis

A model must truly represent the system under consideration so that the pressure drops and discharges can be measured directly without trial and error procedures. The variables like head loss, flow and head loss coefficients in a pipe, as also circuits, junctions, and friction laws that govern the system should be properly represented in the analogous model devices. Two kinds of models, namely hydraulic models and electric analogue models have been used. The hydraulic models however have not proved very popular.

(f) Electric Analogue Model

In the direct electrical analogue mode which is used for pipe network analysis, the analogies existing between hydraulic and electric systems are considered. The use of non-linear resistors in electrical systems has made possible the representative simulation of the hydraulic system.

The source of supply in the hydraulic system is represented in the electrical analogue by a constant voltage generator or battery, take-offs by load resistors or electronically controlled devices and pipes by non-linear resistors. Camp and Hazen built the first electric analyzer designed specifically for the hydraulic analysis of water distribution systems. McIlroy continued this approach to network analysis and developed an analyzer that is manufactured commercially. For each branch of the system, the pipe equation, $H = KQ^n$ is thus replaced by an electrical equation, $V = K_e.I^n$, where V is the voltage drop in the branch, I is the current and K_e is the non-linear resistor coefficient whose value is suited to the pipe coefficient 'K' for the selected voltage-head loss and the amperage-water flow scale ratios. If the current inputs and take-offs are made proportional to the water flowing into and out of the system, the head loss will be proportional to the measured voltage drops.

The most important advantage of the direct electric pipeline network analyser is the physical feel of the network system experienced by the designer or operator. Once the pipe network is simulated in the electric network analyser, results can be obtained in a few minutes for alternative sizes of pipes or alternative flow conditions.

The analyzers give the pressure losses and flows in pipelines at an instant in time and the accuracy of the results depends only on the precision of physical elements and measuring instruments involved and the accuracy of the data introduced.

10.7 DESIGN OF PIPE NETWORKS

The problem of design of pipe networks essentially involves determination of pipe sizes which will meet the physical and operational requirements imposed on the network at minimum cost.

The constraints include the hydraulic laws and operational ones such as the minimum permissible sizes, restriction to commercially available sizes, and mainly, minimum residual pressure requirements at critical nodes. The total cost of the network is generally assumed to include the cost of the pipes, pumps and other components and the present value of the maintenance and operating costs. Several approaches have been suggested for handling this economic design problem over the years. Some significant attempts are summarized in the following sections.

10.7.1 APPROXIMATE METHODS

These methods are simple, approximate and are used as a quick check for an existing system or for obtaining preliminary pipe sizes for a new network before subjecting it to detailed analysis. Such methods include method of sections in which the network is cut by imaginary section lines (chosen with regard to the critical points in the distribution system), for an assumed hydraulic gradient (usually 1 to 3 per 1000) and velocity (0.6 to 1.2 mps). The capacity of the pipeline cut by the lines are matched with the actual demand in the areas to be supplied. Any deficiency in the pipe sizes is rectified by the addition of an extra pipe or replacing by a larger size pipe and rechecking in a similar way.

10.7.2 EQUIVALENT PIPE METHOD

A network can be simplified considerably to obtain useful preliminary information on the flows and head losses at important junctions by this method, where a complex system of pipes is replaced by a single line of hydraulically equivalent capacity. The pipes of the sizes smaller than 150mm in the more elaborate systems as well as the connecting pipes with no appreciable pressure differential may be omitted to skeletonise the system to a workable one. The various combinations of pipes between selected junctions could be replaced by hydraulically equivalent pipes reducing the number of units to be analyzed.

In 1969, Teng, O'Connor, Steams and Lynch published an 'equivalent length method' of balancing hydraulic networks and indicated that an approximate solution to the problem of economic pipe sizing can be simultaneously obtained therefrom. Using Hazen-William's formula for pipe flow, a new term L_e was introduced which was

$$L_e = l(100/c)^{1.85} (0.667/D)^{4.86} \qquad (10.8)$$

Where L_e is the length of a pipe of standard diameter (8-in) and standard Hazen Williams C-Value of 100. This pipe is hydraulically equivalent to a pipe whose actual length 1, diameter D and Hazen Williams coefficient is C. Instead of applying the Kirchoff's loop law to the sum of the head-losses $\sum H$ in the loops, the equivalent length method distributes the available head loss to the several pipes directly meeting the requirements $\Delta H = 0$, and attempts to balance the relative pipe resistances in the form of equivalent lengths, L_e in all the loops of the network i.e.

$$\sum L_e = O \text{ for all loops} \qquad (10.9)$$

An iterative procedure similar to the Hardy Cross method has been used for balancing L_e in this study. Assumed flows in all the pipes of the netwofk are successively adjusted to balance the relative pipe resistances. It is claimed that such a balance leads to a minimum possible total of all the equivalent lengths and thus to least amount of pipe in a network of equal-sized pipes. Also, the imposition of the above condition $\sum L_e = O$, is reported to establish a general 'evenness' of flow throughout the system, and 'optimum' design for any set of fixed conditions of topography, pressure requirements, source of supply, draft and geometric pattern of distribution network. The elimination of the trial and error feature of the Hardy-Cross method was cited as an advantage of this algorithm.

In the search for better methods of water distribution system design, the balancing of equivalent lengths' technique would appear to have merit particularly in initial studies preliminary to a comprehensive systems analysis. However, in networks with multiple sources and pump-type boundary conditions, the flow pattern may not be so obvious and problems of convergence could arise.

10.7.3 Pipe Network Cost Minimization Problems

It can be shown that the problem of minimum-cost design of a distribution pipe network subject to

(i) the provision of required domestic and fire flows at specified draw off junctions, and

(ii) the maintenance of minimum residual pressure at critical junctions

can be cast as one of non-linear, integer programming. Such a model and an engineering approach to its solution are briefly discussed. More detailed exposition and reference to earlier works in the topic can also be found in literature.

10.7.3.1 Formulation Of The Objective Function

The principal part of the total cost function of a distribution pipe network is the cost of pipes. The installed first costs of pipes can be related to their diameter by an empirical exponential function of the form:

$$C' = \alpha \, l \, D^{\eta} \tag{10.10}$$

Where C' is the cost, l is the length of pipe and D is the diameter, α and η are parameters to be determined locally. Then the total installed cost of all the pipes in the networks is

$$C = \sum_{all \ i,j} \alpha \ l_{i,j} \ D^{\eta}{}_{i,j} \tag{10.11}$$

Where the paired subscript (i,j) denotes the j[th] pipes in loop i.

In addition to pipe cost, the cost of friction losses in the pipe network constitutes another important component of the total cost. In pumped systems, it represents the cost of energy required to overcome pipe friction. In gravity systems, the same is an indirect 'cost' on the system if we consider that higher pressures are desirable at the draw-off points. As such, the energy cost of pipe friction losses can be incorporated in the objective function for all supplies. Relating this cost to motive power prices (here assumed as electricity), the present value of costs associated with pipe friction losses in the system can be computed and incorporated in the objective function to be minimized. Such a total cost function is

$$C_T = \alpha . \sum_{i,j} l_{i,j} \, D^{\eta}_{i,j} + P' \frac{(P.wbE)}{\theta} \times \sum_{(i,j)} Q_{i,j} H_{i,j} \tag{10.12}$$

Where $Q_{i,j}$ and $H_{i,j}$ stand for the flow and head loss in pipe (i,j) P_v is the present value of an annuity of 1 Rupee discounted at rate r over the economic time horizon T; w is the unit weight of water; b is a load building factor; E is the unit cost of electricity and θ is the wire-to-water efficiency of pumping.

10.7.3.2 Formulation Of The Constraints

The diameters, flows and head losses in the pipe network must meet certain constraints in the form of hydraulic flow formulae, Kirchoff laws for nodes and loops and certain operational constraints regarding minimum pipe sizes, commercially available pipe sizes and minimum permissible residual pressures. Such constraints can be represented by the following set;

(a) $H_{i,j} - \left[84.1 \frac{1}{1.85} l_{i,j} \, D_{ki,j}{}^{-1.37} \, |Q_{i,j}|^{0.36} \right] x Q_{i,j} = 0$ for all pipes (10.13)

(b) $\sum_{i,j} Q_{i,j} + q_m = 0$ for all nodes (10.14)

(c) $\left[\sum_{j} H_{i,j}\right]_{i} + S_{l} = 0$ for all loops (10.15)

(d) $D_{i,j} > D_{min}$ for all pipes

(e) $D_{i,j} \sum (DA) = (D_1 \ldots\ldots D_a)$ for all pipes

(f) $[\sum H_{i,j}]_n < h_k$ over all specified paths (some i,j)

(g) g(relevant q_m S_l)= 0 for all pumps, if any. (10.16)

In the constraints set, (a) is a version of Hazen Williams formula for flow in pipes, (b) and (c) are Kirchoff's node and loop laws respectively, (d) assures that all pipes are not smaller than the prescribed minimum size D_{min},(e) specifies that the sizes shall correspond to commercially available ones (D_1, D_2, D_3, ...D_n), (f) is the equivalent of maintaining minimum permissible residual pressures at draw off nodes, by requiring that along specified pathways in the network the sum of head losses shall not exceed preset magnitudes, and (g) guarantees that the inflow and pressure at pump nodes shall correspond to the specified characteristic curves of pumps. The quantities q_m, S_l, and h_k stand for inflow (or outflow) at node m, unbalanced head at loop i, and maximum pressure difference permissible over path k, respectively.

10.7.3.3 Analysis

This mathematical model for cost minimization of pipe networks assumes that the layout and lengths of pipes are known and, for the moment, that only one demand pattern is considered. The problem can now be recognized as one of non-linear, constrained minimization in numerous variables. The constraint set (e) restricts the domain of feasible diameters to a few specific values, thereby discretizing the objective function and the set of feasible diameters. In this analysis, it is assumed that P_v, b, E, e and C are known, non-negative parameters and $l_{i,j}$, q_m, S_l, D_{mn}, D and h_k are given input vectors. The three sets of variables $D_{i,j}$, $Q_{i,j}$ and $H_{i,j}$, are treated as decision variables, i.e., the solution seeks that set of feasible $D_{i,j}$, $Q_{i,j}$, and $H_{i,j}$ which minimizes the total cost of the pipe network. For this non-linear, integer programming problem, an iterative, sequential search procedure has been developed and the same is briefly outlined in the following subsections.

10.7.3.4 Constructing A Starting Solution

The most direct way of meeting constraint sets (d) and (e) is to choose diameters as the variables to be set for a trial, and derive other decision variables (Q and H) therefrom. Then, while selecting the diameters, only those feasible with respect to (d) and (c) may be chosen. Such diameter selection is a significant initial step which eliminates the round-off procedures that would be otherwise required. The setting of such a diameter vector ($D_{i,j}$ leaves the flows and head losses to be determined. Solving for $Q_{i,j}$, and $H_{i,j}$, with given $D_{i,j}$ from constraint sets (a), (b) and (c) is the familiar problem of hydraulic network balancing.

10.7.3.5 Constructing A Penalty Function

If the constraints system is now examined, the method of starting with a feasible diameter vector and balancing the network to obtain feasible flows and head losses has given rise to a solution feasible with respect to all constraints except set (f). The resulting head losses may either satisfy or fail to satisfy set (f), i.e. head losses summed over all specified paths may or may not be less than the permissible limits set. A rational approach to the treatment of these constraints is to weigh them and blend them into the objective function in such a form that the violation of these constraints will penalize the causative design while ranking alternative designs. Such is the penalty function approach. This penalty function can be related to the extent of violation of the (f) type constraints.

10.7.3.6 Sequential Random Search Procedure

Having established the model and formulated a function to rank alternative designs, a sequential random search should be conducted starting with a trail design (set of diameters) and improving it in successive iterations until a terminal design with very low probability of improvement results. The rationale of this method differs from that of classical optimization in that it does not attempt to identify the global optimum with complete certainty; rather, it provides a statistical estimator of the best design.

The technique can be summarized as follows:

(i) Select a starting design from a specified population of starting designs

(ii) Proceed sequentially from the starting design to an improved terminal design (T.D) according to a set of rules involving random sampling (sweetening)

(iii) Repeat the above steps until several T.D's are obtained. This provides a sample of, say, n Terminal Designs

(iv) Identify the least costly of the n Terminal Designs as the current estimator of the global optimum (we will call it the 'Optimal Design' hereafter). Such steps (i) to (iv) constitute a search.

The algorithm described is a practical, heuristic tool for a mathematically complex and computationally laborious problem.

A schematic flow chart for the sequential random search procedure is presented in Fig. 10. 1.

10.8 RURAL WATER SUPPLY DISTRIBUTION SYSTEM

The water supply in rural areas is effected by one of the following two methods.

(i) Shallow well or deep bore well fitted with hand pump

(ii) Piped water supply with or without house connection through overhead tank and standpipes located at strategic points within the community.

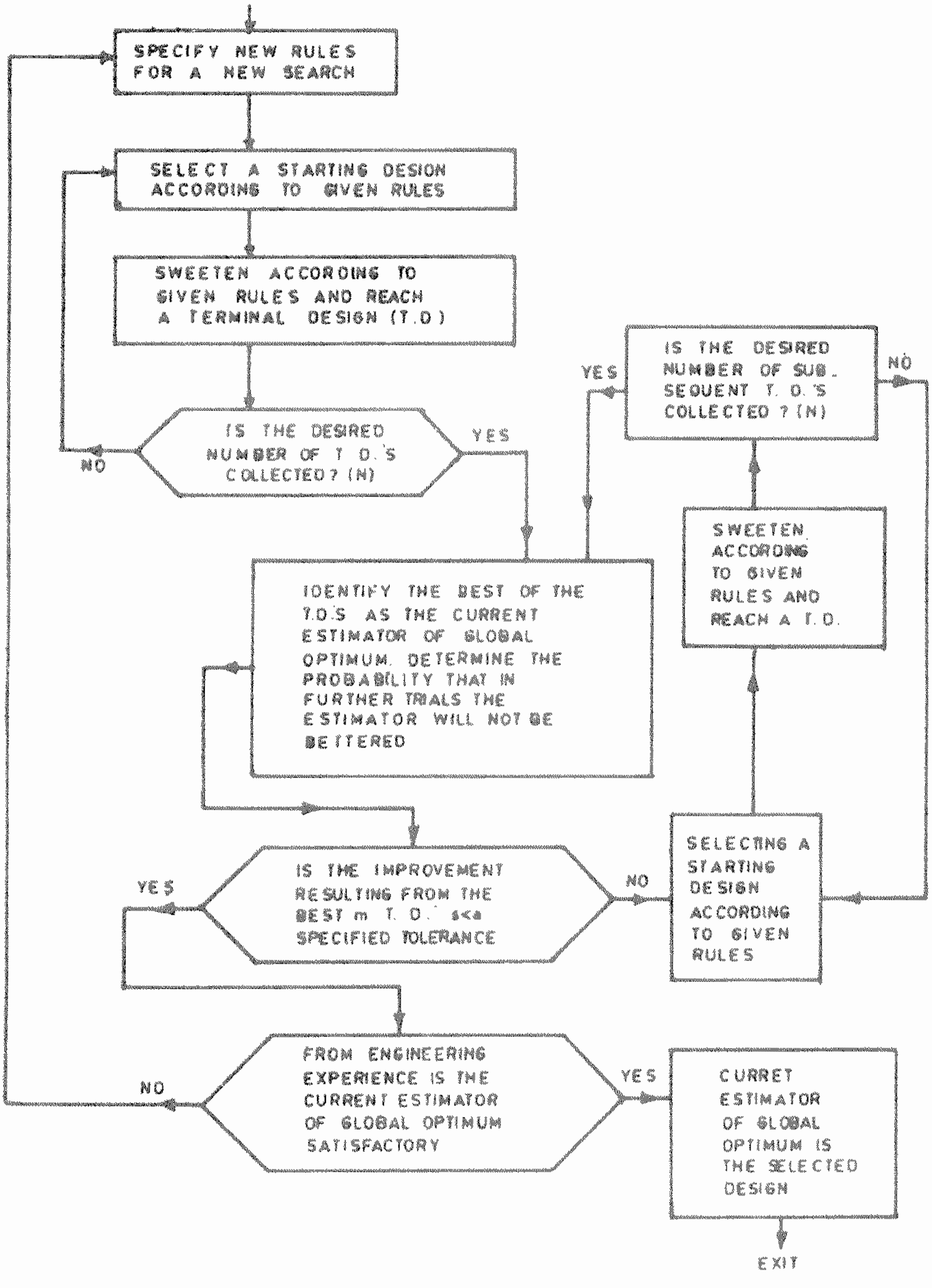

FIGURE 10.1 : SCHEMATIC CHART FOR STOCHASTIC SEARCH TECHNIQUE

The distribution system suitable for the situation is the dead end system of branched system. The system is economic, easy to design and operate. The elevation of the overhead tank is fixed by taking into consideration the residual pressure to be maintained at the farthest end of the distribution system and the length of the connecting pipe. When water is supplied only through stand posts, the tank is generally constructed with a staging height of 6 m for communities with population upto 1500 and with a staging height of 7.5 m for communities with population greater than 1500.

When house connections are also provided, the height of staging may be suitably increased to ensure minimum prescribed terminal pressure.

The distribution system for rural water supply scheme is designed for the peak demand, which is assumed to be four times the average demand (duration of supply is 6 hrs). Techniques are available for the optimization of rural water supply distribution system.

An optimization method is available for single branch dead end system using Lagrangian multiplier technique with an equality constraint on the pressure head in the system. The solution is obtained in the closed form.

A compound water main system consisting of pipes connected in series and with intermediate draw off at the end of each pipe has been subject to cost minimization, using the Lagrangian technique. The input data include water pressure at the inlet, the desired residual head at the extreme end of the pipe, the length of pipes, draw-offs at the end of each pipe and cost function parameters. The closed form analytical solution has been derived for the size of pipe in each leg of compound pipe.

10.9 HOUSE SERVICE CONNECTIONS

10.9.1 GENERAL

The supply from the street main to the individual buildings is made through a house service connection. This consists of two parts viz., the communication pipe which runs from the street main to the boundary of the premises and the service pipe which runs inside the premises. The communication pipe is usually laid and maintained by the local authority at the cost of the owner of the premises while the service pipe is usually laid by the consumer at his cost.

The service connection including the details of the internal plumbing system should conform generally to the National Building Code and particularly to the bye-laws of the concerned local authority. Extreme care should be bestowed for the design and construction of plumbing system. The rational design criteria evolved by CBRI for plumbing should be followed.

10.9.2 SYSTEM OF SUPPLY

The water supply in a building may be through one of the following or combinations of both depending upon the intensity of pressure obtained in the street main and the hours of supply.

(a) Direct supply system, and

(b) Down-take supply system with or without sump and pump.

If the pressures near the premises are adequate to supply water for sufficient number of hours to the water fittings at the highest part of the building, then suitable connections may be allowed to deliver water directly. In cases, where the pressures in the street mains are not sufficient to deliver water supply directly, the down-take supply system with ground level storage and boosting is adopted. Direct supply system is recommended under one condition only when the number of floors in a building is not more than two.

In any case, only one connection is to be granted for the whole building to deliver the total domestic requirement of the day. If there is, however, a non-domestic requirement in the building, then a separate connection shall be given.

The supply in any case is controlled usually by a ferrule on the main, which is throttled sufficiently to deliver the required supply at the pressure contemplated. The supply is also controlled by a stop cock at the beginning of the service pipe. A meter is to be installed beyond the stop cock for measuring the flow. Any temporary disconnection of the supply is made by the stop cock and any permanent disconnection is made at the ferrule. The size of the ferrule should not exceed a quarter of the nominal diametre of the main and also be less than the size of the communication pipe. If a larger size of connection is required, branch with the required number of common service pipe can be used. Where the pipe has to cross a drain, a suitable sleeve pipe may be provided for prevention of cross connection.

10.9.3 Downtake Supply System

(a) General Criteria

In this system, the supply may be delivered directly to the overhead storage tank or to the ground level storage tank. Separate overhead tanks should be provided for flushing and other domestic purposes. The capacity of the overhead and ground level storage tanks are decided by the local bye-laws. Generally a capacity of 50% of the daily requirement is provided in the ground level storage tank. For overhead tanks directly receiving water from public mains, the capacity should take care of the total daily requirement, which could be reduced to 75% if the supply is pumped from the ground level tank.

The pumps shall be designed for peak rate at 3 times the average over 24 hours; or average rate of the 50% of the daily requirement over the actual hours of supply, whichever is greater. A standby pump set of equal capacity shall be provided..

(b) High Rise Buildings

(1) ***Systems:*** The down-take system of water supply in high rise buildings may be one or a combination of the following systems viz., overhead storage system, break pressure tank system and hydro-pneumatic system.

(i) Overhead Storage System

In this system, the tanks are provided on the terrace. A manifold down-take may be taken out from the storage tank which should be laid out horizontally in a loop on the terrace to carry a designed peak load demand. The pressure in the loop at peak demand shall not become negative. Vertical down-takes, as many as necessary, may be taken out from the loop and should be linked to one down take for a zone of 4 storeys at a time and designed for the peak demand it has to serve. A pressure reducing valve shall be provided in the down takes to limit the head to a maximum of 25 m in easily accessible places like ducts, cat walks, etc.

(ii) Break Pressure Tank System

In this system, the entire building is to be conveniently divided into suitable zones of 5 to 8 storeys each. For each such zone, there shall be a break pressure tank, the capacity of which should be such that it holds 10 to 15 minutes supply of the floors it feeds below and shall be not less than 2KL each for flushing and other domestic purposes separately. The down take from the master overhead tank feeds into the break pressure tank.

(iii) Hydropneumatic System

In this system, the supply is through a hydropneumatic pressure vessel fitted with accessories like non return valves and pressure relief valves. Each zone of supply should be restricted to about 7 storeys or 20m, whichever is less. The capacity of the pump should be such as to cope up with the peak demand. Normally three pumps called the lead pump, the supplementary pump and the standby pump respectively, are provided. The last pump is preferably diesel driven to serve when there is a power failure. The hydropneumatic pressure vessel should be an air tight vessel, cylindrical in shape and fabricated from mild steel plates according to pressure tank fabrication code. The capacity should be equivalent to three minutes requirements. The air compressor is also necessary to feed air into vessel so as to maintain the required air-water ratio in the vessel. As soon as the demand exceeds the capacity of the lead pump, the supplementary pump must start automatically.

(c) Fire Storage

Multi storeyed buildings above 25m height have to be provided fire storages in addition to domestic needs, adequate to fight a fire at the rate of 2250 lpm, as a normal fire fighting tanker cannot cope up with fires beyond an elevation of 25m. This limit, however, varies from place to place depending upon the normal height of ladder available with the local fire brigade service for extinguishing of fire.

The tank capacity for fire storage may be of 100 KL, where the supply is intermittent so that it is adequate to fight a fire in the premises at the rate of 2250 lpm for about 45 minutes by which time the replenishment from municipal mains would have commenced. The overflow from the fire fighting tank should flow into the suction tank to maintain a continuous circulation in the static fire tank and also maintain a reserve storage for fire fighting purposes.

The fire fighting pumps may be located in the basement to have a positive suction head and designed to deliver 2250 lpm with a terminal pressure of $3Kg/cm^2$ at the topmost floor so as to obtain from the hydrant 900 lpm discharge with a jet of about 6 m.

On the fire fighting rising main, hydrant tees of 60mm may be provided at every landing of each stair case. A small 25 mm tapping may be provided at each landing with a wheel valve and adequate length of hose pipe for fighting small fires due to electrical short circuiting etc.

The pump set is to be provided with a pressure vessel and automatic pressure switches which start operating when there is a pressure fall on the rising main due to the commissioning of any of the hydrant tees for fighting a fire.

To deal with cases when there is a power failure the high rise buildings should be provided with independent electrical circuits, one connected to the normal external power and the other to the diesel-run generating set in the buildings. This generating set should automatically come into operation in the event of external power failure or fire in the building. The independent electric circuit from the generating set should be for all pumps including fire pumps, emergency lights, lifts, and lights in stair cases and yards.

10.9.4 MATERIALS FOR HOUSE SERVICE CONNECTIONS

The various pipes used for service connections should conform to the relevant Indian Standards.

(a) Normally G.I. pipes are used for service connections. They have the advantage of low cost and high strength. They suffer from the disadvantage of short life in corrosive soils especially at the screwed joints. Bituminous covering for the pipe increases its longevity. The carrying capacity of the pipe may also be reduced due to incrustation. Rigid PVC pipes as well as high density polyethylene pipes are also coming into use. These pipes are flexible and light and the carrying capacity is not reduced with age due to incrustation. They, however, are liable to be damaged easily. They also soften at temperatures above 65^0 C and as such cannot be used for hot water systems

(b) The communication pipe is attached to ferrules or saddles depending on the material of the distribution main in the street. Gun Metal or bronze ferrules are screwed into C.I. mains while special screwed saddles are fixed on cement asbestos and PVC pipes

(c) Since the minimum residual pressures in an area are to be maintained as indicated in 1.2.8.3, ferrules of suitable sizes are to be provided for adjacent buildings of different heights to get equitable supply

(d) Usually 12.5 or 18.75 mm rotary water meters are fixed on the service pipe immediately after the stop cock in the consumers premises and located in a masonry pit.

10.10 PREVENTIVE MAINTENANCE

10.10.1 GENERAL

Preventive maintenance of water distribution system pipelines assures the twin objectives of preserving the hygienic quality of water in the distribution mains and providing conditions for adequate flow through the pipelines. Some of the main functions in the management of preventive aspects in the maintenance of mains are assessment, detection and prevention of wastage of water from pipelines, maintaining the capacity of pipeline and cleaning of pipelines.

10.10.2 Waste Assessment and Detection

Wastage is due to leakage in water mains due to corrosion, fracture, faulty joints, ferrule connection, service pipes and fittings inside the consumer's premises due to joints, corrosion, faulty washers on glands in valves and taps; abandoned service pipes and ferrule connections in mains; and failure to turn off taps in premises willfully or inadvertently. Another important source of waste noticed in intermittent systems, particularly where metering is not enforced, is the tendency of the householder to keep the taps open throughout and also emptying stored water to replace by a batch of fresh water. Wastage is due to leakage from reservoirs and treatment plant which cannot be accounted for by the normal metering and can be as high as 40%. Pilot studies in a few cities in the country reveal that wastage in the mains alone can be 15 to 25%.

A systematic waste and leakage survey and detection, followed by prompt corrective action is of importance in bringing about a reduction in the wastage. The frequency and extent of the survey depends on the cost and the net benefits accruing therefrom.

(a) Assessment Of Waste

In residential areas where 24-hour supply is effected, it is possible to assess the total wastage occurring both in the water mains and the consumer's premises when the consumption is at a minimum which is likely to occur at midnight. The difference between the minimum night flow in the system and the accountable flow at midnight divided by the average daily flow at mid-night can provide the percentage of waste in an area. Levels of wastage upto 10% may be considered as low, 10 to 20% as average, 20 to 50% as excessive and over 50% as alarming. Remedial measures are called for, for levels above 10%.

In intermittent supplies, only leakages related to water mains are assessed. Waste in mains in such cases is assessed in a zone by closing all the taps or stop cocks in the house service connections. The percentage of wastage in intermittent supplies is the ratio of the flow in the mains (with stop cocks or tap closed) to the average daily domestic consumption.

Losses at about 5 to 7% may be considered as satisfactory while 10 and 20% as unsatisfactory and action is advisable, and beyond 20% level, remedial measures are positively indicated.

For any component of a water supply, the information on population, average daily flow, consumption by industry or trade, minimum night flow (in case of continuous supply) or flow in mains with all stop cocks or taps closed in intermittent supply, and transfer of flow from one zone to the other is required for estimation of the waste.

(b) Waste Survey Procedure

The approach of the problem requires careful planning and preparatory work and a large amount of routine field survey and investigations. Waste survey consists of the following steps:

1. Preparatory Work

It consists of:

(i) Delineation of zones and sub-zones of distribution network from field inspection and plans

(ii) Collection of statistics of population, houses, connections (metered and non-metered) of the selected zones

(iii) Location inspection, testing and repairing of valves, fittings, taps and meters

(iv) Correct alignment of pipelines by electronic pipeline locator and by inference

(v) Checking and updating of the distribution networks of zones and sub zones; and

(vi) Testing for isolation of zones and sub-zones from others by feeding water through a single feeder pipe with closure of all boundary valves of zones except the feed valve.

2. Waste Assessment

The steps involved are:

(i) Estimation of total daily consumption of the sub-zone by computation or by flow gauging and studying the water consumption pattern of sub-zone for the day;

(ii) 'Flow Test' for measurement of waste through the leaks by isolation of sub-zones and by means of an integrating type water meter or mobile waste water meter; and

(iii) 'Step Test' to assesses and localize the leakage in various parts of the sub-zone by internal valves.

The daily consumption pattern taken over a period of days can provide data for arriving at the actual average daily consumption of water in the area surveyed. These figures can be obtained through house meter readings, or by actual spot measurements by an integrating meter installed in the pipe feeding a group of houses in metered or unmetered areas. Otherwise, average daily consumption may have to be suitably assumed for the area.

A section or zone of water distribution system is isolated by allowing water to be fed into the zone through a single feeder pipe controlled by a valve. The zone is usually divided into workable sub-zones with a viable number of connections of about 150 to 300 in each sub-zone. Each sub-zone could also be isolated from the rest and be fed through a single entry pipe controlled by a valve. The boundary valves (i.e. the valves connecting the common pipes of two zones or sub-zones) are located in such a way that the water does not enter or flow out of the sub-zone from or to the adjacent ones.

The rate of flow in the zone or subzone is measured by a pitometer inserted in the pipe, if the feeder pipe and the flows are large. Otherwise, the flows are gauged by a mobile waste water meter, or integrating meter temporarily installed or by Deacon's water meter permanently installed in the system.

After gauging of the flows, the next step is to narrow down the area under test to localize the leakage in various parts. This is carried out by the 'step test' by noting the flow into the pipe system of sub-zone after every stepwise reduction in the size of the zone by closing the internal valves in each step.

The internal valves of a sub-zone are checked for water tightness by sounding over the spindle using the sounding rod under unbalanced pressure conditions created by supplying water through a single feed to the system with the direction of flow of water towards one face of the valve only. All the stop cocks or taps in the house service pipes are checked and if necessary rectified to ensure water tightness and complete cut off of the supply to consumers when stop cocks are closed.

The whole system must, as far as practicable, be brought to a 'tree' system by closing such valves in the mains in a loop during the test to prevent circulation. Then step wise isolation of mains in a zone or subzone is feasible and the possible sources and extent of wastage through leaks could be found within a short reach of the main.

(b) Leakage Detection

Leakage detection survey is confined only to the areas with heavy leakages as arrived at by the waste assessment survey. The survey consists of:

(i) Finding leaks in the pipes by visual determination of surface; and

(ii) Traversing the sub-zone in the night by sounding rod, or electronic leak locator for pinpointing of leaks in pipes.

Use of 'electronic pipe locator' can expedite the location and alignment of buried pipes particularly when the records for pipes in distribution maps are not adequate or complete. Some times, by physical inspection of valves or by occasional opening of trenches and through information obtained from valve operator or fitters, the alignment of buried pipes along streets could be made. Sounding rods alone or along with the electronic leak detector are traversed over the surface above the centre line of the alignment pipe for detecting noise generated by possible leaks in the mains. These are carried out usually at midnight when extraneous noise is minimum and the distribution system of the zone is also at a higher pressure.

Methods employing radioactive isotopes, nitrous oxide gas and halogens can easily and exactly pinpoint leaks but are not usually practiced in water works system as a routine measure since they require specialized equipment involving high costs.

Visual indications of leakage in pipes like dampness and stagnant water are noticeable in cases of large leaks, or even small leaks of pipelines located just below the surface depending upon the soil conditions.

The usual way to detect leaks in buried pipes without opening the road surface for visual inspection is by acoustic methods. The sound generated by the leaks through the overburden is picked up by the ear through the conventional sounding rod, stethoscope or the sophisticated electronic leak detector.

(c) *Instruments*

For flow and pressure measurements, location and alignment of pipes and detection of underground leaks through pipes, the following instruments are used and any water undertaking should possess some of the simple and a few of the sophisticated instruments.

(1) Pitometer Assembly

It consists of pitot tube with two orifices immersed in the flow of water and a differential manometer with scale along with special type of pitot-cock for fitting into the pipe. It is calibrated with reference to the pipe and single point velocity measurements are taken at the centre line of pipe

(2) Pressure Gauge (With Recorder)

Spring type pressure gauge is used to measure pressures at the inlet and various points on the zone. Recorder permits the continuous record of pressure with time.

(3) Integrating Type Water Meter

Normal integrating turbine type meter measures the flows between two hydrants connected by pressure hose serving as bypass before feeding into the zone or subzone. Normally 25 mm and 80 mm diametre are used.

(4) Mobile Waste Water Flow Meter

The integrating rate of flow type meter that can be mounted on a trailer is used for measuring the waste flow in a subzone. The rate of flow with reference to time is recorded on a drum chart.

(5) Hydrants and Hose Pipes.

These are required for bypassing the water in the feed pipe to the zone through the integrating of waste meter.

(6) Electronic Valve Box Locator

This is to locate buried metals underground upto a depth of about 0.25 to 0.5 m below the surface.

(7) Electronic Pipe Line Locator

By means of electromagnetic induction and wireless signals, the existence and exact alignment of underground metallic pipelines can be found.

(8) Sounding Rod

It is a 1.2 m long, 12 mm diameter hollow mild steel rod, flat or pointed at one end and fixed with cup shape brass cap of 50 mm diametre at the other. Bamboo canes can also be used. The rod is traversed over the surface along the centre line of pipe and the noises due to water leaking are picked up by the human ear thus locating the possible leaks.

(9) Electronic Leak Detector

It consists of a pickup, amplifier and headphone. The sound vibrations created by water escaping through leaks in pipes are selected and magnified by a magnetic pickup and converted to electrical impulses. These are sensitive and can pinpoint the position of the leaks.

(10) Road Measurer

This is a single wheeled integrating type roller facilitating the measurement of the length traversed as it moves along the road.

(d) Corrective Action

After location of the leaks in the pipes prompt repairs to pipes and valves are to be undertaken and 'Flow Test' of the sub-zones run to determine the extent and efficacy of the corrective measures. If re-testing proves that there are further leakages, they have to be attended to, until the losses in the zone are reduced to the minimum. The experience of waste assessment surveys indicates that a few major leaks in a zone or subzone contribute to about 75% of the total loss. Sizeable reduction in wastage can be brought about by locating and remedying promptly all such leaks first. Sometimes, it is prudent in a zone to go in for the sounding of probable leaks in pipes without being preceded by waste assessment.

The leaks are usually noticed at or near ferrule connections and in corroded G.I. house service pipes or in the joints of mains and house service pipes. The savings in water resulting from this programme more than offsets the investments. In addition to the favourable direct cost benefit analysis due to saving of water, the secondary benefits accruing out of such surveys are easy updating of distribution system drawings, maintaining valves, hydrants and stop cocks, the improved quality of water in the system due to prevention of back flow of pollution into the mains in non-supply hours and above all, the public goodwill earned due to the improved supply. Some of these cannot be exactly quantified.

10.10.3 CLEANING OF PIPES

The necessity for systematic and periodic cleaning of pipelines is borne out by the fact that the carrying capacity of the pipes gets reduced due to growth of slimes, incrustations or deposits. Flushing and swabbing of pipes, which are simple and inexpensive can go a long way in maintaining the capacity.

The old cast iron and steel pipes which are cleaned can be protected from further incrustations or corrosion by cement lining. Insertion of a plastic pipes has also been practiced with success.

Disinfection of the mains has been discussed in Appendix 5.8. This can be done at site specially for large diameter pipelines.

(a) Flushing

Water at high velocity is allowed to flow in the pipe and finally escape through a scour valve or hydrant. The minimum velocity to be induced varies from 90 to 120 cm/s and it is to be ensured that the flows are in one direction and the dirty water does not enter the cleaned sections. Flushing can only remove loose deposits of small size and not the slimy layers, large sized deposits and hard incrustations. Flushing also disentangles microscopic biological growths which, if left unattended, are likely to grow further and create problems. The period of flushing is determined by the quality of outgoing water in hydrants or valves. Usually this amounts to the flushing out of a volume of water equal to twice the capacity of the pipe length under consideration. About 100 to 300 m length of pipe can be flushed in one operation.

(b) Swabbing

The swab used is made of polyurethene foam of cylindrical shape and 30 to 60 cm long with varying diameters. It is soft and flexible, highly compressible and can retain the original shape when released from compression. Two varieties are available, one soft and the other relatively hard.

This swab is pushed into the pipe by the momentum of the flowing water. As the swab moves, it sweeps out the loose and slimy layer adhering to the inside of the pipelines and the deposits are carried away by the flowing water. Swabbing is not successful for dealing with hard deposits.

Swabs are slightly larger in diameter than the pipe to be cleaned. In certain cases with heavily incrusted pipes, swabs of the same diameter as the pipe are used initially. For pipe diameters of 75 to 100 mm, the swab diameter is usually 25 mm larger in size while for larger diameter pipe it is 50 to 75 mm larger.

For cleaning pipelines of 150 mm diametre or less, the following procedure is adopted. In distribution systems, where hydrants are connected vertically above a main without a duck-foot bend, the insertion of swab and its expulsion from the pipe are carried out at the hydrants. In situations, where the hydrants are laterally connected to the main, insertion of the swab has to be either through an existing valve in the line or by pumping water under pressure through the hydrant. The exit can be through another hydrant or a tee connected to the other end of the pipe and kept open.

The length of the main to be cleaned is isolated by valves. The swab is dipped in bleaching powder solution of strength 50 mg/l of chlorine prior to insertion. After insertion, the hydrant valve is closed or the valve body is covered. Water is allowed into the pipe by opening the valve near the hydrant and keeping the exit hydrant valve open, while the valve on the other side of pipe is kept closed. This ensures water flows in one reach only between the point of insertion and point of exist of the swab.

The movement of swab depends on the rate of flow or velocity of flush in the pipe which usually should not be less than 30 cm/s. If swab gets stuck or blocked in the pipe, water can be passed from the opposite direction in the pipe to release it.

As a permanent measure, tee-branches can be provided near the junction points of the pipe network preceded by the valves. (Fig. 10.2). These tee connections are covered by blank flanges. The tee can be vertical or horizontal and the outlet end with blank flanges can be enclosed in a chamber. Whenever swabbing or flushing is desired, the blank flange can be opened after closing the downstream valve and allowing the water and swab to escape through the tee.

For large diameter pipelines particularly greater than 300 mm, the existing valves have to be used for removal of the swab. Providing a tee connection in important large diameter mains may be a problem apart from being costly. To take out swab from such mains while used for cleaning, the top half of the valve is opened, and the water is allowed to escape through a grating provided. The swab gets stuck at the grating which can be then taken out conveniently.

10.11 PROTECTION AGAINST POLLUTON NEAR SEWERS AND DRAINS

10.11.1 Horizontal Separation

A water main should be laid such that there is at least 3 m separation, horizontally, from any existing or proposed drain or sewer line. If local conditions prevent this lateral separation, a water main may be laid closer to a storm or sanitary sewer, provided that the main is laid in a separate trench, or on an undisturbed earth shelf located on one side of the sewer at such an elevation that the bottom of the water main is at least 0.5 m above the top of the sewer.

10.11.2 Vertical Separation

In situations where water mains have to cross house sewer, storm drain, or sanitary sewer, it should be laid at such an elevation that the bottom of the water main is 0.5 m above the top of the drain or sewer with the joints as remote from the sewer as possible. This vertical separation should be maintained for a distance of 3 m on both sides measured normal to the sewer or drain it crosses.

10.11.3 Unusual Conditions

Where conditions prevent the minimum vertical separation set forth above from being maintained, or when it is necessary for the water main to pass under a sewer or drain, the water main should be laid with flanged cast iron pipe, with rubber gasket joints for a length on either side of the crossing to satisfy the lateral separation of 3 m. A vertical separation of 0.5 m between the bottom of the water main and the top of the sewer should be maintained, with adequate support for the larger sized sewer lines, to prevent them from settling on or breaking the water main. In making such crossings, it is preferable to have the sewer also of cast iron flanged pipe with rubber gasket joints and both the water and sewer mains pressure tested to assure water tightness before back filling.

Where a water main has already been laid and where a new sewer is to be laid, the above aspects may also be taken into consideration and the water main may be realigned to the extent necessary, when it is not possible to lay the sewer consistent with the above recommendations.

10.12 PROTECTION AGAINST FREEZING

Since water expands nearly about 10% in volume with an irresistible pressure, freezing solid conditions should not be allowed in any pipe system to avoid interruption of service and prevent damage to the pipes. Keeping water flowing is one of the simplest methods of preventing freezing. Keeping reasonable amount of flow in and out of an overhead reservoir with a stand pipe riser of 0.5 to 1 m diametre to prevent freezing solid in the riser pipe, may be resorted to in very cold climates. Water flowing at a velocity of 1.3 mps will not freeze

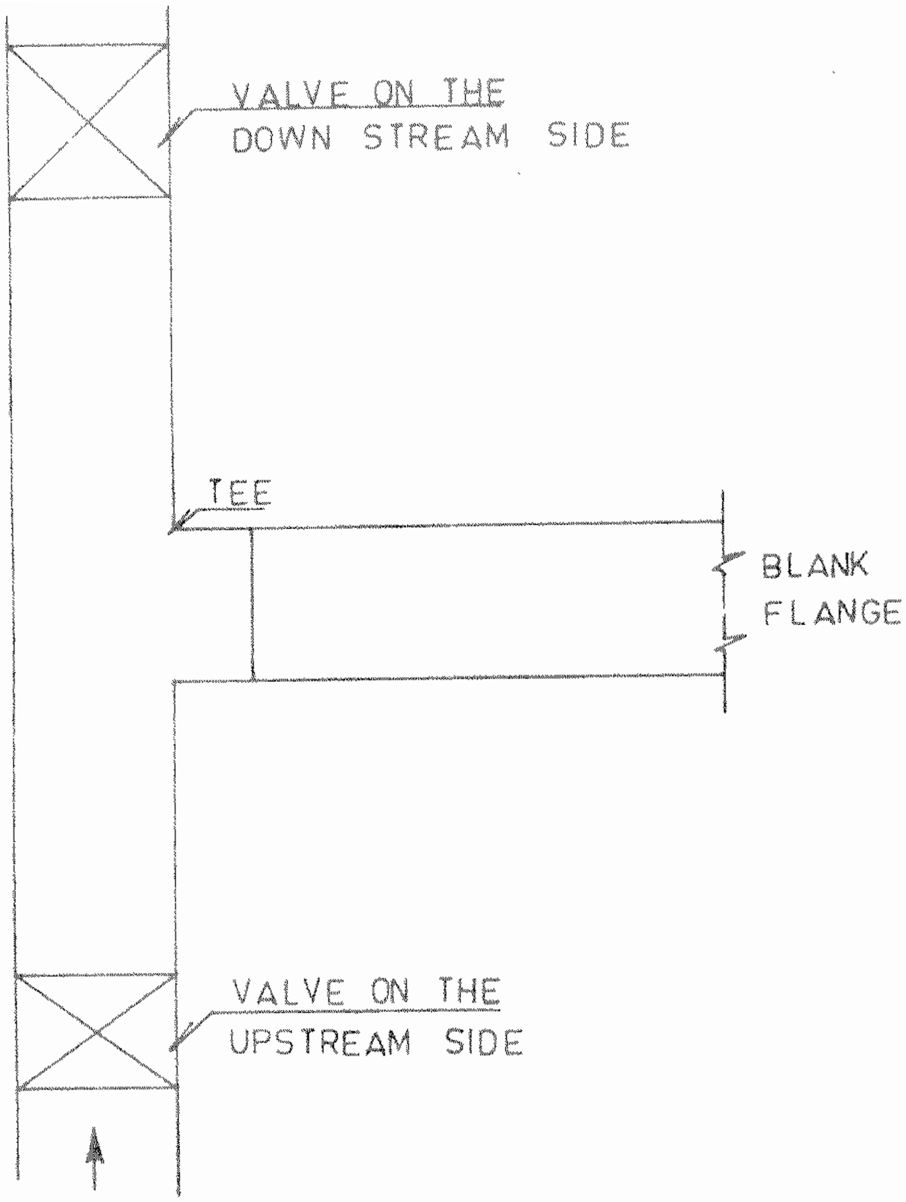

FIGURE 10.2 : THE BRANCHES TO ENABLE SWABBING

even in an unprotected welded steel pipe at 0^0C. With lower outside temperatures water is likely to freeze irrespective of the velocity in long main unless they are laid below the frost line. In Indian conditions, frost does not penetrate more than 2 m even in extreme cold areas. The cover of 1 to 1.5 m needed from the structural and traffic considerations should be adequate to take care of freezing also. Where water pipes are exposed to air temperatures below 0^0C, they are kept at a minimum size of 50 mm diametre and usually insulated, the common specification being three layers of standard hairfelt (total nominal thickness 75 mm) protected by means of weather proof wrapping of tarred roofing jacket lapped and sealed at all joints. This specification is suitable where water circulation is maintained continuously or where circulation is interrupted for only brief periods.

CHAPTER 11
PUMPING STATIONS AND MACHINERY

11.1 REQUIREMENTS

Planning and operation of a pumping station embraces considerations of the following points:

(i) Selection of the pump/s

(ii) Intake design

(iii) Piping layout

(iv) Providing space, equipment and facilities for

 (a) Substation, if needed, for receiving and distributing the power supply

 (b) Auxiliary power unit, generally diesel

 (c) Control panel

 (d) Bays for loading and unloading

 (e) Overhauling, repairs and maintenance of pumps and all other equipments

 (f) Head room and material handling tackle

 (g) Ventilation

 (h) Lighting

 (i) Safety from fire

 (j) Railings, ladders and passages for safe, easy and efficient movement of people.

 (k) Office and administrative areas, including room for lockers, dress change and utilities for sanitary and hygienic needs of the working staff.

(v) Installation of pumps

(vi) Operation of pumps

(vii) Maintenance of pumps

(viii) Trouble shooting of pumps

(ix) Selection of motors

(x) Selection of starters

(xi) Provisions in control panel

(xii) Selection of cables

(xiii) Planning of transformer substation

(xiv) Maintenance and repairs of the electrical equipments

(xv) Trouble shooting of the electrical equipments

Guidelines on the above are detailed in the following paragraphs.

11.1.1 SELECTION OF PUMPS

In a water supply system, pumping machinery serves the following purposes:

(a) lifting water from the source (surface or ground) to purification works or to the service reservoir;

(b) boosting water from source to low service areas and to the upper floors of multi storied buildings; and

(c) transporting water through treatment works, draining of settling tanks and of other treatment units, withdrawing sludge, supplying water especially water under pressure to operating equipment and pumping chemical solutions to treatment units.

While deciding the type of pump for the specific requirements, it is necessary to analyse different type of pumps and their suitability to meet the requirements.

11.1.2 TYPES AND CONSTRUCTIONS OF PUMPS

There are various ways of classifying pumps.

11.1.2.1 Pump Types Based On The Underlying Operating Principle

This analysis develops into a chain of classes and sub classes. However, broadly there are four classes, viz.

(a) Velocity (kinetic energy) adaptations as in centrifugal types, regenerative types and jet centrifugal combinations,

(b) Positive displacement pumps either reciprocating, such as simplex, duplex, triplex, etc., in piston, plunger and diaphragm types or rotodynamic, such as gear pumps, screw pumps, lobe pumps, vane pumps and peristaltic pumps, etc.,

(c) Buoyancy operated (air lift) pumps,

(d) Impulse operated, such as hydraulic rams,

Of these, the centrifugal pumps and the reciprocating type positive displacement pumps are more popular. Prominently, the reciprocating pumps are good on high head (high pressure) duties and for metering/dosing requirements. Centrifugal pumps, on the other hand are of mechanically simpler construction and give non pulsating continuous flow.

11.1.2.2 Pump Types Based On The Type Of Energy Input

Pumps need external energy to be input. It can be manual, as for hand pumps, from engines or from electric motors.

11.1.2.3 Pump Types Based On The Method Of Coupling The Drive

Pumps are coupled to the drives, direct through flexible couplings or are close coupled or are distantly driven through belt and pulley arrangement, sometimes with gearing arrangement or even with infinitely variable speed arrangement.

11.1.2.4 Pump Types Based On The Position Of The Pump Axis

Pumps normally work with their axis horizontal. Vertical turbine pumps, bore well submersible pumps and volute type sump pumps have their axis vertical. Dry pit pumps are often arranged to work with their axis vertical. In specific situation, pumps of the Archimedian screw-type are arranged with inclined axis also.

11.1.2.5 Pumps of Types Based On Constructional Features

For ease of maintenance, pumps are made with axially split casing or with back pull out arrangement. Pumps for high heads are built with multi staging. Pumps to handle solids and sewage are provided with access for inspection and cleaning the choking and also with the provision for flushing and draining. Submersible pumps to handle raw water should be with mechanical seals. In this manner, a large variety of constructional features are provided in pumps for different purposes in different situations.

Pumps are also made in a variety of materials, to withstand corrosion, erosion, abrasion and for longer life under wear and tear.

11.1.3 Criteria for Pump Selection

Prior to the selection of a pump for a pumping station, detailed consideration has to be given to various aspects, viz.:

(a) Nature of liquid, may be chemicals or if water, then whether raw or treated

(b) Type of duty required, i.e. Whether continuous, intermittent or cyclic

(c) Present and projected demand and pattern of change in demand

(d) The details of head and flow rate required

(e) Type and duration of the availability of the power supply

(f) Selecting the operating speed of the pump and suitable drive/driving gear

(g) The efficiency of the pump/s and consequent influence on power consumption and the running costs

(h) Various options possible by permuting the parameters of the pumping system, including the capacity and number of pumps including standbyes, combining them in series or in parallel,

(i) Options of different modes of installation, their influence on the costs of civil structural constructions, on the ease of operation and maintenance and on the overall economics

11.1.4 Considerations of the Parameters of Head, Discharge and Speed in the Selection of a Pump

These parameters are combined together in the term Specific Speed of a pump, which is calculated by the following formula

$$n_q = \frac{3.65NQ^{0.5}}{H^{0.75}} \tag{11.1}$$

Where,

n_q	=	Specific speed
N	=	The operating speed of the pump in rpm
Q	=	The rate of flow in cubic meters per second
H	=	The rated head per stage of the pump in meters

Most aspects of the performance characteristics of the different types of pumps can be compared, based on their specific speed. Some useful observations are summarized below.

(a) Positive displacement pumps are prominently used when high pressures high heads are to be developed or for metering/dosing duties as also handling sludge

(b) Centrifugal pumps are made with specific speeds above 36. Fig. 11.1 illustrates the relationships amongst the pump-efficiency, the shape of the impeller and the nature of the curves of Head (H) versus Discharge (Q), Power versus Q and Efficiency versus Q as influenced by the specific speed of the pump. The figure also helps in obtaining estimates of pump efficiency, which are useful in planning a pumping plant.

(c) For high discharges, by which specific speed becomes high the corresponding Net Positive Suction Head required (NPSHr see 11.1.4.1) also becomes high, it can be arranged that the discharge be shared by two impellers or by two sides of an impeller as in a double suction pump. While estimating the attainable efficiency for such pumps, only half of the total Q should be considered.

(d) Similarly, for high heads, by which the specific speed becomes low, and hence the attainable efficiency becomes low, it can be arranged that the head be distributed amongst a number of impellers as in multi stage pumps, thus improving the specific speed of each stage and consequently the attainable efficiency.

11.1.5 Consideration of the Suction Lift Capacity In Pump Selection

11.1.5.1 The Meaning Of NPSHr

The suction lift capacity of a pump depends upon its NPSHr characteristics. The meaning of NPSHr can be explained by considering an installation of a pump working under suction lift as illustrated in Fig. 11.2

When a pump, installed as shown is primed and started, it throws away the priming water and has vacuum developed at its suction. The atmospheric pressure acting on the water in the suction sump then pushes the water through the foot valve, into the suction line, raising

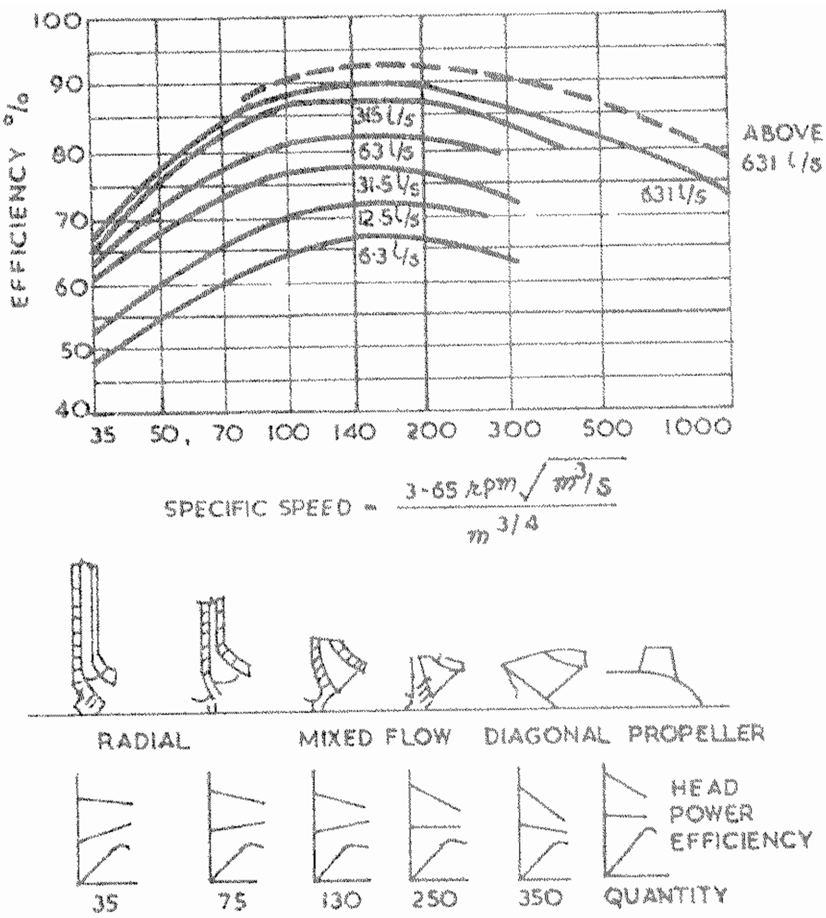

FIG. 11.1 SPEED AND EFFICIENCY CHARACTERISTICS

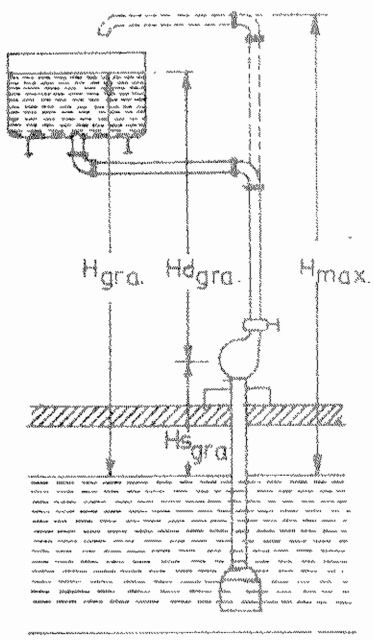

FIG. 11.2 SCHEMATIC REPRESENTATION OF NPSHr

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it upto the suction of the pump. While reaching upto the suction of the pump, the energy content of the water, which was one atmosphere when it was pushed through the foot valve would have reduced, partly in overcoming the friction through the foot valve and the piping and the pipe fittings, partly in achieving the kinetic energy appropriate to the velocity in the suction pipe and partly in rising up the static suction lift. The energy content left over in the water at the suction face of the pump is thus less than one atmosphere until here the flow is a fairly streamlined flow. But with the impeller rotating at the pump suction, the flow suffers turbulances and shocks and will have to lose more energy in the process. This tax on the energy of the water demanded by the pump, before the pump would impart its energy, is called the NPSHr of the pump.

The NPSHr characteristics of a pump is parabolic, increasing with flow rate.

Pumps of high specific speed have high NPSHr.

11.1.5.2 Vapour Pressure And Cavitation

The energy of the water at the pump suction, even after deducting the NPSHr should be more than the vapour pressure Vp, corresponding to the pumping temperature. The vapour pressures in meters of water column (mWC), for water at different temperatures in degrees Celsius are given in Table 11. 1.

TABLE 11.1
VAPOUR PRESSURE OF WATER

^{0}C	(mWC)
0	0.054
5	0.092
10	0.125
15	0.177
20	0.238
25	0.329
30	0.427
35	0.579
40	0.762
45	1.006
50	1.281

If the energy of the water at the pump suction would be less than the vapour pressure, the water would tend to evaporate. Vapour bubbles so formed will travel entrained in the flow until they collapse. This phenomenon is known as cavitation. In badly devised pumping systems, cavitation can cause extensive damage due to cavitation erosion or due to the vibration and noise associated with the collapsing of the vapour bubbles.

11.1.5.3 Calculating NPSHa

To insure against cavitation, the pumping system has to be so devised that the water at the pump suction will have adequate energy. Providing for this is called as providing adequate Net Positive Suction Head available (NPSHa). The formula for NPSHa hence becomes as follows.

NPSHa = Pressure on the water in the suction sump.

$$= P_s - Hf_s - \frac{Vs^2}{2g} - Z_s - V_p$$

P_s = suction pressure

Hf_s = friction losses across the foot valve, piping and pipe fittings

V_s = velocity-head at the suction face

Z_s = the potential energy corresponding to the difference between the levels of the pump-centre line and of the water in the suction-pump

V_p = the vapour pressure

While calculating NPSHa, the atmospheric pressure at the site should be considered, as the atmospheric pressure is influenced by the altitude of the place from the mean sea level (MSL). Data on the atmospheric pressure in mWC for different altitudes from MSL is given in Table 11.2.

TABLE 11.2

ATMOSPHERIC PRESSURE IN mWC AT DIFFERENT ALTITUDES ABOVE MSL

altitude above MSL in m	mWC
upto 500	10.3
1000	9.8
1500	9.3
2000	8.8
2500	8.3
3000	7.8
3500	7.3
4000	6.8

11.1.5.4 Guidelines On NPSHr

The NPSHa has to be so provided in the systems that it would be higher than the NPSHr of the pump. The characteristics of the pump's NPSHr are to be obtained from the pump-manufacturers. However some general guidelines for max. suction lift or min. NPSHa based on the type of a pump and based on the range of head and the specific speed are compiled in Figs. 11.3, 11.4 and 11.5.

11.1.5.5 General Observations

(a) Horizontal centrifugal pumps are installed with suction-lift.

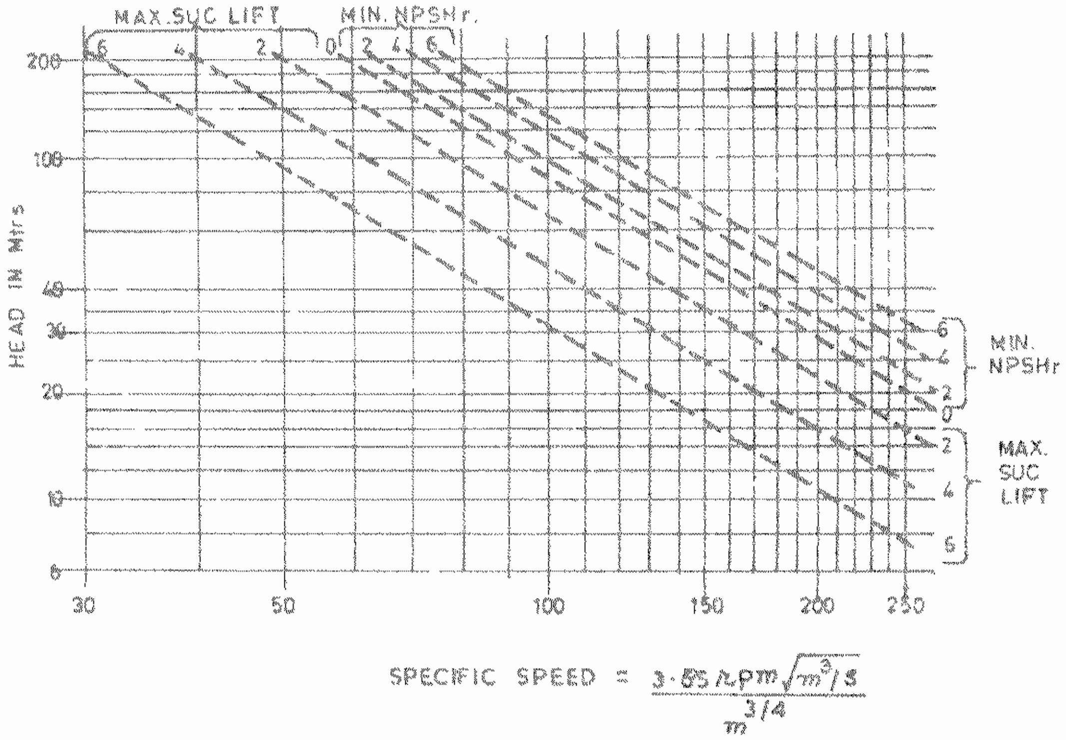


FIG. 11.3 NPSHr FOR SINGLE SUCTION PUMPS WITH OVERHUNG IMPELLER

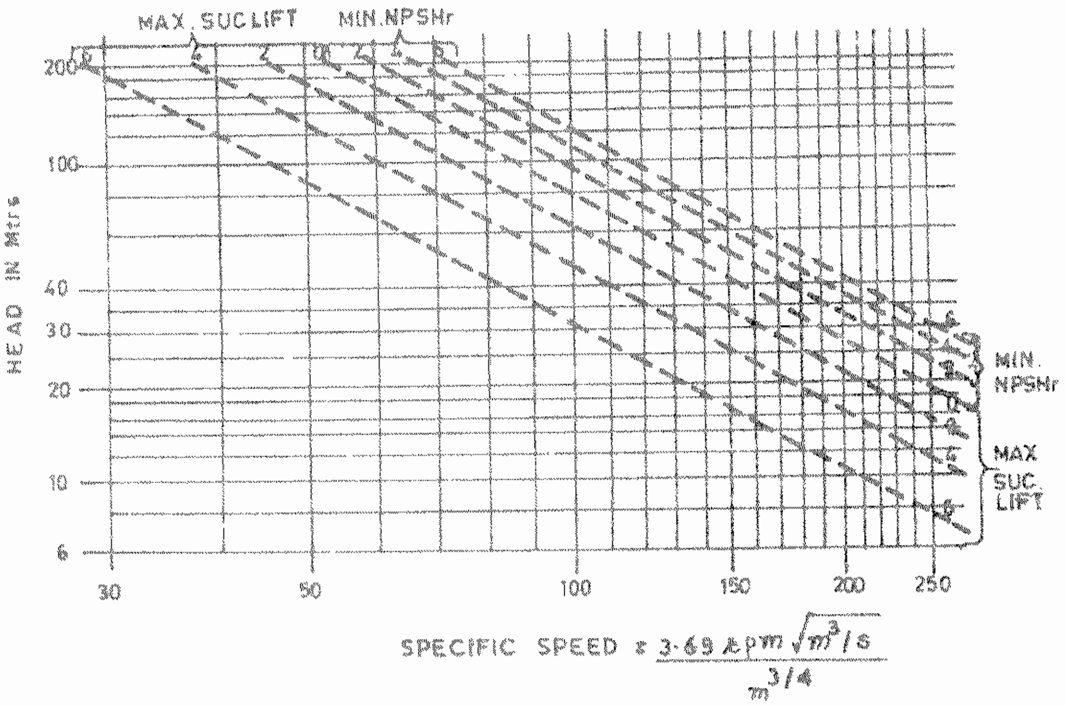


FIG. 11.4 NPSHr FOR SINGLE SUCTION PUMPS WITH SHAFT THROUGH EYE OF IMPELLER

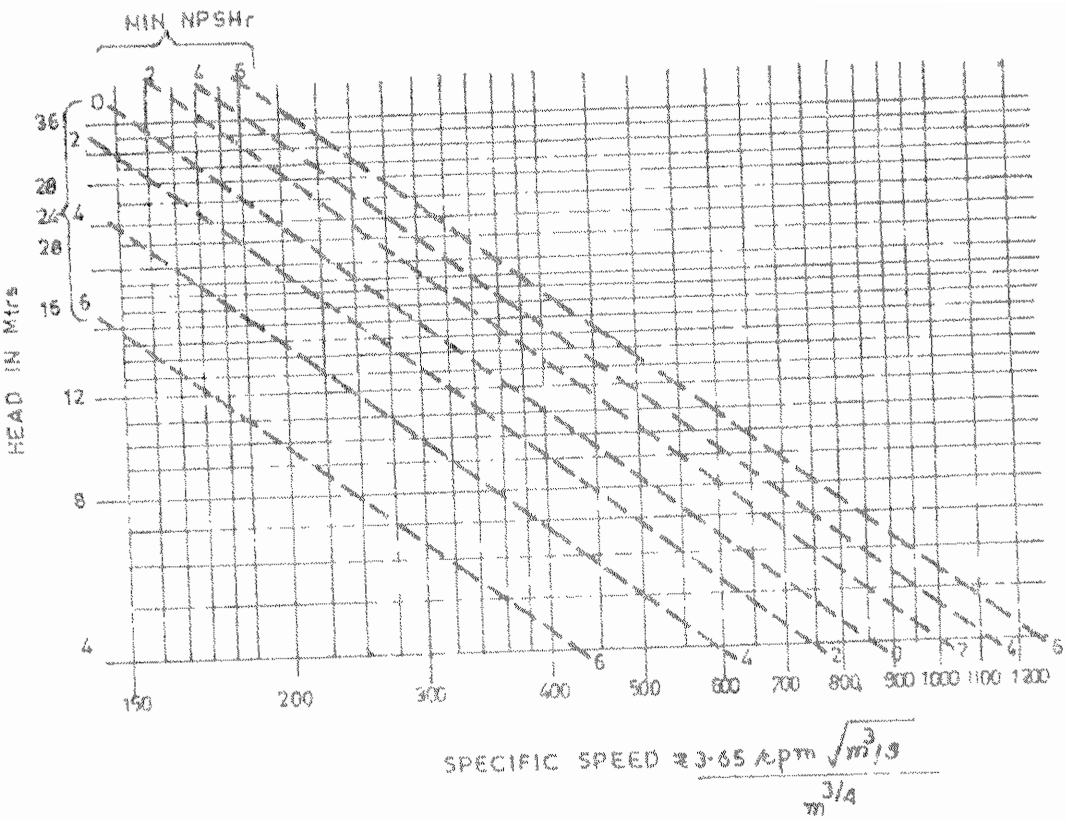


FIG. 11.5 NPSHr FOR SINGLE SUCTION, MIXED FLOW AND AXIAL FLOW PUMPS

(b) For vertical pumps, mainly of the vertical turbine type and of the bore well submersible type, suction lift has to be totally avoided. Even for these pumps, when the discharge required is high, they have to be installed providing the minimum submergence. The minimum submergence required may at times demand submerging more than the first stage of the pump. It should also be checked whether the submergence would be adequate for vortex free operation (see Table 11.3).

(c) Jet centrifugal combinations can work for lifting from depths upto 70m. However, the efficiency of the pumps is very low.

(d) Positive displacement pumps are normally self-priming. However this should not be confused with the NPSHr. Even if the NPSHa is not adequate, the pump may prime itself and run, but would cavitate.

11.1.6 Considerations of the System Head Curve in Pump Selection

A pump or a set of pumps has to satisfy the needs of the pumping system. Hence one has to first evaluate the head needed to be developed by the pump for delivering different values of flow-rate. A plot of these values is called as the System Head Curve. Each point on the System Head Curve denotes the head comprised of the following:

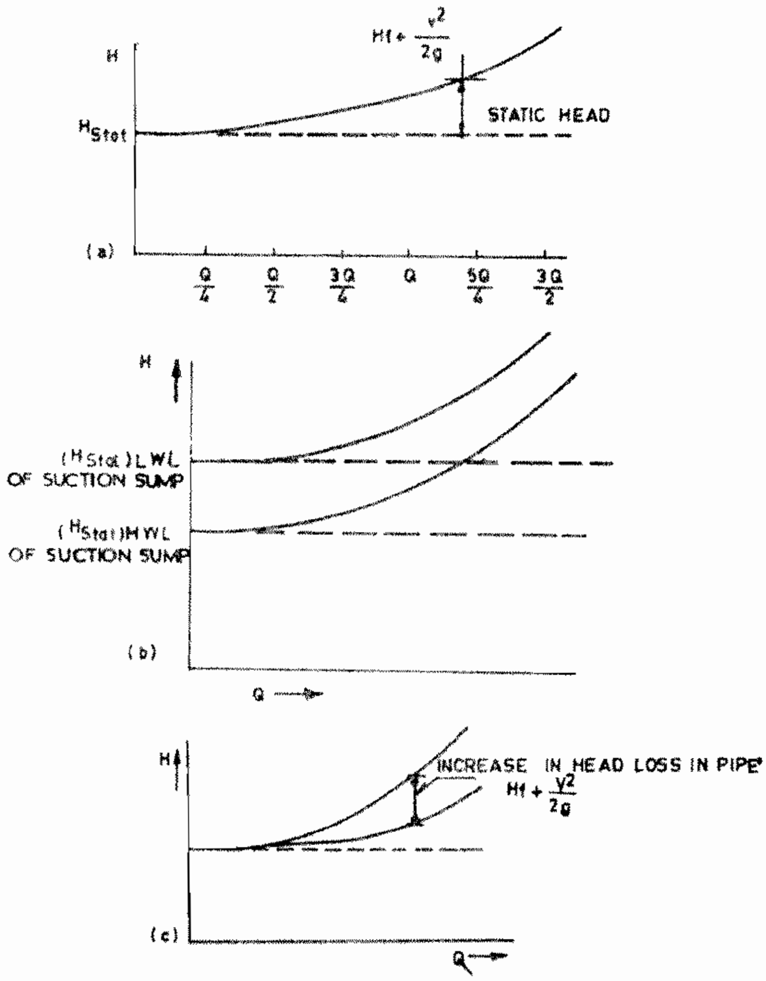


FIG. 11.6 SYSTEM HEAD CURVE

(a) Static Head

This is the difference between the level of the liquid in the suction-sump and the level of the highest point on the delivery piping, obviously the static head is more at the low water level (LWL) and less at the high-water level (HWL).

(b) Friction Head

This is sum of the head-losses in the entire length of the piping, from the foot valve to the final point of delivery piping, also the losses in all the valves i.e. the foot valve, the non-return (reflux) valve and the isolating (generally, sluice or butterfly) valves, and the losses in all pipe-fittings such as the bends, tees, elbows, reducers, etc. The friction head varies particularly with the rate of flow. Details for calculating the friction heads are given in Chapter 6.

(c) Velocity Head

At the final point of delivery, the kinetic energy is lost to the atmosphere. To recover part of this loss, a bell-mouth is often provided at the final point of delivery. The kinetic energy at the final point of delivery has also to be a part of the velocity head. Figs. 11.6 (a, b & c) show typical System Head Curves. As shown in Fig. 11.6(b) the System Head Curves for HWL and LWL are parallel to each other.

The system head curve will change by any changes made in the system, such as change in the length or size of the pipings, change in size and/or number of pipe fittings, changes in the size, number and type of valves by operating the valves semi-open or fully open. These changes can cause the System Head Curve to be steep or flat as shown in Fig. 11. 6 (c).

11.1.7 Summary View of Application Parameters and Suitability of Pump

Based on the considerations in 11.1.4 and 11.1.5, a summary view is compiled of the application-parameters and suitability of pumps of various types and presented in Table 11.3. However, these are general guidelines. Specific designs may either not satisfy the limits or certain designs may exceed the limits.

TABLE 11.3

APPLICATION OF PUMPS

Pump type	Suction-capacity to lift			Head range			Discharge range		
	Low 3.5m	Medium 6m	High 8.5m	Low Upto 10m	Medium 10- 40m	High Above 40m	Low Upto 30L/s	Medium Upto 500L/s	High Above 500L/s
Centrifugal, horizontal end-suction	Ok	Ok	Ok	Ok	Ok	No	Ok	Ok	No
Centrifugal horizontal axial split casing	Ok	No	No	Ok	Ok	No	No	Ok	Ok
Centrifugal, horizontal multistage	Ok	Ok	No	No	Ok	Ok	Ok	Ok	No
Jet-centrifugal, combinations	When limitations of suction lift are to be overcome			Ok	Ok	No	Ok	No	No
Centrifugal, vertical turbine	when suction lift is to be avoided			Ok	Ok	Ok	Ok	Ok	Ok
Centrifugal, vertical submersible	when suction lift is to be avoided			Ok	Ok	Ok	Ok	Ok	Ok
Positive displacement pumps	Normally self priming			Limited only by the pressure which casing can withstand			Ok Ok No Easy adaptation for dosing or metering		

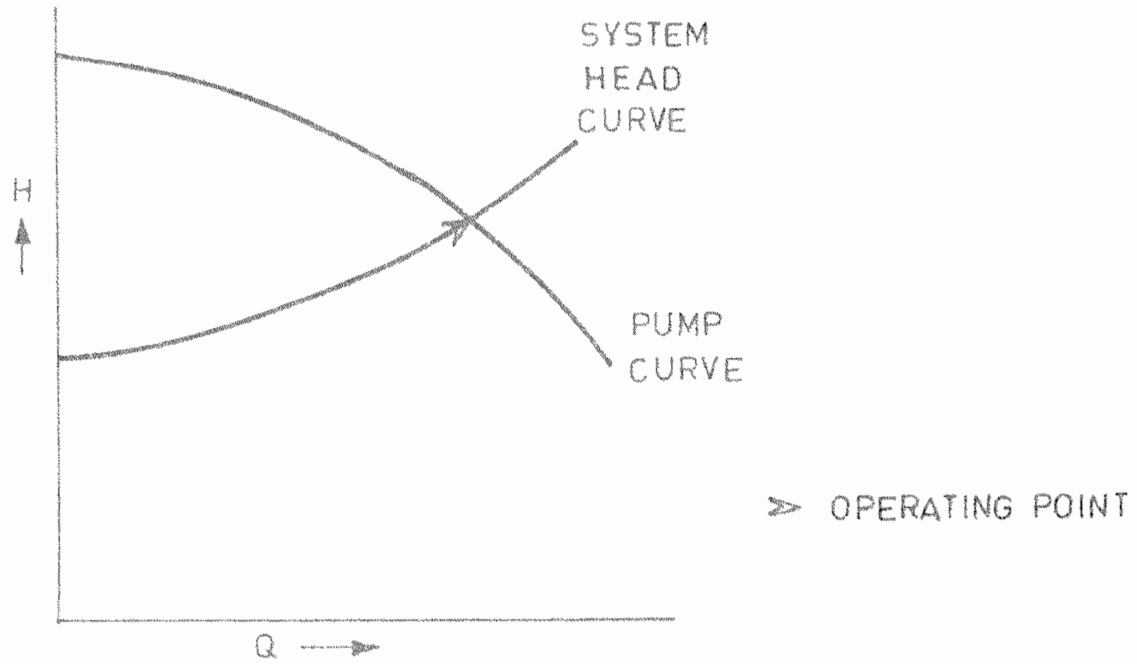


FIG. 11.7(A) OPERATING POINT OF THE PUMP

11.1.8 Defining the Operating Point or the Operating Range of a Pump

The operating point of a pump is the point of intersection of the System-Head Curve with the H versus Q characteristics of the pump. See Fig. 11.7 (A). Shifting of the System-Head Curve will cause a change in the operating point of the pump. Hence, the following points are worth noting;

(a) If the level of water in the suction sump would deplete during pumping from HWL to LWL, the operating point of the pump would vary from a low-head-high discharge point to a high head low-discharge point. See Fig. 11.7 (B).

(b) If in a pumping system, the discharge is re circulated to the suction sump, as is often the case at the testing setups at the manufacturers' works, the throttling of the delivery valve from full open to close, shifts the system-head curve from a flat curve, intersecting the pump's H-Q curve at high flow initially to a steep system head curve intersecting the pump's H-Q curve at high head. See Fig. 11.7 (C).

Similarly, a pumping system can be devised with flat or steep system-head curve. Alternatively, throttling the delivery valve would shift the system-head curve from flat to steep.

The most average water level in the suction sump and the most average system-head curve would define the operating point of the pump. For such operating point of the pump, the pump should have its point of maximum efficiency at or nearest to it. To provide for marginal changes in the operating point, e.g. between HWL and LWL, the nature of the efficiency characteristics of the pump should be as flat as possible in the vicinity of the point of its best efficiency, often called as the BEP.

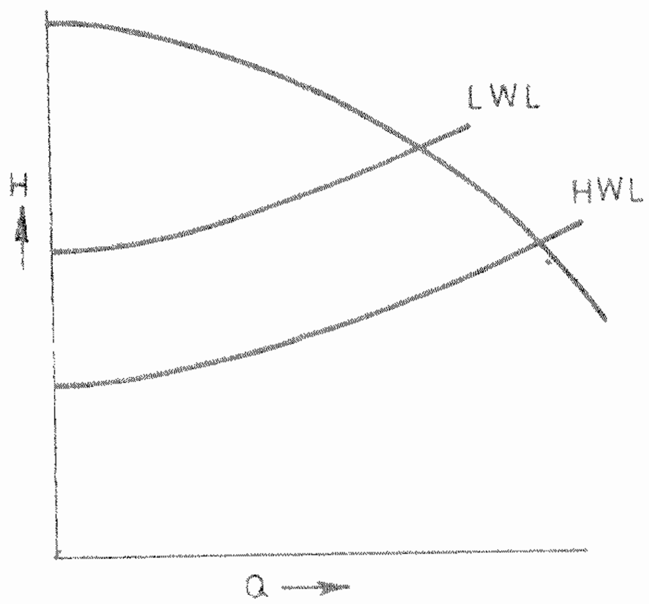


FIG. 11.7(B) CHANGE IN OPERATING POINT OF PUMP WITH CHANGE IN WATER LEVEL IN SUCTION SUMP

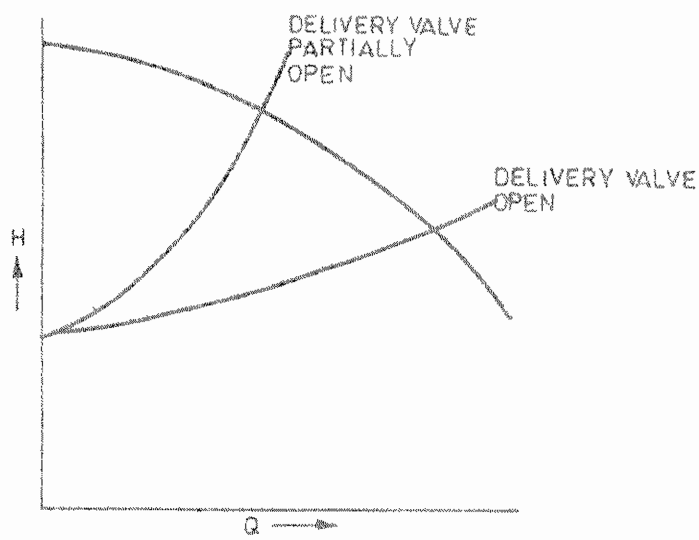


FIGURE 11.7(C) : CHANGE IN OPERATING POINT OF PUMP BY OPERATION OF DELIVRY VALVE

(c) When specifying the operating point of the pump, margins and safety factors, especially in specifying head should be avoided. On providing margins and safety factors, the rated head for the pump would work out high. In actual running the pump would work at a head less than the rated head and yield high discharge. From Fig. 11. 1, it would be noted that the Power versus Q characteristics of pumps of specific speeds upto 300 is with positive gradient, hence demanding more power at higher discharge. By such higher power demand, the drive may get over loaded.

By working at high discharge, the NPSHr demanded by the pump would be higher. If NPSHa is not adequate for this higher NPSHr, the pump may cavitate.

Due to the high discharge included, the pump may vibrate. Sometimes this may result in serious damage to the shaft and bearings.

11.1.9 Drive Rating

After the operating point of a pump is decided as discussed in 11.1.7, the efficiency of the pump can be estimated from Fig. 11. 1. The rating of the drive should be such that it would not get overloaded when the pump would be delivering the high discharge, as with HWL in the suction-sump. Also, the drive rating should be adequate to provide for the negative tolerance on efficiency and the positive tolerance on discharge, applicable for variations in actual Pump-performance from the rated performance.

The power needed to be input to the pump is the power to be output by the drive, i.e. at the pump-shaft. Since, most drives are coupled direct to the pump, the power at the pump-shaft denotes the brake power of the drive. All drives are rated only as per their brake power capacity, often quoted in Brake Kilowatts (BKW).

To provide margins over the BKW required at the operating point, so that the overloading would not happen at HWL, the following margins are recommended.

TABLE 11.4

MARGINS TO DECIDE DRIVE RATING

BKW required at the operating point	Multiplying factor to decide drive rating
upto 1.5	1.5
1.5 to 3.7	1.4
3.7 to 7.5	1.3
7.5 to 15	1.2
15 to 75	1.15
above 75	1.1

11.1.10 Stability of Pump Characteristics

In the H-Q characteristic of the centrifugal pump, the flow reduces as the head increases. If the head increases continuously until zero flow or until full close i.e., shutoff of the delivery valve, as shown in Fig. 11.8 (a) the H-Q characteristic is said to be stable. However, it is also probable that the shut off head of a pump may be less than the maximum head, as shown in Fig. 11.8 (b) which may be realized at some positive flow. Such a characteristic of a pump is called as unstable characteristic. When operating such a pump at any head between the shut-off head and the maximum head, the flow will keep hunting between two values. Because of this, the performance of the pump becomes erratic and unstable.

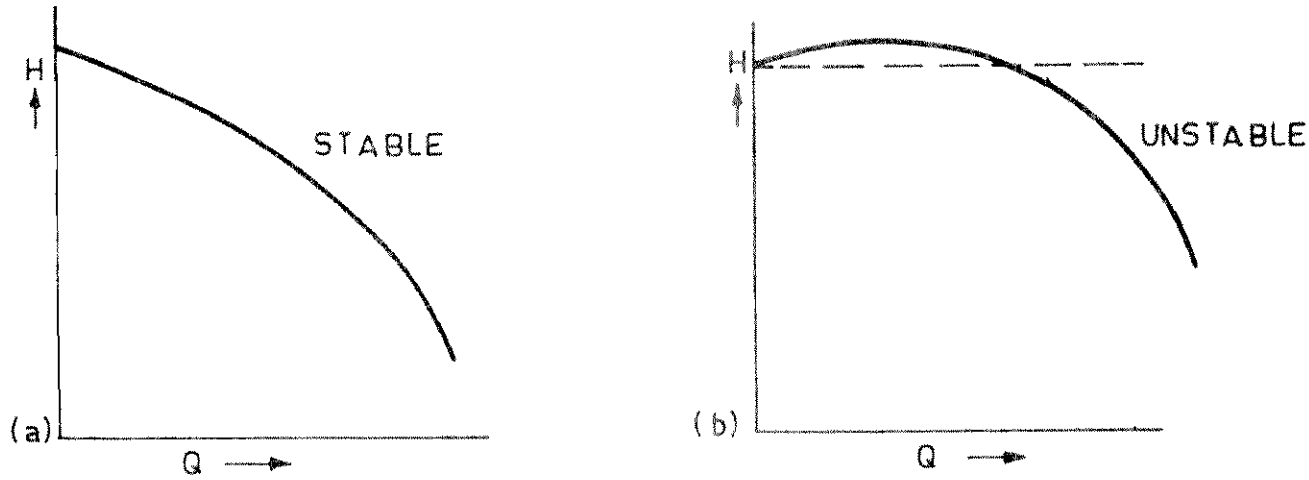


FIG. 11.8 STABLE AND UNSTABLE CHARACTERISTICS OF PUMPS

While selecting a pump, it ought to be checked that the highest head by the intersection of the system head curve would be less than the shut off head, in the case of pumps with unstable characteristics.

11.1.11 Considerations while Selecting Pumps for Series or Parallel Operation

(a) When pumps are to run in parallel, to obtain the combined H-Q characteristics, for different values of head, the values of the flow of individual pumps are to be found and to be added. See Fig. 11 .9 (a). The system head curve then intersects the combined H-Q characteristics at higher head and discharge. Each individual pump ought to be capable of developing such high head, that too within its zone of stability. Rather, it is always desirable to put into parallel operation only pumps having stable H-Q characteristics.

(b) A pumping system is often sought to be modified to meet increasing demand by commissioning additional pumps in parallel. It must be noted however that because the system head curve intersects the combined H-Q curve at a point having the head also higher, an additional pump would not increase the discharge proportionately, i.e. by making two identical pumps to work in parallel, when one is previously operative, the discharge would not double.

(c) Conversely, if a system is to run with a number of pumps in parallel, but is modified to run with only a few of the pumps as in summer, for example, then the duty flow of each pump becomes more than when all the pumps be running. The individual pump would demand higher NPSHr at the higher duty flow. If the

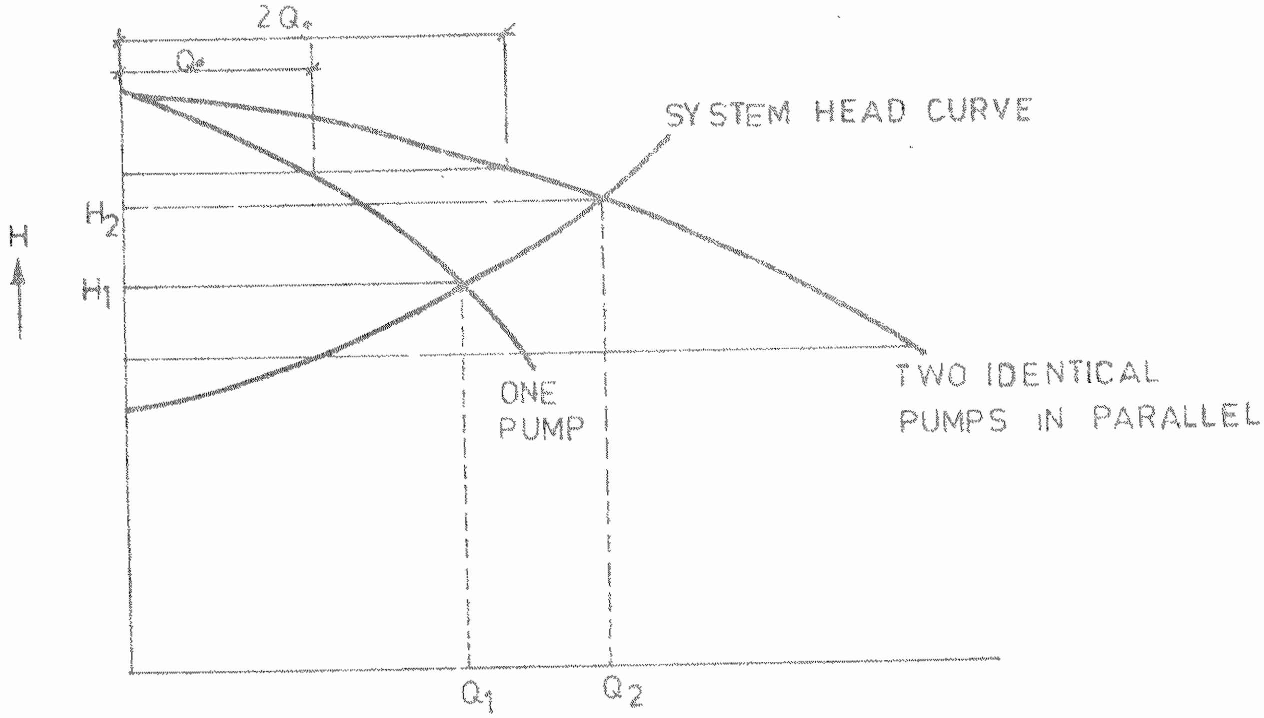
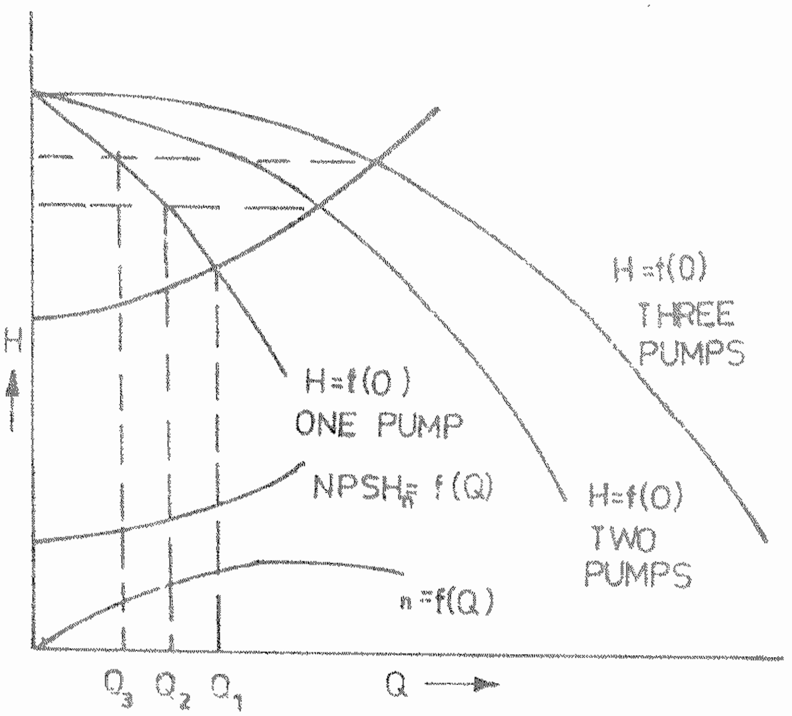


FIG. 11.9 (A) COMBINED CHARACTERISTICS OF TWO PUMPS IN PARALLEL



NO. OF PUMPS IN OPERN.	FLOW OF EACH PUMP.
1	Q_1
2	Q_2
3	Q_3

FIGURE 11.9(B) : ONE OR MORE PUMPS IN PARALLEL

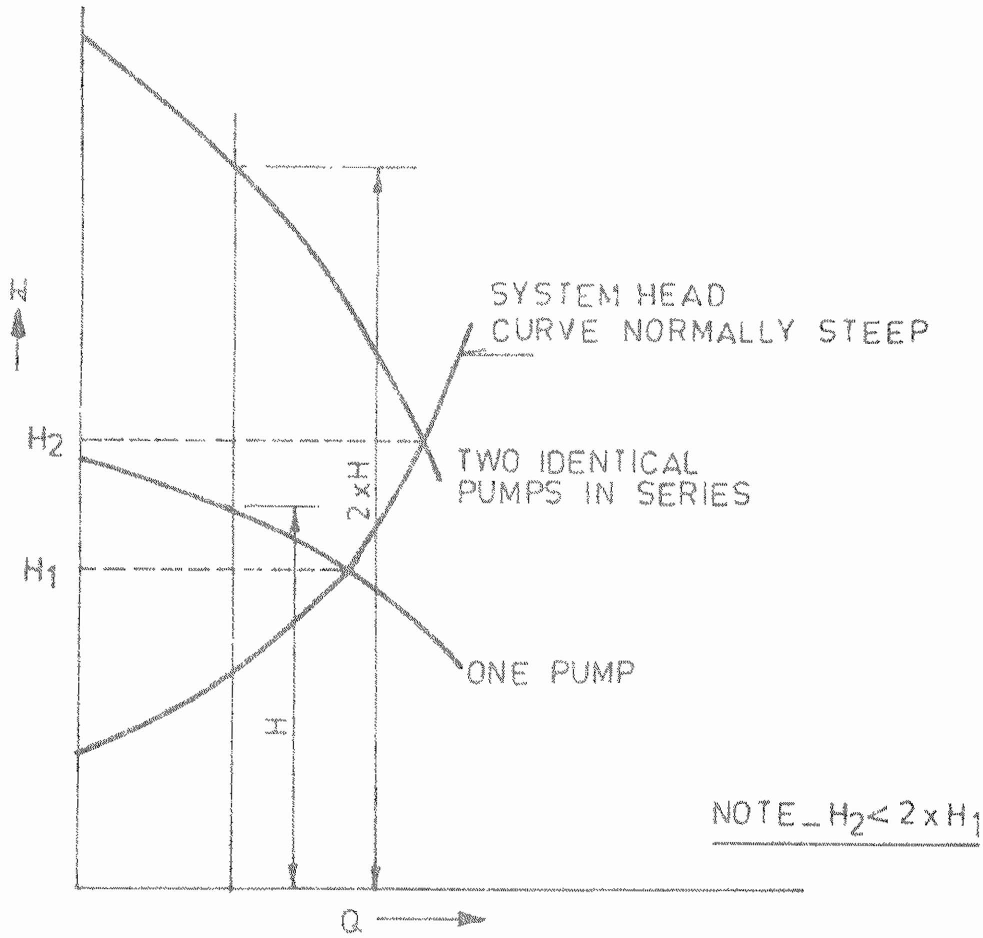


FIGURE 11.10 : SERIES OPERATION OF PUMPS

NPSHa would not be adequate, the pump/s would cavitate. To prevent such possibility, individual pumps, which are to be put into parallel operation, would be so selected that the duty flow of combined parallel operation would be to the left of the BEP of the individual pump. By this, when only a few pumps are to run, the duty flow of the individual pump would shift to the higher flow nearer to its BEP Fig. 11.9 (b).

(d) Pumps in series are similar to multi stage pumps. Rather, multi stage pumps are only a compact construction, where series operation is inbuilt. To obtain the combined H-Q characteristics of pumps in series, for various values of discharge, the values of head from the H-Q characteristics of individual pumps are to be noted and added . The system-head curve would intersect the combined H Q curve at a point of higher head and discharge. See Fig. 11.10. The individual pump in this case ought to be capable of giving the higher discharge.

If the system head curve comprises high static head and a flat curve, the intersection at higher discharge on the combined H-Q characteristics may be at such discharge where the NPSHr of the individual pump would be high and the pump/s may cavitate.

Series operation is most appropriate, where the system-head curve is steep.

For the pumps to be put in series operation, each pump should be capable of withstanding the highest pressure that is likely to be developed in the system.

The head towards the potential difference between the centre-line of one pump and the suction of the next pump, plus the friction losses in the pipeline between the delivery of one pump upto the suction of the next pump has to be considered as a part of the total head of the pump giving the delivery. In a series system, the total head of each pump may have to be individually calculated, especially when the features contributing to head calculations are significantly different, as in the case of booster stations along a long conveyance pipeline.

11.1.12 Considerations of the Size of the System and the Number of Pumps to be Provided

(a) For small pumping systems, generally of capacity less than 15 mld, two pumps (One duty and one standby) should be provided. Alternatively, two duty and one standby, each of 50% capacity may be provided. Although this alternative would need larger space, it facilitates flexibility in regulating the water supply. Also, in an emergency of two pumps going out of order simultaneously, the third helps to maintain at least partial supply.

(b) In the case of medium and large pumping stations, at least two standby should be provided.

11.1.13 Considerations Regarding Probable Variations of Actual Duties from the Rated Duties

11.1.13.1 Affinity Laws

The running speed of the electric induction motors is at a slip from its synchronous speed. The running speed of the motor is also influenced by variations in the supply frequency. Since the pump characteristics furnished by the pump manufacturers is at certain nominal speed, depending upon the actual speed while running, the actual pump performance would be different from the declared characteristics. Estimates of the pump performance in actual running can be worked out from the declared characteristics, by using the following affinity laws.

$$if\ \frac{n''}{n'} = k,\ then \frac{Q''}{Q'} = k;$$

$$\frac{H''}{H'} = k^2;\ then \frac{P''}{P'} = k^3;$$

In the above formulae, n denotes the speed of the pump, p denotes the power input to the pump, the superscript " denotes the values at the actual speed and the superscript ' denotes the values at the nominal speed.

Recalculating the pump-performance at the actual speed would reveal the following.

(a) If the actual speed is less than the nominal speed, then the values of the discharge, head and power required to be input to pump would all be less than the values from the declared characteristics.

(b) Similarly, if the actual speed is more than the nominal, it should be checked that the higher power input required would not overload the motor.

(c) When the actual speed is more, because the discharge is also correspondingly more, the NPSHr would also be more, varying as per the following formula.

$$\frac{NPSHr''}{NPSHr'} = k^2$$

11.1.13.2 Scope For Adjusting The Actual Characteristics

To avoid overloading or cavitation, marginal adjustment to the pump performance may be done at site, either by employing speed-change arrangements or by trimming down the impeller. The modifications in the performance on trimming the impeller can be estimated using the following relations:

$$if \ \frac{D''}{D'} = k, \ then \frac{Q''}{Q'} = k;$$

$$\frac{H''}{H'} = k^2; \quad then \frac{P''}{P'} = k^3;$$

Such modifications are recommended to be done within 10 to 15 percent of the largest diameter of the impeller. The percentage depends upon the design, size and shape of the impeller. The pump manufacturer should be consulted on this.

11.1.14 PUMP TESTING

The objective of pump testing is to verify that the performance characteristics of the pump are appropriate for the service desired.

The testing is done both at the manufacturers' works and only for preventive maintenance and in the field, with the following limitations:

(1) The testing at the manufacturers' work is done with water under ambient conditions. It is not practical for the manufacturer to provide the service fluid to be the test fluid. It is also not practical to exactly duplicate the site conditions viz. suction sump, piping layout, atmospheric pressure, fluid temperature and pressures etc.

(2) For the testing at the site, it is often impractical to provide adequate instrumentations of appropriate class of accuracy. Setting up the instrumentation

may disrupt the on-line operation of the pump. Field test of the pump has to be scheduled considering when the disruption of the on-line operation can be tolerated. Apart from the disruption, certain temporary modifications will be needed to introduce flow-measuring devices like orifice plates, etc. in the line. In situations where service-fluid is likely to entrain solids, this is likely to cause the measuring instruments either to give erratic reading or even suffer damage. Then the field test may not be feasible at all. The field test even where feasible, is often done to keep a track of deterioration in efficiency due to increase in running clearances, particularly at the wearing rings. The objective of the field test is one of preventive guidelines and not one of obtaining very elaborate details of the pump characteristics.

(3) Since the testing at the manufacturers' works is done with water under ambient conditions, the duties desired with service-fluid have to be translated to equivalent duties with water under ambient conditions. In the Standards on testing, viz. IS: 9137-1978 or IS:10981-1983 permissible tolerances for the variation of test results from guaranteed duties are also given. Out of these two standards, IS: 9137-1978 details class C code of testing and IS:10981-1983 details Class B code of testing. The Class B code of testing specifies narrower band for tolerance. The implicit stringency affects both the cost and the period of delivery. The class C code of testing is the most widely followed and adequate in most of the cases.

The scheme of testing includes taking readings, doing calculations and plotting of

(i) the H-Q characteristics

(ii) the P-Q characteristics and

(iii) the efficiency versus Q characteristics.

The actual speed of the shaft at the time of each reading would be different from the nominal speed. The value of the total head, flow-rate and power-input are to be converted to the nominal speed, using the affinity laws.

The readings of power-input, noted during testing are often the values of power input to the motor. Values of power-input to the pump have to be derived by multiplying the values of power input to the motor with the appropriate values of motor-efficiency.

For the values of motor-efficiency, reference has to be made to the motor-characteristics. Often these are available as motor output to motor-efficiency relationship. Since the readings during the test are for the motor input, the motor-characteristics need to be converted into the appropriate motor-input to motor-efficiency relationship.

After the performance-characteristics are plotted, an assessment has to be made to check whether the plottings reveal variations from the guaranteed duties. The pump can be approved if the variations are within the permissible tolerances.

It may be noted that the tolerances specified in IS: 9137-1978 and IS:10981-1983 give limits also for positive variances. However, in most water-supply situations, positive variances on discharge and efficiency would not be critical, if the motor would not get overloaded. This aspect, is so provided in IS:11346-1985, which deals with testing of pumps for agricultural purposes. The technical provisions therein can be extended to pumps for water supply.

Only occasionally the testing is extended to cover testing the NPSHr characteristics of the pump. Basically care is always to be taken to provide NPSHa such that it has adequate margin over NPSHr at all flow rates in the operating range. Hence the data of NPSHr provided by the manufacturer need not be verified by an actual test. This is so advocated considering that

(i) conducting test for NPSHr requires elaborate and often special arrangement on the test bed and becomes costly and time consuming,

(ii) even on readily available test rigs, the actual conducting of the test itself becomes time consuming and exerting and a cost-element,

(iii) the variations from the declared data are mostly on the safer side.

However, if the site-plan is laden with such constraints that NPSHa cannot have adequate margins over NPSHr, then testing for NPSHr may be stipulated very clearly in the purchase specifications. Unless stipulated, routine testing of a pump does not comprise in its scope the test for NPSHr.

11.1.14.1 Testing At Site

At site the testing is done soon after installation to assess whether any adjustments are required to the pump characteristics as detailed in 11.1.13.2. Further testing is done at site , mostly once in a year to assess whether there is any deterioration in the performance of the pump due to wear and tear.

The objective of the field test is to serve as a timely caution for preventive maintenance and not one of obtaining very elaborate details of the pump-characteristics.

During the testing at site, it is often impractical to provide adequate instrumentation of appropriate class of accuracy. Setting up the instrumentation may disrupt on-line operation of the pump. Apart from the disruption, certain temporary modifications may be needed to introduce flow-measuring devices like the orifice plates, etc. in the line. Field test has to be scheduled considering when the disruption of the on-line operation can be tolerated.

11.2 INTAKE DESIGN

11.2.1 THE OBJECTIVES OF INTAKE DESIGN

Detailed consideration needs to be devoted to the intake design to serve various objectives, as follows:

(a) to prevent vortex formation,

(b) to obtain uniform distribution of the inflow to all the operating pumps and to prevent starvation of any pump,

(c) to maintain sufficient depth of water to avoid air entry during draw down.

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11.2.2 GUIDELINES FOR INTAKE DESIGN

Figs.11.11 (a, b, c, d and e) illustrate the recommended and the not-recommended practices for pump or intake design. Following points are to be noted in this respect. Note, D is the diameter of the suction bell mouth.

(a) Avoid mutual interference between two adjoining pumps by maintaining sufficiency clearance, the dimension 'S' in Figs. 11.11 (a) and (b) equal to 2D to 2.5 D. It is also advisable to provide dividing walls between the pumps, as shown in Fig. 11.11 (b). The walls should have rounded or ogive ends.

(b) As shown in Fig. 11. 11 (b), avoid dead spots by keeping rear clearance, the dimension 'B' to about 5D/6 from the centre line of the pump. A dummy wall may be provided, if necessary.

(c) As shown in Fig. 11. 11 (c), provide tapered walls between the approach channel and the sump. By this the velocity should reduce gradually to about 0.3 m/s near the pump. This also helps to avoid sudden change in the direction of the flow.

(d) Avoid dead spots at the suction bell mouth by maintaining the bottom clearance, dimension C in Fig. 11.12, between D/4 to D/2, preferably D/ 3. The suction flow becomes guided by providing vertical splitters under the centre line of the pump. See Fig. 11.13 (a). A cone may be added to reduce the possibility of submerged vortex formation. See Fig. 11.13 (b).

(e) Avoid sudden drop between the approach channel and the sump. A slope of maximum 15^0 is recommended as shown in Fig. 11.12. The floor underneath the pump suction should be flat upto 3D.

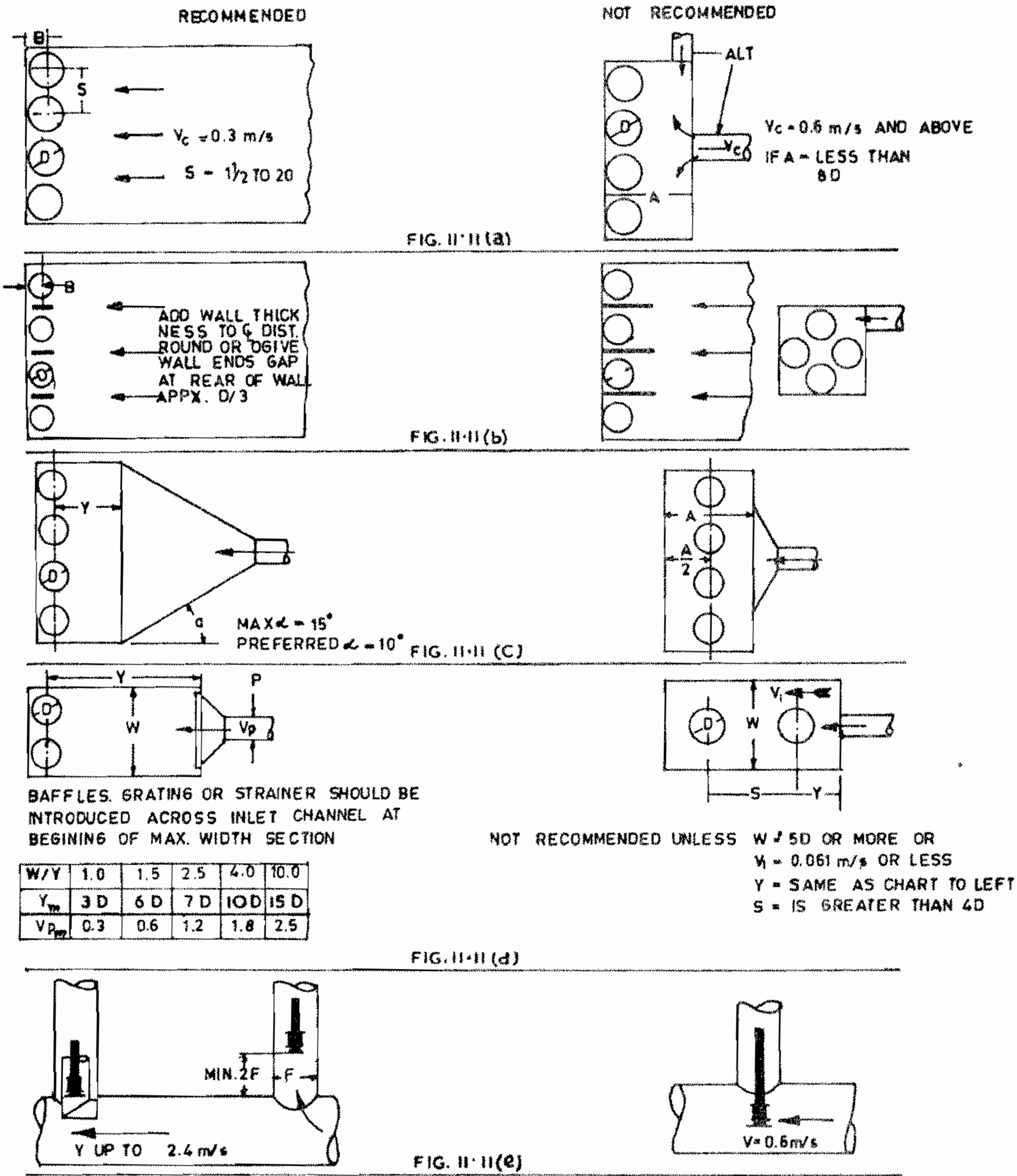
(f) Keep adequate submergence of the pump under the LWL, dimension H in Fig. 11.12, so as to prevent entry of air during draw-down and to satisfy NPSHr.

For the typical proportions illustrated in Fig. 11.12, recommendations for the values of the dimensions A and H, based on the recommended main-stream velocity of 0.6 m/s are given in Table 11.5.

The Dimension D is generally the diameter of the suction bell measured at the inlet . This dimension may vary depending upon pump design. Refer to the pump manufacturer for specific dimensions.

TABLE 11 .5
RECOMMENDATIONS REGARDING INTAKE-DESIGN

Flow-rate in m^3/hr	Minimum submergence Dimension 'H', m	position of trash-rack dimension 'A', m
1000	1.23	3.28
1600	1.50	5.20
2500	1.80	8.07
4000	2.17	12.83
6400	2.63	20.37
10000	3.15	31.61
16000	3.80	50.20
25000	4.56	77.91
40000	5.52	123.74



W/Y	1.0	1.5	2.5	4.0	10.0
Y_m	3 D	6 D	7 D	10 D	15 D
Vp_m	0.3	0.6	1.2	1.8	2.5

FIGURE 11.11 : MULTIPLE PUMP PITS

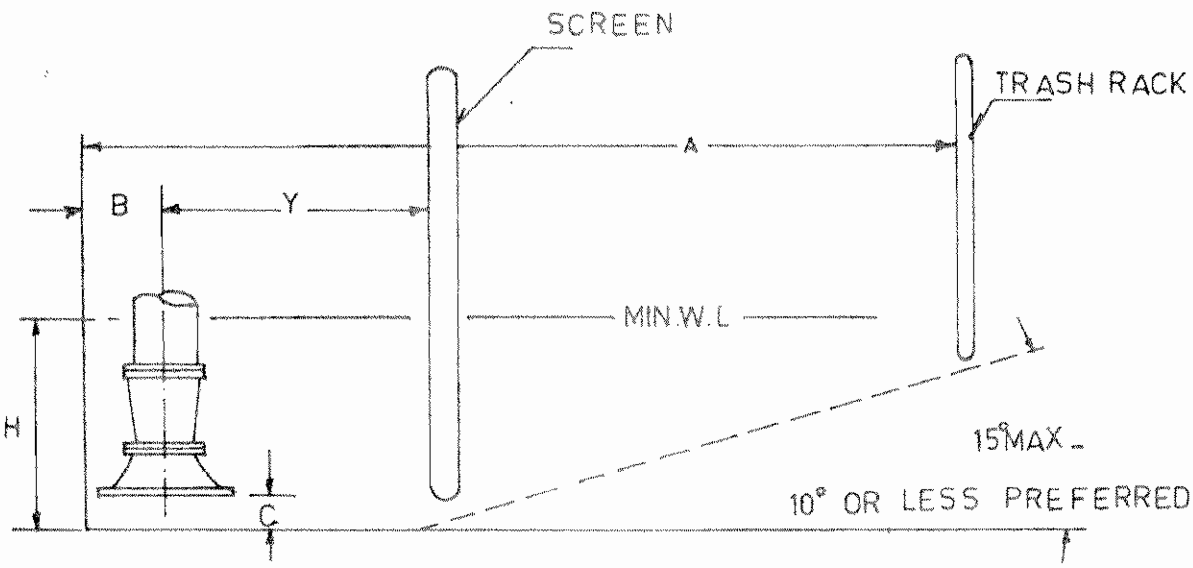


FIG. 11.12 SUMP DIMENSIONS ELEVATION VIEW

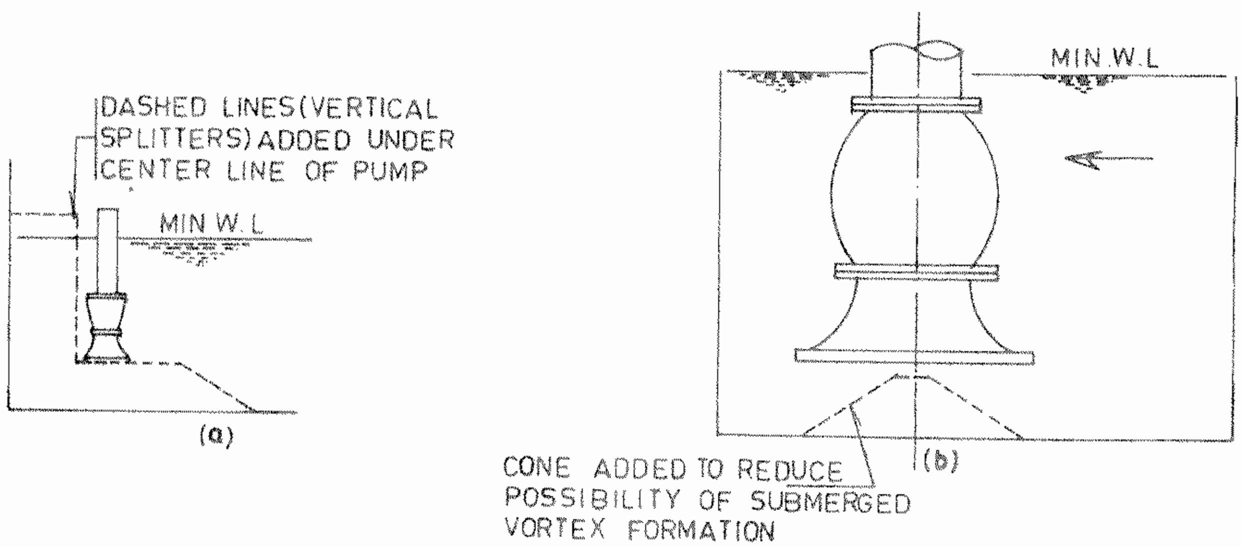


FIG. 11.13 VERTICAL SPLITTERS IN THE SUMP

11.3 PIPING LAYOUT

11.3.1 SUCTION PIPING

(a) The suction piping should be as short and straight as possible,

(b) Any bends or elbows should be of long radius,

(c) As a general rule the size of the suction pipe should be one or two sizes larger than the nominal suction size of the pump. Alternatively the suction pipe should be of such size that the velocity shall be about 2 m/s. Where bell-mouth is used, the inlet of the bell-mouth should be of such size that the velocity at the bell-mouth shall be about 1.5 m/s,

(d) Where suction-lift is encountered, no point on the suction pipe should be higher than the highest point on the suction part of the pump,

(e) When a reducer is used, it should be of the eccentric type. When on suction-lift, the taper side of the reducer should be below the centre line of the pump,

(f) The suction strainer should have net open area, minimum equal to three times the area of the suction pipe.

11.3.2 DISCHARGE PIPING

(a) The size of the discharge piping may be selected one size higher than the nominal delivery size of the pump. Alternatively, the delivery pipe should be of such size that the velocity shall be about 2.5 m/s,

(b) Discharge piping connection to a common manifold or header should be connected by a radial tee or by 30^0 or 45^0 bend,

(c) A dismantling joint must be provided between the pump and the valves. The design of the dismantling joint should be such that no pull or moment is transmitted to the pump.

11.3.3 VALVES

11.3.3.1 Suction Valves

(a) When suction lift is encountered, a foot valve is provided to facilitate priming. The pump can be primed also by a vacuum pump, if the pump is of large size, usually with suction-pipe larger than NS 300 mm.

The foot valves are normally available with strainers. The strainer of the foot valve should provide net area of its openings, to be minimum equal to three times the area of the suction pipe.

(b) When there is positive suction head, a sluice or a butterfly valve is provided on the pump suction, for isolation. The sluice valves should be installed with their axis horizontal to avoid formation of air-packets in the dome of the sluice valve.

11.3.3.2 Delivery Valves

Near to the pump, a non-return (reflux) valve and a delivery valve (sluice or butterfly valve) should be provided. The non-return valve should be between the pump and the delivery valve. The size of the valve should match the size of the piping.

11.3.3.3 Air Valves

Whenever there are distinct high points in the gradient of the pipeline, an air valve should be installed to permit expulsion of air from the pipeline. If the air is not expelled, it is likely to be compressed by the moving column of water. The compressed air develops high pressures, which can even cause the bursting of the pipeline.

Air valves also permit air to enter into the pipeline, when the pipeline is being emptied during shut down. If air would not enter during emptying, the pipeline will have vacuum inside and the atmospheric pressure externally. The pipeline would hence get subjected to undue stresses.

Details on provision and sizing of valves are given in 6.16.3.

11.3.4 Supports

All valves (including the foot valve, where necessary) and piping should be supported independent of each other and independent of the pump foundation.

11.3.5 Surge Protection Devices

When starting or stopping a pump (or by operating the regulating valves rapidly) certain pressure fluctuations are caused, which travel up and down in the pipeline during the transient conditions. This can cause low pressure zones, particularly at apex points on the pumping main and subsequently cause very high pressures causing hammering noise. If such pressure surges exceed the pressure permissible in the pipeline, the pipeline may even burst. To prevent against such occurrences, the recommended practices are detailed in 6.17.

11.4 SPACE REQUIREMENT AND LAYOUT PLANNING OF PUMPING SYSTEM

(a) Sufficient space should be available in the pump house to locate the pump, motor, valves, pipings, control panels and cable trays in a rational manner with easy access and with sufficient space around each equipment for the maintenance and repairs.

The minimum space between two adjoining pumps or motors should be 0.6 m for small and medium units and 1 m for large units.

(b) Space for the control panels should be planned as per the Indian Electricity (I.E.) Rules. As per these:

(i) a clear space of not less than 915 mm in width shall be provided in front of the switch board,

In case of large panels, a draw out space for the circuit breakers may exceed 915 mm. In such cases the recommendations of the manufacturers should be followed,

(ii) If there are any attachments or bare connections at the back of the switch board, the space, if any behind the switch-board shall be either less than 230 mm or more than 750 mm in width measured from the farthest part of any attachment or conductor,

(iii) If the switch board exceeds 760 mm in width, there shall be a passage-way from either end of the switch-board clear to a height of 1830 mm,

(c) A service bay should be provided in the station with such space that the largest equipment can be accommodated there for overhauling and repairs.

(d) A ramp or a loading and unloading bay should be provided. In large installations the floors should be so planned that all pipings and valves can be laid on the lower floor and the upper floor should permit free movement.

(e) Head room and material handling tackle.

(i) In the case of vertical pumps with hollow shaft motors, the clearance should be adequate to lift the motor clear off the face of the coupling and also carry the motor to the service bay without interference with any other apparatus. The clearance should also be adequate to dismantle and lift the largest column assembly.

(ii) In the case of horizontal pumps (or vertical pumps with solid shaft motors) the head room should permit transport of the motor above the other apparatus with adequate clearance.

(iii) The mounting level of the lifting tackle should be decided considering the above needs and the need of the head room for the maintenance and repair of the lifting tackle itself.

(iv) The traverse of the lifting tackle should cover all bays and all apparatus.

(v) The rated capacity of the lifting tackle should be adequate for the maximum weight to be handled at any time.

11.5 INSTALLATION OF PUMPS

The procedure of installation depends upon whether the pump is to be mounted horizontal or vertical. Most pumps to be mounted horizontal are supplied by the manufacturers as a wholesome, fully assembled unit. However, pumps to be mounted vertically are supplied as sub assemblies. For the installation of these pumps the proper sequence of assembly has to be clearly understood from the manufacturer's drawings.

The installation of a pump should proceed through, five stages in the following order:

(i) Preparing the foundation and locating the foundation bolts,

(ii) Locating the pump on the foundation bolts, however resting on leveling wedges, which permit not only easy leveling but also space for filling in the grout later on,

(iii) Leveling,

(iv) Grouting,

(v) Alignment

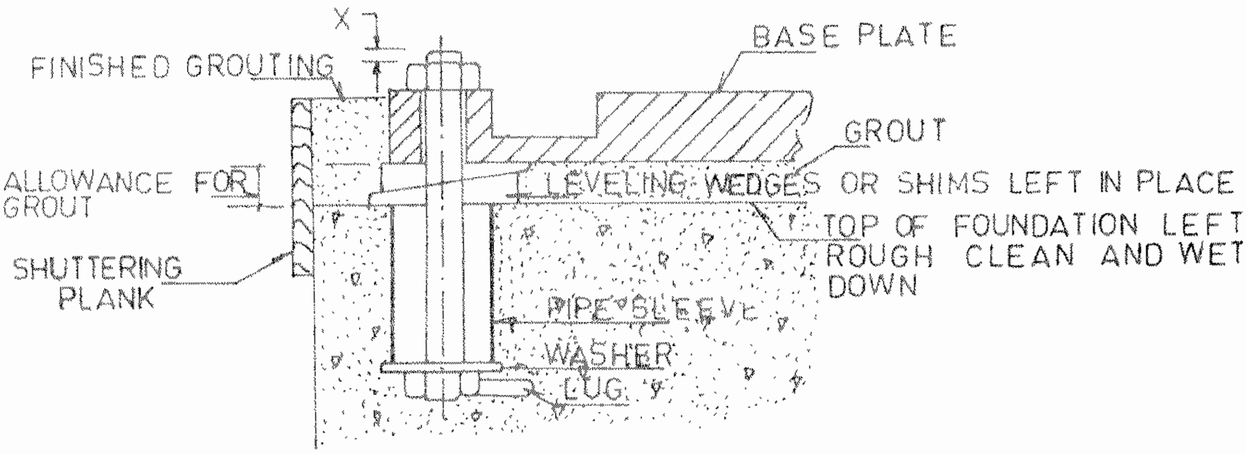


FIGURE 11.14 : TYPICAL FOUNDATION DESIGN

(a) The foundation should be sufficiently substantial to absorb vibrations and to form a permanent, rigid support for the base-plate. A typical foundation is illustrated in Fig. 11.14.

(b) The capacity of the soil or of the supporting structure should be adequate to withstand the entire load of the foundation and the dynamic load of the machinery. As mentioned in clauses 6.2.2 and 6.2.3 of IS: 2974 (Part iv) - 1979 the total load of the pump and the foundation should include the following:

(i) constructional loads

(ii) three times the weight of the pump

(iii) two times the total weight of the motor

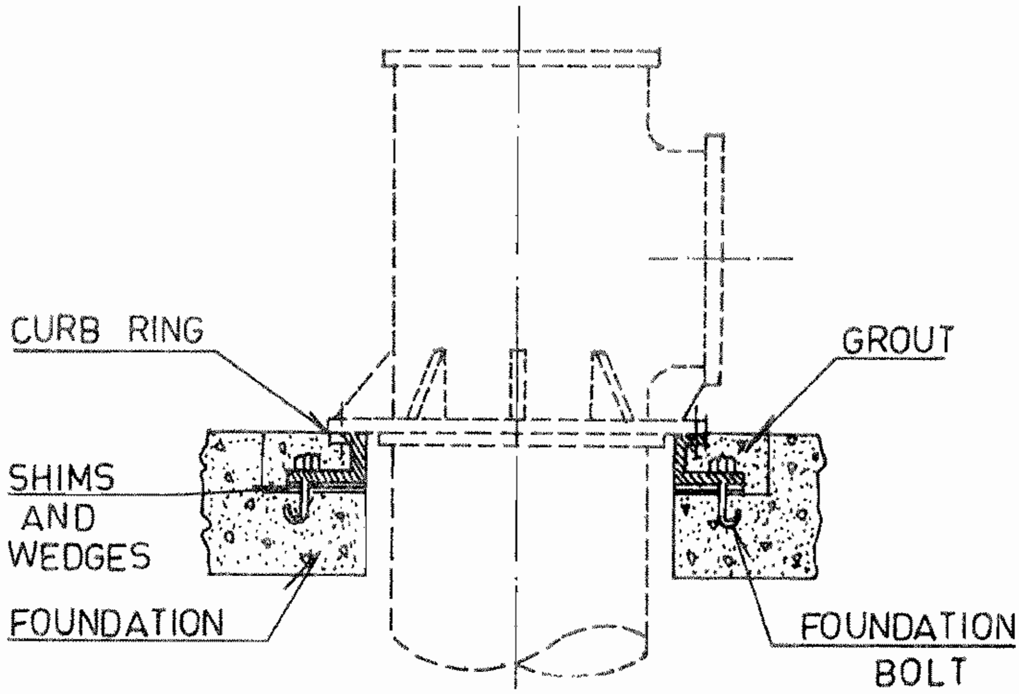
(iv) weight of water in the column pipe

(v) half of the weight of the unsupported pipe connected to the pump flanges.

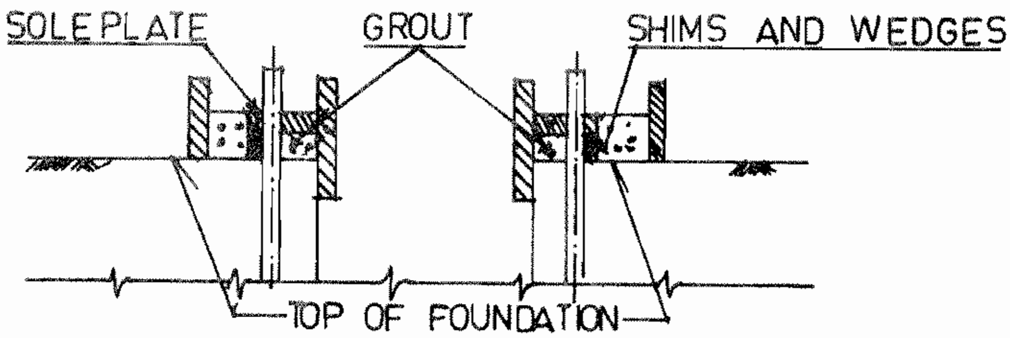
(c) If the pumps are mounted on steel structures, the location of the pump should be nearest as possible to the main members (i.e. beams or walls). The sections of structurals should have allowance for corrosion also.

(d) A curb ring or sole plate with machined top should be used as a bearing surface for the support flange of a vertical wet pit pump. The mounting face should be machined because the curb ring or sole plate is used to align the pump. Fig 11.15 shows typical arrangement with curb ring and with sole plate,

(e) Pumps kept in storage for a long time should be thoroughly cleaned before installation,



(a) ROUND TYPE CURBING FOR ABOVE GROUND DISCHARGE VERTICAL PUMP



(b) GROUTING FORM FOR VERTICAL PUMP SOLEPLATE

FIG. 11.15 FOUNDATION FOR VERTICAL PUMPS

(f) Submersible pumps with wet type motors should be fitted with water and the opening should be properly plugged after filling the water.

(g) Alignment of the pump sets should be checked even if they are received aligned by the manufacturers. The alignment should be proper both for parallelism (by filler gauge) and for coaxiality (by-straight edge or by dial gauge).

During all alignment-checks both the shafts should be pressed hard, over to one side while taking the readings.

Alignment should also be checked after fastening the piping and thereafter, periodically during operation.

11.6 COMMISSIONING

It should be ensured that the direction of the motor agrees with the arrow on the pump.

A specimen test should be conducted to derive the system-head curve and to understand the actual operating point/range of the pump and the variation, if any, from the original estimated duties. In the case of variations some analysis may be done to explore any feasible modifications of the system to bring it nearer to the original estimates or to generally improve the system so that it can work better and work trouble free for long.

11.7 OPERATION OF THE PUMPS

Summarized below are a few points to be observed while operating the pumps.

(a) Dry running of the pumps should be avoided. Centrifugal pumps have to be primed before starting. Helical rotor pumps, although they are self priming, being of the positive displacement type, need the rubber stator to be wetted before starting.

(b) Pumps should be operated only within the recommended range on the H-Q characteristics of the pump.

Operation near to the shut off should be avoided, as in the operation near the shut off, there happens substantial recirculation within the pump, which causes over heating.

(c) Whether the delivery valve should be open or closed at the time of starting is to be decided by studying the power characteristics of the pump.

As seen in Fig. 11.1 pumps of low and medium specific speeds draw more power as the flow increases. So to minimise the load on the motor while starting, such pumps are started with the delivery valve closed. Conversely pumps of high specific speed draw more power at shut off. Such pumps should hence be started with the delivery valve open. While stopping, the position of the delivery valve should be as at the time of starting.

(d) The delivery valve should be operated gradually to avoid surges,

(e) When pumps are to operate in parallel, the pumps should be started and stopped with a time lag between two pumps. The time lag should be adequate to let the pressure gauge stabilize,

(f) When the pumps are to operate in series, they should be started and stopped sequentially, but with the minimum time lag as possible. Any pump, next in sequence should be started immediately after the delivery valve of the previous pump is even partly opened.

Due care should be taken to keep the air vent of the pump next in sequence, open before starting that pump.

(g) The stuffing box should let a drip of leakage to ensure that no air is passing into the pump and that the packing is getting adequate water for cooling and lubrication. When the stuffing box is grease sealed, adequate refil of the grease should be maintained,

(h) The running of the duty pumps and of the standbys should be so scheduled that all pumps are in ready-to-run condition.

11.8 MAINTENANCE OF PUMPS

11.8.1 PERIODIC INSPECTION AND TEST

The maintenance schedule should enlist items to be attended to at different periods, such as daily, semi- annually, annually, etc.

11.8.2 DAILY OBSERVATIONS

A log-book should be maintained to record the observations, which should cover the following items.

(i) timings when the pump was run during the previous 24 hours,

(ii) at the time of observation, whether the leakage through the stuffing box is alright,

(iii) bearing temperature/s,

(iv) whether any undue noise or vibration,

(v) readings of pressure, voltage and current.

11.8.3 SEMI ANNUAL INSPECTION

(i) free movement of the gland of the stuffing box,

(ii) cleaning and oiling of the gland bolts,

(iii) inspection of the packing and repacking, if necessary,

(iv) alignment of the pump and the drive,

(v) cleaning of oil lubricated bearings and replenishing fresh oil. If bearings are grease lubricated, the condition of the grease should be checked and replaced/replenished to correct quantity. An antifriction bearing should have its housing so packed with grease that the void spaces in the bearings and the housing be 1/3 to 1/2 filled with greases. A fully packed housing will cause the bearing to overheat and will result in reduced life of the bearing.

11.8.4 ANNUAL INSPECTION

(i) cleaning and examination of all bearings for flaws developed, if any.

(ii) examination of shaft sleeves for wear or scour.

(iii) checking clearances.

Clearances at the wearing rings should be within the limits recommended by the manufacturer. Excessive clearances cause a drop in the efficiency of the pump. If the wear is only one side, it is indicative of misalignment. Not only that the misalignment should be set right, but also the causes for the disturbance of the alignment should be investigated. When the clearances have to be redeemed to the values recommended by the manufacturers, some general guidelines detailed in Table 11.6 would come handy.

If the clearance on wear is seen to be 0.2 or 0.25 mm more than the original clearance, the wearing ring should be renewed or replaced to get the original clearance.

In using the tolerance given in Table 11.6, they are to be used unilaterally. For example, while machining the i.d. of the wearing ring of basic size, say 175 mm the limits for machining would be 175.00 minimum and 175.04 maximum. For the corresponding O.D. at the hub of the impeller, the basic size will be with a clearance of 0.35, hence 174.65 mm and the machining limits will be 174.65 maximum and 174.61 minimum.

TABLE 11.6

WEARING RING I.D. DIAMETER CLEARANCE AND MACHINING TOLERANCE

Inside dia. of wearing ring mm	Diametral clearance mm	Machining Tolerance mm
upto100	0.3	0.04
100-150	0.35	0.04
150-200	0.4	0.06
200-300	0.45	0.06
300-500	0.55	0.06
500-750	0.58	0.06
750-1200	0.69	0.08
1200-2000	0.79	0.1

(iv) Impeller hubs and vane tips should be checked for any pitting or erosion.

(v) End play of the bearings should be checked.

(vi) All instruments and flow meters should be recalibrated.

(vii) Pump should be tested to determine whether proper performance is being obtained. In the case of vertical turbine pumps, the inspection can be bi-annual. Annual inspection is not advisable, because it involves disturbing the alignment and clearances.

11.8.5 FACILITIES FOR MAINTENANCE AND REPAIRS

11.8.5.1 Consumables And Lubricants

Adequate stock of such items as gland packings, belts, lubricating oils, greases should be maintained.

11.8.5.2 Replacement Spares

To avoid downtime, a stock of fast moving spares should be maintained. A set of recommended spares for two years of trouble free operation should be ordered alongwith the pump.

11.8.5.3 Repair Work-Shop

The repair workshop should be equipped with:

- tools such as bearing, pullers, clamps, pipe wrenches, etc.
- general-purpose machinery such as welding set, grinder, blower, drilling machine, etc.

11.9 TROUBLE SHOOTING

The check charts detailed in Tables 11.7, 11.8 and 11.9 provide guidelines for diagnosing the causes of troubles likely to arise during the operation of centrifugal, rotary and reciprocating pumps, respectively. As remedial measures, the cause/s of the trouble will have to be corrected.

TABLE 11.7

CHECK CHART FOR CENTRIFUGAL PUMP TROUBLES

Symptoms	Possible cause of trouble (Each number is defined in the list below)
Pump does not deliver water:	1, 2, 3, 4, 6, 11, 14, 16, 17, 22, 23
Insufficient capacity delivered:	2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 14, 17, 20, 22, 23, 29, 30, 31.
Insufficient pressure developed.	5, 14, 16, 17, 20, 22, 29, 30, 31.
Pump loses prime after starting.	2, 3, 5, 6, 7, 8, 11, 12, 13
Pump requires excessive power	15, 16, 17, 18, 19, 20, 23, 24, 26, 27, 29, 33, 34, 37.
Stuffing box leaks excessively:	13, 24, 26, 32, 33, 34, 35, 36, 38, 39, 40.
Packing has short life.	12, 13, 24, 26, 28, 32, 33, 34, 35, 36, 37, 38, 39, 40.

Symptoms	Possible cause of trouble (Each number is defined in the list below)
Pump vibrates or is noisy:	2, 3, 4, 9, 10, 11, 21, 23, 24, 25, 26, 27, 28, 30, 35, 36, 41, 42, 43, 44, 45, 46, 47.
Bearings have short life:	24, 26, 27, 28, 35, 36, 41, 42, 43, 44, 45, 46, 47
Pump overheats and seizes.	1, 4 , 21, 22, 24, 27, 28, 35, 36, 41

SUCTION TROUBLES

1. Pump not primed.
2. Pump or suction pipe not completely filled with liquid.
3. Suction lift too high.
4. Insufficient margin between suction pressure and vapour pressure.
5. Excessive amount of air or gas in liquid.
6. Air pocket in suction line.
7. Air leaks into suction line.
8. Air leaks into pump through stuffing boxes
9. Foot valve too small
10. Foot valve partially clogged.
11. Inlet of suction pipe insufficently submerged.
12. Water-seal pipe plugged.
13. Seal cage improperly located in stuffing box, preventing sealing fluid from entering space to form the seal.

SYSTEM TROUBLES

14. Speed too low.
15. Speed too high.
16. Wrong direction of rotation.
17. Total head of system higher than design head of pump.
18. Total head of system lower than pump design head.
19. Specific gravity of liquid different from design.
20. Viscosity of liquid different from that for which designed.
21. Operation at very low capacity.
22. Parallel operation of pumps unsuitable for such operation.

MECHANICAL TROUBLES

23. Foreign matter in impeller.

24. Misalignment.
25. Foundations not rigid.
26. Shaft bent.
27. Rotating part rubbing on stationary part.
28. Bearings worn.
29. Wearing rings worn
30. Impeller damaged
31. Casing gasket defective, permitting internal leakage.
32. Shaft or shaft sleeves worn or scored at the packing.
33. Packing improperly installed.
34. Incorrect type of packing for operating conditions.
35. Shaft running off center because worn bearings or misalignment.
36. Rotor out of balance, causing vibration.
37. Gland too tight, resulting in no flow of liquid to lubricate packing.
38. Failure to provide cooling liquid to water cooled stuffing boxes.
39. Excessive clearance at bottom of stuffing box between shaft and casing, causing packing to be forced into pump interior.
40. Dirt or grit in sealing liquid leading to scoring of shaft or shaft sleeve.
41. Excessive thrust caused by a mechanical failure inside the pump or by the failure of the hydraulic balancing device, if any
42. Excessive grease or oil in antifriction bearing housing or lack of cooling, causing excessive bearing temperature.
43. Lack of lubrication.
44. Improper installation of anti-friction bearings (damage during assembly, incorrect assembly of stacked bearings, use of unmatched bearings as a pair, etc.)
45. Dirt in bearings.
46. Rusting of bearings from water in housing.
47. Excessive cooling of water-cooled bearing, resulting in condensation of moisture from the atmosphere in the bearing housing.

TABLE 11.8

CHECK CHART FOR ROTARY PUMP TROUBLES

Symptoms	Possible cause of trouble (Each number is defined in the list below)
Pump fails to discharge	1, 2, 3, 4, 5, 6, 8, 9, 16.
Pump is noisy	6, 10, 11, 17, 18, 19
Pump wears rapidly	11, 12, 13, 20, 24.
Pump not up to capacity.	3, 5, 6, 7, 9, 16, 21, 22
Pump starts, then loses suction.	1, 2, 6, 7, 10
Pump takes excessive Power	14, 15, 17, 20, 23

SUCTION TROUBLES

1. Not properly primed
2. Suction pipe not submerged.
3. Strainer clogged
4. Leaking foot valve
5. Suction lift too high
6. Air leaks in suction.
7. Suction pipe too small

SYSTEM PROBLEMS

8. Wrong direction of rotation.
9. Low speed.
10. Insufficient liquid supply.
11. Excessive pressure.
12. Grit or dirt in liquid.
13. Pump runs dry.
14. Viscosity higher than specified.
15. Obstruction in discharge line.

MECHANIC4L TROUBLES

16. Pump worn.

17. Bent drive shaft
18. Coupling out of balance or alignment.
19. Relief valve chatter.
20. Pipe strain on pump casing.
21. Air leak at packing.
22. Relief valve improperly seated.
23. Packing too tight.
24. Corrosion.

TABLE 11.9

CHECK CHART FOR RECIPROCATING PUMP TROUBLES

Symptoms	Possible cause of trouble (Each number is defined in the list below)
Liquid end noise.	1, 2, 7, 8, 9, 10, 14, 15, 16
Power end noise.	17, 18, 19, 20
Overheated power end :	10, 19, 21, 22, 23, 24.
Water in crankcase.	25
Oil leak from crankcase	26,27
Rapid packing or plunger wear.	11, 12, 28, 29.
Pitted valves or seats	3,11,30
Valves hanging up	31,32
Leak at cylinder-valve hole plugs.	10,13,33,34
Loss of prime.	1,4,5,6

SUCTION TROUBLES

1. Insufficient suction pressure
2. Partial loss of prime.
3. Cavitation.
4. Lift too high
5. Leaking suction at foot valve
6. Acceleration head requirement too high.

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SYSTEM PROBLEMS

7. System shocks
8. Poorly supported piping, abrupt turns in piping, pipe size too small, piping misaligned.
9. Air in liquid.
10. Overpressure or overspeed.
11. Dirty liquid.
12. Dirty environment.
13. Water hammer

MECHANICAL TROUBLES

14. Broken or badly worn valves.
15. Packing worn.
16. Obstruction under valve.
17. Loose main bearings.
18. Worn bearings.
19. Low oil level.
20. Plunger loose.
21. Tight main bearings
22. Inadequate ventilation.
23. Belts too tight.
24. Driver misaligned.
25 Condensation
26. Worn seals
27. Oil level too high.
28. Pump not set level and rigid.
29. Loose packing.
30. Corrosion
31. Valve binding.
32. Broken valve spring
33. Loose cylinder plug
34. Damaged 0-ring seal

11.10 SELECTION OF ELECTRIC MOTORS

11.10.1 General

In water supply systems, mainly three types of motors are used.

- Induction (A.C.) motors.
- Synchronous (A.C.) motors.
- D.C. motors.

Amongst these, induction motors are the most common.

Synchronous motors merit consideration when large HP, low speed motors are required. D.C. motors are used occasionally for pumps where only direct current is available as in ships, railways, etc.

11.10.2 Selection Criteria

Type of motor has to be selected considering various criteria such as the constructional features desired, environment conditions, type of duty, etc.

11.10.2.1 Constructional Features Of Induction Motors

Squirrel cage motors are most commonly used. Normally the starting torque requirement of centrifugal pumps is quite low and squirrel cage motors are therefore suitable.

Slip ring or wound rotor motors are to be used where required starting torque is high as in positive displacement pumps or for centrifugal pumps handling sludge.

The slip ring motors are also used when the starting current has to be very low, such as 1 time the full load current, such regulatory limits being specified by the Power Supply Authorities.

11.10.2.2 Method Of Starting

Squirrel cage motors when started direct on line (with DOL starter) draw starting current about 6 times the full load (FL) current. If the starting current has to be within the regulatory limits specified by the Power Supply Authorities, the squirrel cage motors should be provided with the star delta starter or auto-transformer starter.

11.10.2.3 Voltage Ratings

Table 11.10 would give general guidance on the standard voltages and corresponding range of motor ratings.

For motors of ratings 225 KW and above, where high-tension (HT) voltages of 3.3 KV, 6.6 KV and 11 KV can be chosen, the choice should be made by working out relative economics of investment and running costs, taking into consideration costs of transformer, motor, switchgear, cables etc.

11.10.2.4 Type Of Enclosures (Table 11.11)

TABLE 11.10

SELECTION OF MOTORS BASED ON SUPPLY VOLTAGES

Supply	Voltage	Range of Motor rating in KW Min.	Max.
Single-phase A.C.	230 V	0.3	2.5
Three-phase A.C.	415 V	-	250
	3.3 KV	225	750
	6.6 KV	400	-
	11KV	600	-
D.C.	230V	-	150

N.B. When no minimum is given, very small motors are feasible. When no maximum is given very large motors are feasible.

TABLE 11.11

TYPES OF ENCLOSURES

Type	**Environment Code as per IS**	**Where used**
screen-protected drip proof (SPDP)	IP.23	Indoor, clean (dust-free) environment.
Totally enclosed fan cooled (TEFC)	IP.44	Indoor, dust-prone areas.
	IP.54	Normal Outdoor
	IP.55	Outdoor at places of Heavy rainfall

11.10.2.5 Class Of Duty

(i) All motors should be suitable for continuous duty i.e. Class SI as specified IS: 325-1978.

(ii) Additionally, it is recommended that motors should be suitable for minimum 3 equally spaced starts per hour.

(iii) The motor should also be suitable for atleast one hot restart.

11.10.2.6 Insulation

Class B insulation is generally satisfactory, since it permits temperature rise upto 80^0 C.

At cool places having ambients upto 30^0C, motors with Class E insulation can also be considered.

At hot places having ambients above 40^0C, motors with Class F insulation should be considered.

11.10.2.7 Selection Of Motor Rating

Motors are rated as per the output shaft horsepower (Brake kilowatts, BKW). The motor rating should be selected as to provide margins, over the required BKW, calculated for the pump.

11.11 STARTERS

11.11.1 TYPES

Starters are of different types, viz. Direct on-line (DOL), Star Delta, auto-transformer and stator rotor. Of these, the last one is used with slip ring motors. The other three are used with squirrel cage motors.

11.11.2 STARTERS FOR SQUIRREL CAGE MOTORS

Starters draw starting current, which is considered as a multiple of the full load current (FLC) of the motor. Different types of starters help control the starting current required. General guidelines are given in Table 11.12.

TABLE 11.12
GUIDELINES FOR STARTERS FOR SQUIRREL CAGE MOTORS

Type of Starter	Percentage of voltage reduction	Starting Current	Ratio of starting torque to locked rotor torques, %
DOL	Nil	6 x FLC	100
Star delta	58%	2 x FLC	33
Auto-transformer	Tap 50%	1.68 x FLC	25
	Tap 65%	2.7 x FLC	42
	Tap 80%	4 x FLC	64

Note : As per the torque speed characteristics of the motor, the torque of the motor at the chosen percentage of reduced voltage should be adequate to accelerate the pump to the full speed.

11.11.2.1 Selection Of The Tapping Of Auto Transformer Type Starter

The torque available from the motor is generally much higher than the starting torque required by the pump, as the starting torque required by the pump is also regulated by starting the pump with the delivery valve closed or open, depending upon the nature of the power versus Q characteristics of the pump.

The torque available from the motor being more than the starting torque required by the pump draws an unnecessary excessive current. This can be controlled as the torque available from the motor varies as the square of the applied voltage. For reducing the excessive torque available from the motor, the voltage to be applied to the motor can be reduced by selecting the appropriate percentage tapping on the auto transformer starter. The value of the percentage for the tapping position can be decided by the following formula.

$$Tapping\% = 100 \times \sqrt{\frac{Torque\ \ for\ \ pump}{Torque\ \ for\ \ motor}}$$

where,

Torque for pump is the torque required by the pump at its rated speed and at its maximum power demand; and

Torque from motor is the torque available from the motor at its full-load capacity at its rated speed at rated voltage.

Based on the above calculation, the nearest higher available position of tapping should be selected.

11.12 PANELS

11.12.1 REGULATIONS

The regulations, as per I.E. Rules, in respect of space to be provided around the panel are detailed under 11.4.

11.12.2 VARIOUS FUNCTIONS

The various functions, which the panel has to serve and the corresponding provisions to be made in the panel are detailed below:

1. For receiving the supply - Circuit breaker or switch and fuse units.
2. For distribution - Bus bar, Switch fuse units, circuit breakers.
3. For controls - Starters: level-controls, if needed: Time- delay relays.
4. As protections - Under voltage relay, Over-current relay, Hot fault relay, Single Phasing Preventor.

5. For indications and readings - Phase indicating lamps, voltmeters, Ammeters, Frequency meter, power factor meter, temperature scanners, Indications for state of relays, indications for levels, indications of valve positions, if valves are power actuated.

The scope and extent of provisions to be made on the panel would depend upon the size and importance of the pumping station.

11.12.3 Improvement of Power Factor

For improvement of power factor, appropriate capacitors should be provided. Capacitors may be located in the control panel or separately.

Some useful guidelines regarding the selection, installation, operation and maintenance of the power capacitors are compiled in the following paragraphs.

11.12.3.1 Selection Of Capacitors

It is generally advisable that capacitors be installed across individual machines. However, in the case of intermittently running machines, it is advisable to select the capacitor of rating appropriate to the average active load for a group of such machines, installing the capacitor across the mains through a fuse switch. A rationalised combination of individual machine-mounting of capacitors and a mains installation of capacitors, for a group of machines running intermittently, can also be made in order to maintain a power factor yielding optimum economy.

To have a flexible arrangement for maintaining the power factor within set limits would require an automatic power factor correction panel, monitoring a bank of capacitors through a power factor sensing relay and appropriate contactors, the recommended capacitor rating for direct connection to induction motors is given in Table 11.13.

11.12.3.2 Installation Of Capacitors

While installing a capacitor, ensure following :

(a) A capacitor should be firmly fixed to a base.

(b) Cable lugs of appropriate size should be used.

(c) Two spanners should be used to fasten or loosen capacitor terminals. The lower nut should be held by one spanner and the upper nut should be held by the other to avoid damage to or breakage of terminal bushings and leakage of oil.

(d) To avoid damage to the bushings, a cable gland should always be used and it should be firmly fixed to the cable-entry hole.

(e) The capacitor should always be earthed appropriately at the earthing terminal to avoid accidental leakage of the charge.

(f) There should be a clearance of at least 75 mm on all sides for every capacitor unit to enable cooler running and maximum thermal stability. Ensure good ventilation and avoid proximity to any heat source.

TABLE 11.13

RECOMMENDED CAPACITOR RATING FOR DIRECT CONNECTION TO INDUCTION MOTORS

(To improve power factor to 0.95 or better)

Capacitor rating in KVAr when motor speed is							Capacitor rating in KVAr when motor speed is						
Motor H.P.	3000 r.p.m.	1500 r.p.m.	1000 r.p.m.	750 r.p.m.	600 r.p.m.	500 r.p.m.	Motor H.P.	3000 r.p.m.	1500 r.p.m.	1000 r.p.m.	750 r.p.m.	600 r.p.m.	500 r.p.m.
2.5	1	1	1.5	2	2.5	2.5	105	22	24	27	29	36	41
5	2	2	2.5	3.5	4	4	110	23	25	28	30	38	43
7.5	2.5	3	3.5	4.5	5	5.5	115	24	26	29	31	39	44
10	3	4	4.5	5.5	6	6.5	120	25	27	30	32	40	46
12.5	3.5	4.5	5	6.5	7.5	8	125	26	28	31	33	41	47
15	4	5	6	7.5	8.5	9	130	27	29	32	34	43	49
17.5	4.5	5.5	6.5	8	10	10.5	135	28	30	33	35	44	50
20	5	6	7	9	11	12	140	29	31	34	36	46	52
22.5	5.5	6.5	8	10	12	13	145	30	32	35	37	47	54
25	6	7	9	10.5	13	14.5	150	31	33	36	38	48	55
27.5	6.5	7.5	9.5	11.5	14	16	155	32	34	37	39	49	56
30	7	8	10	12	15	17	160	33	35	38	40	50	57
32.5	7.5	8.5	11	13	16	18	165	34	36	39	41	51	59

Capacitor rating in KVAr when motor speed is							Capacitor rating in KVAr when motor speed is						
Motor H.P.	3000 r.p.m.	1500 r.p.m.	1000 r.p.m.	750 r.p.m.	600 r.p.m.	500 r.p.m.	Motor H.P.	3000 r.p.m.	1500 r.p.m.	1000 r.p.m.	750 r.p.m.	600 r.p.m.	500 r.p.m.
35	8	9	11.5	13.5	17	19	170	35	37	40	42	53	60
37.5	8.5	9.5	12	14	18	20	175	36	38	41	43	54	61
40	9	10	13	15	19	21	180	37	39	42	44	55	62
42.5	9.5	11	14	16	20	22	185	38	40	43	45	56	63
45	10	11.5	14.5	16.5	21	23	190	38	40	43	45	58	65
47.5	10.5	12	15	17	22	24	195	39	41	44	46	59	66
50	11	12.5	16	18	23	25	200	40	42	45	47	60	67
55	12	13.5	17	19	24	26	205	41	43	46	48	61	68
60	13	14.5	18	20	26	28	210	42	44	47	49	61	69
65	14	15.5	19	21	27	29	215	42	44	47	49	62	70
70	15	16.5	20	22	28	31	220	43	45	48	50	63	71
75	16	17	21	23	29	32	225	44	46	49	51	64	72
80	17	19	22	24	30	34	230	45	47	50	52	65	73
85	18	20	23	25	31	35	235	46	48	51	53	65	74
90	19	21	24	26	33	37	240	46	48	51	53	66	75
95	20	22	25	27	34	38	245	47	49	52	54	67	75

Capacitor rating in KVAr when motor speed is							Capacitor rating in KVAr when motor speed is						
Motor H.P.	3000 r.p.m.	1500 r.p.m.	1000 r.p.m.	750 r.p.m.	600 r.p.m.	500 r.p.m.	Motor H.P.	3000 r.p.m.	1500 r.p.m.	1000 r.p.m.	750 r.p.m.	600 r.p.m.	500 r.p.m.
100	21	23	26	28	35	40	250	48	50	53	55	68	76

Note : The recommended capacitor rating given in the above table are only for the guidance purpose. (The capacitor rating should correspond approximately to the apparent power of the motor on no-load)

(g) While making a bank, the bus bar connecting the capacitors should never be mounted directly on the capacitor terminals. It should be indirectly connected through flexible leads so that the capacitor bushings do not get unduly stressed.

This may otherwise result in oil leakage and/or porcelain breakage.

(h) Ensure that the cables, fuses and switchgear are of adequate rating.

11.12.3.3 Operation And Maintenance Of Capacitors

(a) The supply voltage at the capacitor bus should always be near about the rated voltage and the supply voltage including the allowable fluctuations should not exceed 110% of the rated voltage of the capacitor.

(b) Frequent switching of the capacitor should be avoided.. There should always be an interval of about 60 seconds between any two switching operations.

(c) The discharge resistance efficiency should be assessed periodically by sensing, if shorting is required to discharge the capacitor even after one minute of switching off. If the discharge resistance fails to bring down the voltage to 50V in one minute, it needs to be replaced.

(d) Leakage or breakage should be mended immediately. Care should be taken that no appreciable quantity of impregnant has leaked out.

(e) Before physically handling the capacitor, short circuit the capacitor terminals one minute after disconnection from the supply to ensure total discharging of the capacitor.

11.13 CABLES

Table 11.14 gives guidance of the types of cables to be used for different voltages.

TABLE 11.14

TYPES OF CABLES FOR DIFFERENT VOLTAGES

SNo.	Range of Voltage	Type of cable to be used	IS Ref
1.	10-230 V or 30-415 V	PVC insulated, PVC sheathed	IS 1554
2.	Upto 6.6 KV	PVC insulated, PVC sheathed	IS 1554
		Paper insulated, lead sheathed	IS 692
		XLPF, Cross linked, Polyethylene Insulated, PVC Sheathed.	IS 7098
3.	11 KV	Paper insulated, lead sheathed	IS 692

The size of the cable should be so selected that the total drop in voltage, when calculated as the product of current and the resistance of the cable shall not exceed 3%. Values of the resistance of the cable are available from the cable manufacturers.

In selecting the size of the cable the following points should be considered :

(i) The current carrying capacity should be appropriate for the lowest voltage, the lowest power factor and the worst condition of installation i.e. duct condition.

(ii) The cable should also be suitable for carrying the short circuit current for the duration of the fault.

(iii) The duration of the fault should preferably be restricted by 0.1 second by proper relay setting.

(iv) Appropriate rating factors should be applied when cables are laid in group (parallel) and/ or laid below ground.

(v) For laying cables, suitable trenches or racks should be provided.

11.14 TRANSFORMER SUBSTATION

11.14.1 Essential Features

Normally outdoor substations are provided. However on considerations of public safety and for protection from exposure to environmental pollution, the substations may be indoors.

(i) Lightning arresters

(ii) Gang operated disconnectors (GOD) are provided in outdoor substation. In indoor substation, circuit breakers are provided. In case of outdoor substations of capacities 1000 KVA and above, circuit breakers should be provided in addition to GOD.

(iii) Drop out fuses for small out door substations.

(iv) overhead bus bars and insulators.

(v) Transformer.

(vi) Current transformer and potential transformer for power measurement.

(vii) Current transformers and potential transformers for protection in substations of capacity above 1000 KVA.

(viii) Fencing.

(ix) Earthing.

Earthing should be very comprehensive, covering every item in the substation and in accordance with IS: 3043.

11.14.2 Duplicate transformer may be provided, where installation so demands

11.15 MAINTENANCE AND REPAIRS OF ELECTRICAL EQUIPMENT

11.15.1 Consumables

Adequate stock of lubricating oil and transformer oil should be maintained.

11.15.2 Replacement Spares

To avoid downtime, stock of fast moving spares and of spares likely to be damaged by short circuit should be maintained. A set of recommended spares for two years of trouble free operation should be ordered alongwith the equipments.

11.15.3 Tools and Test Equipments

Tools such as crimping tools, soldering, brazing and usual electrical tools should be available.

11.15.4 Preventive maintenance

As preventive maintenance, it is advisable to follow a schedule for the maintenance of the equipments. The schedule covers recommendations for checks and remedial actions, to be observed at different periodicities such as daily, monthly, quarterly, semi annually, annually and bi-annually.

11.15.4.1 Daily

(i) For Motors

(a) Check bearing temperatures.

(b) Check for any undue noise or vibration.

(ii) For panel, circuit-breaker, starter;

(a) Check the phase-indicating lamps

(b) Note readings of voltage, current, frequency etc.

(c) Note energy-meter readings.

(iii) For transformer substation

(a) Note voltage and current readings.

11.15.4.2 Monthly

(i) For motor:- nothing special other than the daily checks.

(ii) For panel, circuit-breaker, starter.

(a) Examine contacts of relay and circuit breaker. Clean, if necessary.

(b) Check setting of over-current relay, no volt coil and tripping mechanism and oil in the dashpot relay.

(iii) For transformer substation

(a) Check the level of the transformer oil.

(b) Check that the operation of the GOD is okay.

(c) Check contacts of GOD and of over- current (OC) relay.

(d) Check temperatures of the oil and windings.

(e) Clean radiators to be free of dust and scales.

(f) Pour 3 to 4 buckets of water in each earth-pit.

11.15.4.3 Quarterly

(i) For motor:

(a) Blow away dust and clean any splashing of oil or grease.

(b) Check wear of slip ring and bushes, smoothen contact-faces or replace, if necessary. Check spring-tension. Check bush-setting for proper contact on the slip-ring.

(c) Check cable connections and terminals and insulation of the cable near the lugs, clean all contacts, if insulation is damaged by overheating investigate and rectify. All contacts should be fully tight.

(ii) For panel, circuit-breaker, starter, etc.

(a) Check fixed and moving contacts of the circuit breakers/ switches. Check and smoothen contacts with fine-glass- paper or file.

(b) Check condition and quantity of oil/liquid in circuit-breaker, auto- transformer starter and rotor-controller.

(iii) For transformer substation;

(a) Check condition of the H.T. bushing.

Check the condition of the dehydrating breather and replace the silica-gel charge, if necessary. Reactivate old charge for reuse.

11.15.4.4 Semi-Annual

(i) For motor

(a) Check condition of oil or grease and replace if necessary. While greasing, avoid excessive greasing.

(b) Test insulation by megger.

(ii) For panel, etc.

Check for corrosion and take remedial measures. Check by megger the insulation-resistance of switches, busbar, starter- terminals, auto-transformer, etc. for phase- tp-earth and phase-tp-phase, resistance.

(iii) For transformer substation.

(a) Check die-electric strength and acid- test of transformer oil and filter, if necessary.

(b) Test insulation by megger.

(c) Check continuity for proper earth connections.

11.15.4.5 Annual

(i) For motors;

(a) Examine bearings for flaws, clean and replace if necessary.

(b) Check end-play of bearings and reset by lock nuts, wherever provided.

(ii) For panel, etc.;

(a) All indicating meters should be calibrated.

(iii) For transformer substation

(a) Check resistance of earth pit/earth electrode.

11.15.4.6 Bi-Annual

(i) for motor: Same as annual

(ii) for panel, etc. same as annual

(iii) for transformer substation

(a) Complete examination including internal connections, core and windings.

11.16 TROUBLE SHOOTING FOR ELECTRICAL EQUIPMENT

Trouble-shooting comprises detecting the trouble, diagnosing the cause and taking remedial action. Detection of the trouble is prompted by noticing the symptoms. The trouble-shooting details are hence outlined hereunder for various symptoms.

11.16.1 MOTOR GETS OVERHEATED

(i) Check whether voltage is too high or too low. Change tapping of transformer, if HT supply is availed. Otherwise approach power supply authorities for correction of the supply voltage.

(ii) Check whether air ventilation passage of motor is blocked. Clean the passage.

(iii) Check whether the motor bearings are properly lubricated or damaged. Replace the damaged bearings and provide proper lubrication.

(iv) Check whether the cable terminals at the motor are loose. Tighten the terminals.

11.16.2 MOTOR GETS OVER LOADED: (DRAWING MORE THAN THE RATED CURRENT AT THE RATED VOLTAGE)

(i) Check any excessive rubbing in the pump or any clogging of the impeller passages.

(ii) Check whether characteristics of pump (i.e. the related driven equipment) are of the overloading type.

(iii) Check for any vortices in the sump.

(iv) Check that there is no short-circuiting or single-phasing.

(v) Check whether any foreign matter has entered the air-gap, causing obstruction to the smooth running of the motor.

11.16.3 Starter/Breaker Trips

(i) Check whether the relay is set properly. Correct, the setting, if necessary.

(ii) Tripping can also occur, if motor is drawing more than the rated current, for which details are mentioned above.

(iii) Oil in dashpot may be either inadequate or of low viscosity.

(iv) Check that there are no loose connections.

(v) Check whether the timer setting of star delta or auto transformer starter is proper.

11.16.4 Vibration in Motor

(i) Check for structural rigidity of supporting frame and foundation.

(ii) Check alignment of pump and motor.

(iii) Check that the nuts on foundation bolts are tight.

(iv) Check if rotor has an imbalance.

(v) Check the resonance from supporting structure or foundation or from critical speed of rotor or from vibration of adjoining equipment.

11.16.5 Cables Get Over-Heated

(i) Check whether the cable is undersized. Change the cable or provide another cable in parallel.

(ii) Check for loose termination or joint. Fasten the termination and make proper joint.

(iii) Check whether only a few strands of the cable are inserted in the lug. Insert all strand using a new lug, if necessary.

CHAPTER 12
INSTRUMENTATION AND CONTROLS IN WATER TREATMENT PLANT

12.1 INTRODUCTION

Instrumentation and control plays an important role in efficient and effective operation of any water treatment plant. In order to monitor the quality and quantity of water produced and to have trouble free operation of water treatment plant, it is desirable to provide proper instrumentation and control system in the plant. The impact of sudden changes in raw water quality, peak demands and seasonal variations require quick responses and proper action. This is possible only if the plant is provided proper instrumentation and control systems.

This chapter covers the general applications of instrumentation and control system in water treatment plant. Water treatment plant equipments are generally of a rugged nature and not prone to much mechanical defects. It may, therefore, not be desirable to go in for complex automatic control systems.

12.2 PURPOSE AND OBJECTIVE

The purpose and objectives of Instrumentation & Control systems in a water treatment plant are:

(a) To produce water at a lower cost in lesser time.

(b) To control certain key functions in order to maintain balance in plant processes.

(c) To obtain plant operating data such as (i) characteristics of raw & treated water, (ii) flow and quantity measurements including the record of consumables.

(d) To guide the operator by providing all related data for efficient functioning of various units of water treatment plants.

12.2.1 INSTRUMENTS & CONTROL SYSTEMS

The instruments and control systems when properly applied and used will provide:

(i) Precision of operation and instantaneous response to changes in important process variables.

(ii) Indication and recording of key operating data.

(iii) Means of better utilization of manpower and treatment chemicals and reduction in down time due to disruption in normal operating procedure.

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The instruments and control system have been classified in two categories : Essential and optional. Systems which are considered essential from the point of view of safety of chemical dosing, control and operational ease, constitute the essential systems. The essential system should preferably be incorporated in all the plants. Optional items can be considered where the owner intends to use them for data collection and information and are used where skilled manpower is available.

12.3 SYSTEMS AVAILABLE

The most commonly used instrument and control systems in water treatment plants are:

(i) Mechanical;

(ii) Pneumatic;

(iii) Electric;

(iv) Electropneumatic; and

(v) Hydropneumatic.

12.3.1 MECHANICAL

These instruments are locally mounted or connected for the measurement of parameters at the specific point of measurement. These instruments are operated on mechanical principles by use of floats, pulleys and gears. These include pressure gauges, level indicators and flow indicating devices.

12.3.2 PNEUMATIC

Pneumatically operated instrument and control system uses clean, dry and filtered air for both transmission and power media for activating the control elements. An example is the pneumatically operated valves for filter beds.

12.3.3 ELECTRIC

The electrically operated system employs electrical signal for transmission as well as control signal to the control element. An example for such a system is the motorised valves for filter beds or for sludge withdrawal from clariflocculator.

12.3.4 ELECTROPNEUMATIC

This system employs an electric transducer (with integral transmitter), electric receiver, electric set point and electric controller. The controller sends an electric signal to a positioner with a pneumatic four valve to activate a pneumatic operator to final control element. In this system the transmission medium is an electric signal and the power medium is air.

12.3.5 HYDROPNEUMATIC

This system is basically identical to pneumatic system except that power medium is oil or water. The transmission medium is air.

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12.3.6 METHOD OF CONTROL

The common method of control of water treatment plant can be manual, semiautomatic, or automatic.

The system to be adopted will depend upon location, capacity, skilled man power availability, spare parts availability etc. The adoption of any particular system also depends upon the determination of the extent of information required by the operating personnel for proper operation of the plant.

12.3.6.1 Manual

This control involves the use of instruments to read plant variables manually. Adjustment in the processes when required are made manually by turning of a valve, pushing a button or such simple operation.

12.3.6.2 Semi Automatic

This control involves the use of instruments to automatically control a function or series of functions after control points are set manually or button is pushed to initiate an automatic sequence operation programme.

12.3.6.3 Automatic

This control involves use of instruments to automatically control and maintain in balance the process of functions. A close loop system is used with feed back of operation. When there is change in any process variable, the change is sensed and transmitted to a control instrument that adjusts a device to restore the system to balance.

12.4 DESIGN PRINCIPLES AND PRACTICES

There is no "Standard" design for an instrumentation or control system applied to a water treatment plant. However, there are certain basic considerations that govern the application of instrumentation and control to water treatment plant design concepts regardless of plant capacity, water quality or man-power factors. Examples of these considerations include flow measurements, rate controller, loss of head control, and level indication, etc.

The various requirements of control system, therefore, are grouped so that requirements can be selected based on above principles.

12.5 LEVEL MEASUREMENT

Level measuring units determine the amount of material in containers of all types either continuously or at intervals. There is no universal method available, due to wide range of processes within the container. Therefore, different measuring methods have been developed. With the exception of weight measurement, all measuring method determine the amount of material in the container by measuring the level within the container. The possible methods may be:

(i) Mechanical measuring methods: Here mechanical devices are employed such as flaps, membranes, float operated system, use of static pressures etc.

(ii) Physical measuring methods: Here certain physical characteristics are utilised such as electrical conductivity of material, use of optical ultrasonic beams etc.

12.5.1 ESSENTIAL INSTRUMENTS

The following level measuring instruments are considered essential:

(a) Chemical Tanks

Each chemical tank in the plant should be provided with float operated type local level indicator except in cases where the line tanks have M.S. cup type agitators rotating in the horizontal plane which lead to fluctuating solution levels. Float and wire rope should be of corrosion resistant material. The indicator should be either vertical arrow scale type or horizontal arrow scale type but the graduations should be of such a size that reading can be viewed clearly from a distance not less than 2.5 to 3.0 m.

(b) Overhead Tank

Generally all water treatment plants have an overhead tank which caters to the water requirements for chemical solution preparation and filter backwashing. The overhead tank is usually at a higher elevation. It is necessary to have a remote indication of the water level in overhead tank in the filter house or chemical house or near overhead tank filling pumps in case the overhead tank is filled by pumps in the treatment plant area. In case of a float operated level indicator, the float should be S.S. 316 or equivalent and coupled to a two wire transmitter for signal for remote indication.

(c) Tanks/Sumps

For safety of the pumps against dry running, each tank/sump where draw-off is by pumping, should be provided with magnetic type or electronic level switch which will be actuated by low level in the tanks. It is advisable to instal displacer type tank top mounted magnetic or electronic control switches. It should also be possible to adjust the actuation level of these switches in field. The auto-stopping of respective pumps should be controlled through these level switches. Level switch displacer and wire rope should be of SS-316 while switch assembly can be housed in Al-alloy enclosure, which should be weatherproof.

(d) Loss Of Head For Filters

For loss of head across filters, a float operated direct reading meter is used. The pressure of water beyond the filter outlet valve and the pressure of water above the sand bed are directly transmitted to float chambers where two different floats correspond to the two different levels. Separate chain, sprocket and counter weight arrangements on each float cause the indicator pointer over the engraved dial to rotate in one case and the dial itself in case of the second level. The dial and the indicator pointer move in the same direction and the difference between two levels in two float tanks is obtained directly from the gauge calibrated accordingly. A more simple system is often used where differential is read from a graduated glass tube manometer.

The floats used for the Loss of head Meter is generally of G.I. but in cases where the water proves to be corrosive FRP or any other corrosion resistant material can be used for longer trouble free life.

12.6 FLOW MEASUREMENT

In treatment of water, a distinction is made between two configurations of measurements of water quantities.

(a) Open system where flow rate is a function of water level.

(b) Close system where flow rate is a function of liquid velocity.

Each system has its specific applications. The first method is used where water can be transported by gravity, where as the second method is normally used when the flow is under pressure.

The flow measurement in open channel implies that the system is connected to outside air. The water flowing through the channel has an open surface and cross section of channel is restricted in order to increase the velocity of the liquid. The acceleration results from the conversion of potential energy. The resulting level difference is suitable for the measurement of flow rates.

Various systems have been developed using this basic principle for measurement of flow for many units of water treatment plant. The instruments for this purpose may be of following types.

(i) Flow rate measurement with float device.

(ii) Flow rate measurement with bubble injection.

(iii) Flow rate measurement with capacitive measuring system.

(iv) Flow rate measurement with Echo sounder devices.

Each system has its own advantages and disadvantages. Therefore, the selection has to be made on a case to case basis. Generally the types which are quite extensively used at various units of water treatment plant are the float operated ones which comprise of a float operated flow indicator operating in conjunction with Parshall flume which acts as a basic flow element. Float and wire rope material should be non-corrosive while indicator enclosure should be weather-proof suitable for outdoor installation. In case a flow transmeter is also incorporated in the enclosure for remote indication of flow, then the electronic circuit should be coated with humidity resistant paint to make it suitable for high humidity atmosphere, say up to 95% RH.

The methods incorporating bubble injection and capacitive measuring system have inherent disadvantages and hence their application is limited.

The system which is becoming very popular for such an application is Echo sounder device, due to its freedom of movement, lack of wear, easy installation and simple transmission of measured devices, accuracy and reliability. A sensor which is mounted above the channel transmits a sonic or ultrasonic signal which is reflected from the surface of the water and returns an echo. The travelling time of signal is a proportion of the water level. The measuring amplifier generates a linear current which is proportional to the level and transmits it to a linearizer which converts the current into a signal proportional to flow rate.

Meters, recorders, analogue/digital converters etc. can be connected to the output of linearizer. Since the liquid does not come in contact with any part of the instrument and due to the absence of any moving parts, such systems may require less maintenance.

12.6.1 FLOW MEASUREMENT IN CLOSED SYSTEMS

Where flow measurement is in closed systems, rotameters, turbine flow meters, electro magnetic flow meters, venturi or orifice meters etc., can be used depending upon the application. Some of measuring devices in closed system are:

a. Propeller/turbine meter

b. Venturi tube

c. Pitot tube

d. Variable area meters

e. Kennison or parabolic nozzle

f. Electromagnetic flow meters

g. Ultrasonic flow meters

(a) Propeller/Tubrine Meter

These operate on the principle that liquid impinging on the propeller or turbine will rotate it at a speed proportional to the flow rate. The meter is self contained and requires no auxiliary energy. The meter, however, should run full at all times. The accuracy is generally ± 2%. However, this meter should not be used with liquid containing solids in suspension.

(b) Venturi Tube

This operates on a principle that a fluid flowing through a meter section that contains a convergence and constriction of known shape and area will cause a pressure drop at the constriction area. The difference in pressure between inlet and the throat pressure is proportional to the square of the flow. The accuracy is generally around 0.75% of flow rate. The venturi tube must run full at all times.

(c) Pitot Tubes

Pitot tubes operate on the principle where velocity head is converted to static through a meter section that contains a convergence and constriction of known shape and area will cause a pressure drop at constriction area. The difference between inlet and throat pressure is proportional to the square of the flow. Pitot tube must be calibrated individually. The accuracy of pitot tubs is around ± 1% of actual flow rate. Upstream piping configuration substantially effect the accuracy of measurement.

(d) Variable Area Meter

The area meter or rotameter operates as the pitot tube with float position a function of viscous drag that is differential head or pressure. In the variable area meter, the constriction area varies while maintaining an essentially constant pressure drop.

The meter consists of a plummet or float and an upright tapered tube. This plummet is lifted to a state of equilibrium due to various forces acting upward and downward. The reading scale attached to the tube is essentially linear because of constant differential pressure. The accuracy is usually ± 2% of maximum scale.

(e) Kennison or Parabolic Nozzle

This operates on the principle that liquid flow through partially filled and properly graded pipes when passed through known constriction will produce a hydraulic head at a specific area of constriction. This head is essentially directly proportional to flow provided nozzle has free discharge. The accuracy for these nozzles is usually ± 2%.

(f) Electro Magnetic Flow Meters

This meter has an insulating inner finer and operates on the principle that any conductor, be it a bar of steel or column of conductive liquid passing through the lines of force of a fixed magnetic field which generates an electromotive force (d-c voltage) directly proportional to rate at which the conductor is moving through the field. Each meter requires calibration and must run full at all times. The system accuracy is ± 1% of meter scale for velocity range of 0.9 to 9.1 m/s. Below this velocity, the system accuracy may fall to ± 2%.

(g) Ultrasonic Flow Meter

This is an obstruction-less flow measuring system that can be installed in pipelines carrying liquids. The two interrelated components comprise the ultrasonic flow metering system and the sensor containing ultrasonic transducers and transmitter package. The transducer receives and sends pulses. The pulses are transmitted against or along the direction of flow alternately. The pulse transit time is expressed in terms of frequencies. The average fluid velocity is proportional to the difference between these two frequencies.

12.7 FILTER FLOW CONTROL

During operation of rapid gravity filter, impurities brought up are deposited in the pores of filter bed, increasing the resistance against downward water movement. With the other factors unchanged, a drop in filtration rate would occur. A similar drop in filtration rate would take place when raw water level above filter bed goes down or the filtered water level downstream of the bed goes up, while the reverse movement would result in an increase of the rate of filtration. With regard to effluent quality, however, the filtration rate should be kept as constant as possible, while particular sudden fluctuations should be avoided. An abrupt increase in filtration rate might cause impurities from the raw water to breakthrough the filter bed, impairing effluent quality, while with negative heads a sudden reduction in the rate of filtration might release gas bubbles that have accumulated in the filter bed. When these gas bubbles travel upward, holes might be produced in the filter bed, through which the raw water can pass without proper treatment. A positive control of the rate of filtration is, therefore, necessary.

Rate control is obtained by inserting an additional loss of head in influent line (upstream control) or effluent line (downstream control) and adjusting this loss of head in such a way as to keep the supply of raw water or the abstraction of filtered water constant at the desired value. Since the rapid change in operating- conditions occur, constant supervision and having automatic control by mechanical, hydraulic, pneumatical or electrical means becomes essential.

Generally the flow controller is a double beat equilibrium valve controlled by a float placed in the controller chamber of filter. The flow control valve is designed and works on the principle of hydraulics. The filtered water from the filter discharges into an inspection box and then over a weir built in the wall of the inspection box into pure water channel at a lower level. The height of water over weir indicates the rate of flow and this rate setter is used for operating the outlet control valve which consists essentially of a double beat equilibrium valve controlled by a float placed in the control chamber built within inspection box but isolated from it. In the inspection box is fitted, a rate setter, which consists of a vertical tube, the bottom of which opens into the control chamber and into which is fitted another sliding tube, the top of later being in the form of circular weir.

The height of the circular weir is so adjusted that when the water in the inspection box flows at any particular level (which determines the rate of flow of filtrate) a portion of water spills over the circular weir into the trumpet pipe and trickles into the control chamber. In the later is provided a small submerged orifice through which there is a continuous discharge of water flowing into the drain and when there is a balance of flow between the water trickling in and out of the control chamber the level of the float in this chamber remains constant, the slightest difference in the rate of inflow and outflow will upset this balance and later the level of the float which will automatically adjust the opening of the control valve until this balance is restored The trumpets pipe can be raised or lowered to give any desired flow and a calibrated scale is provided on the rate setter to set the filter to operate at required rate of filtration. This double beat valve is also controlled by a second float in the filter. This float closes down the control valve when the level of the water drops to about 15 cm above filtering surface where by preventing the filter being run dry. The controller is also provided with an automatic slow starting device.

12.7.1 Filter Flow Control Valve

The general arrangement of this type of outlet controller with associated accessories, is illustrated in Fig. 12.1 and 12.2.

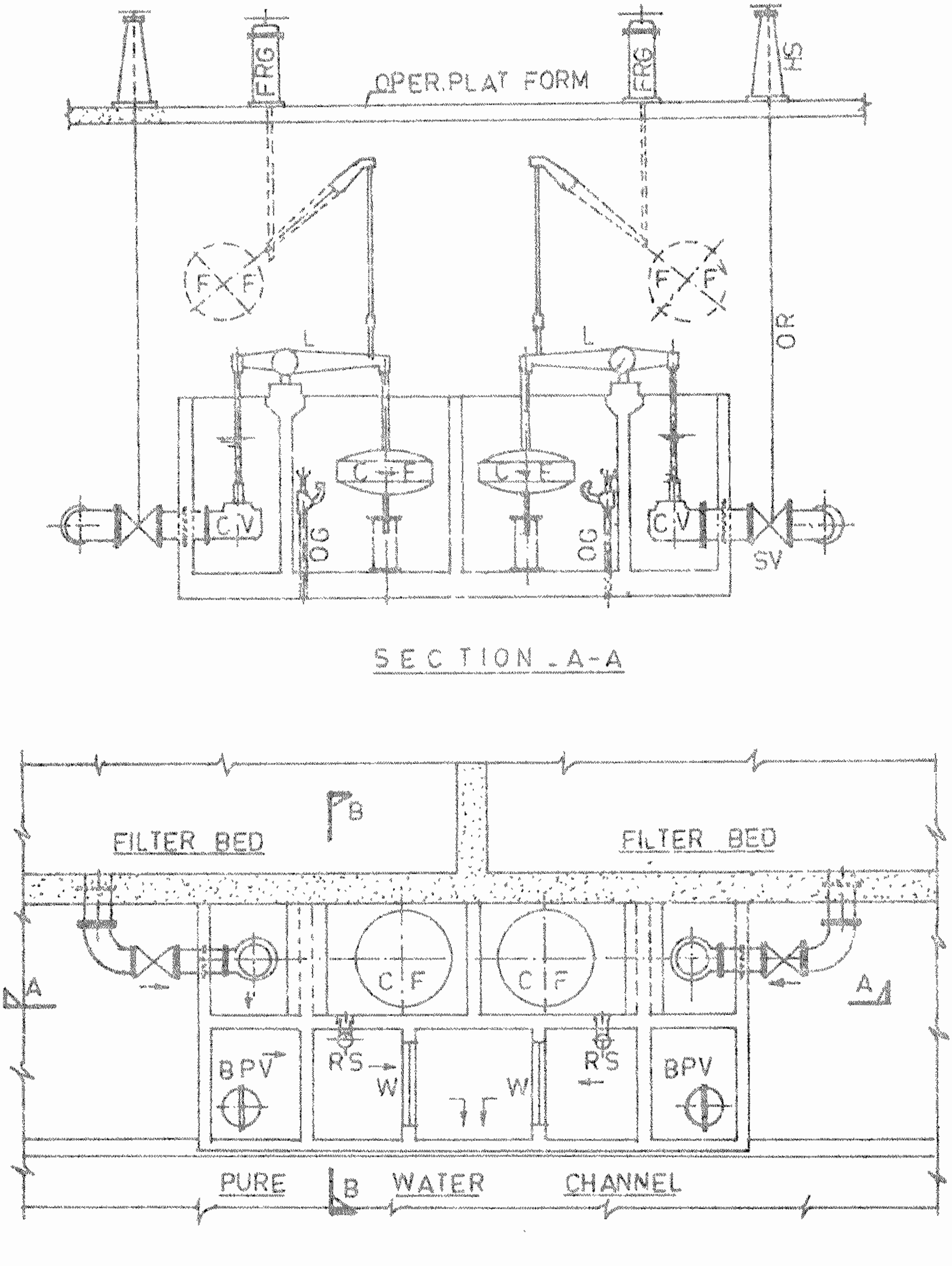
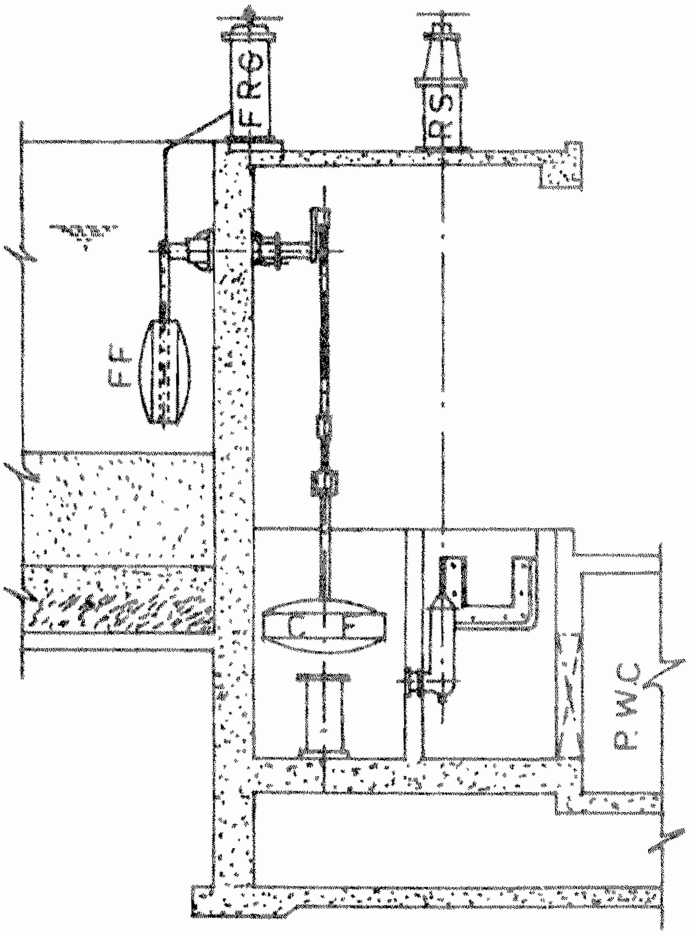


FIGURE 12.1 : PLAN OF OUTLET CONTROLLER CHAMBER
(NOTE FOR LEGEND REFER Fig. 12.2)



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LEGEND

CV......CONTROL VALVE
CF......CONTROLLER FLOAT
OG......ORIFICE GEAR
F.F.....FILTER FLOAT
FRG..FLOAT RAISING GEAR
RS.....RATE SETTER
BPV...BY PASS VALVE
W......WEIR
L......LEVER
SV.....SLUICE VALVE
OR....VALVE OPERATING ROD
HS.....HEAD STOCK
PWC.PURE WATER CHANNEL

FIGURE 12.2 : DETAIL OF FILTER CONTROLLER CHAMBER

The filter control valve is of CI construction with floats of GI, FRP, copper or any other corrosion resistant material.

The other type of controllers such as venturi, etc., work on pressure differential system which sends a signal to the controller. These differential pressures are reflected directly on the piston moving at a certain distance depending upon the difference between the pressures being exerted. The pressure is balanced by counter weights thus regulating the valve opening and closing. The controllers compare the actual flows with the set flow control points. According to the difference, the controller closes or opens the component, giving the discharge (butterfly valve, diaphragm, syphon).

The declining rate filter, however, does not require such control arrangement. However, to control the excess flow beyond the design capacity of any filter, a restrictor valve is introduced at the outlet so that filter is not allowed to operate at a filtration rate higher than assumed design values.

12.8 RATE OF FLOW OF CHEMICALS

For regulating alum flow or polyelectrolyte flow where used, the chemical solution is fed by gravity from solution tanks to a constant head box generally located near the dosing point. The constant head box (illustrated in Fig. 12.3.) is fitted with a PVC float operated stainless steel valve to keep constant level in dosing box. The rate of chemical flow is regulated by a stainless steel tapered needle valve over a stainless steel orifice in the constant head box. A scale gives directly the rate of flow of chemicals corresponding to opening of tapered needle valve.

For regulating lime solution being dosed, generally a V-nonch assembly with adjustable M.S shutter and a graduated scale is used. Regulated flow of lime solution as observed by head over V-notch indicated by graduated scale is allowed to flow by adjusting opening of M.S. shutter while the excess lime solution over flows back to the chemical tanks. (Fig. 12.4.)

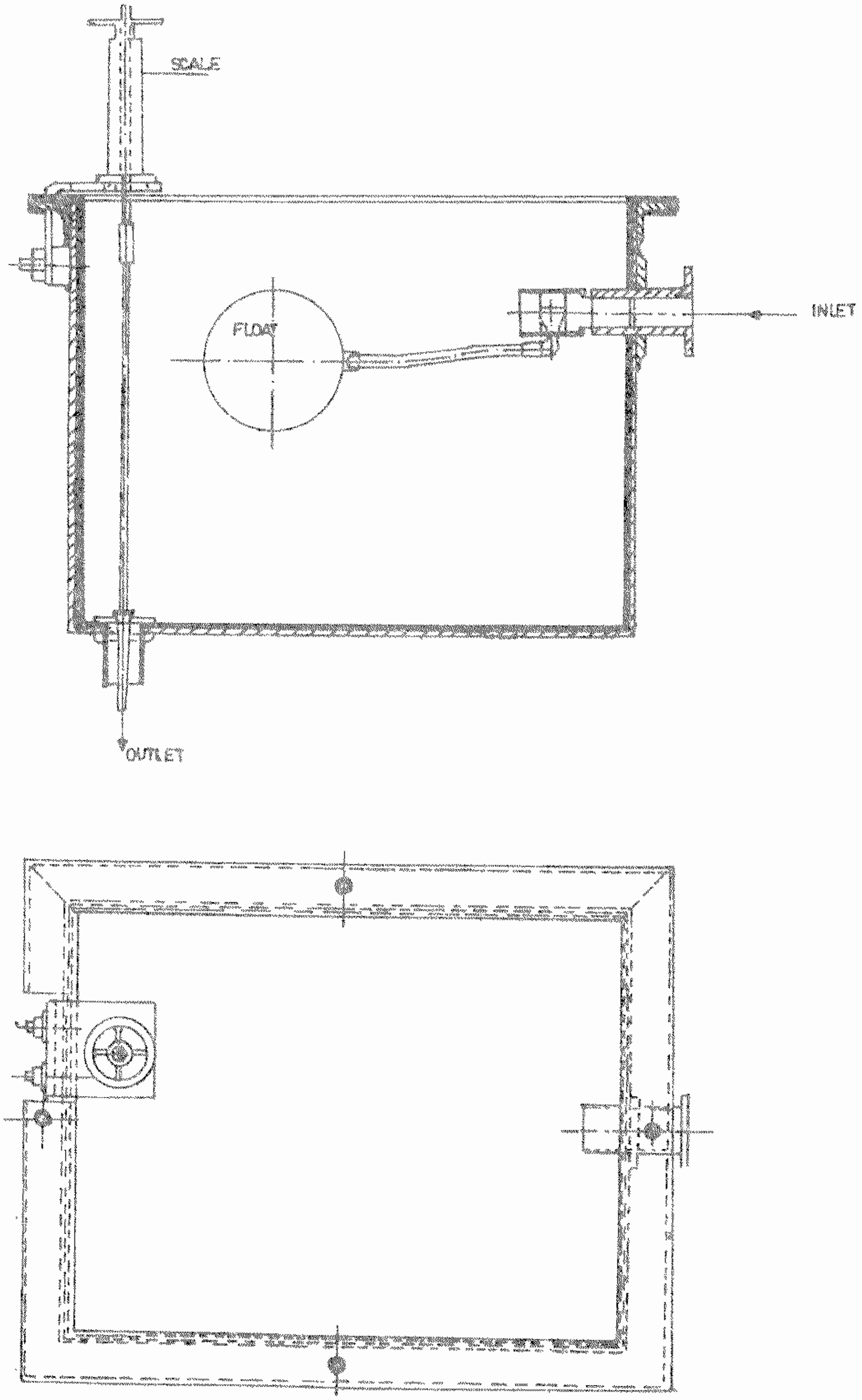

FIGURE 12.3 : M.S. RUBBER LINED CONSTANT HEAD BOX FOR ALUM DOSING

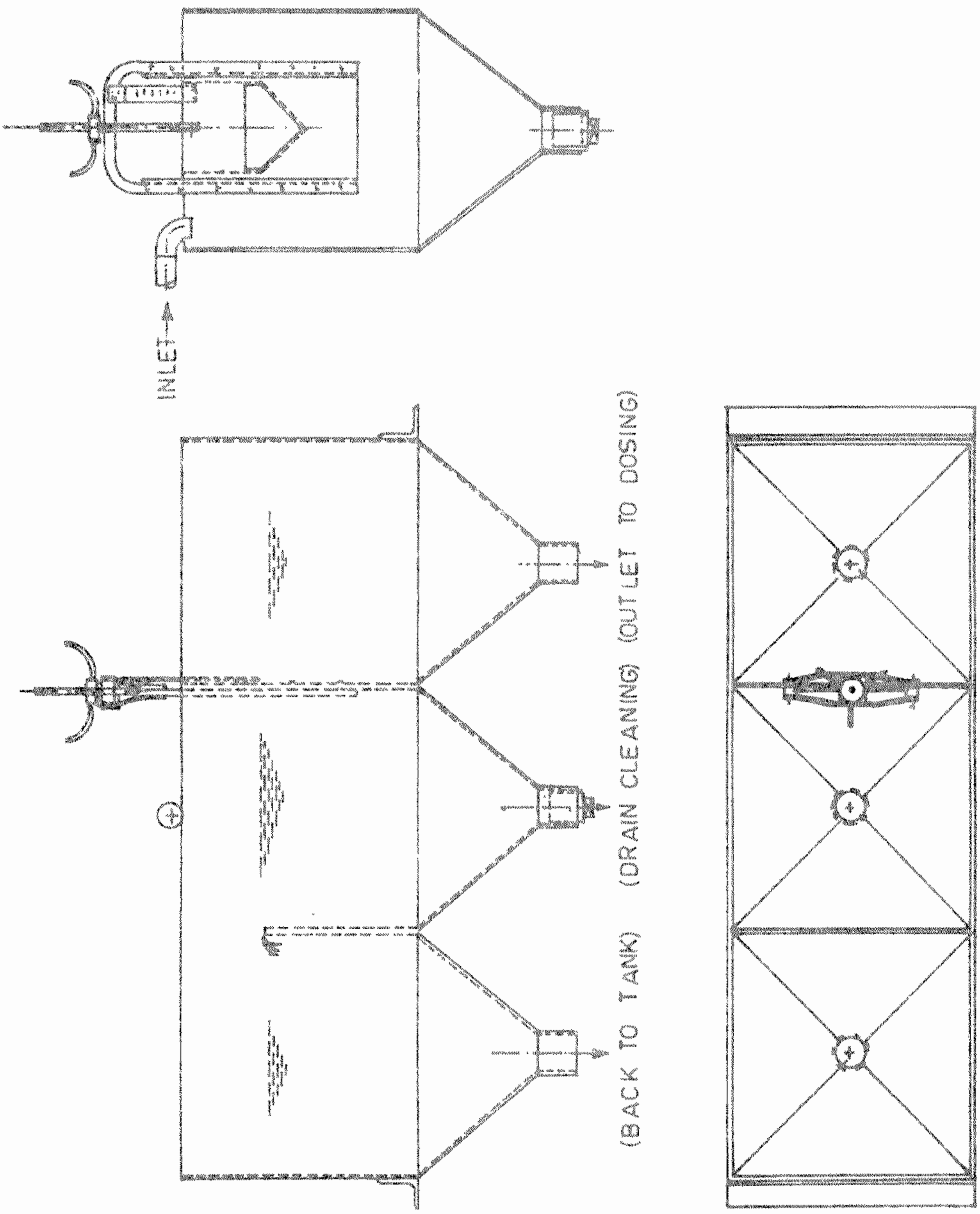

FIGURE 12.4 : LIME DOSING TANK

12.9 PRESSURE MEASUREMENT

Pressure is a parameter that is used extensively in testing and monitoring the performance of various types of pumping equipment, the monitoring of pressure gradients in pressure conduit and the regulation of pressure within required limits.

The most common types of pressure gauges are:

(1) Liquid: air and liquid: liquid diaphragm

(2) Bourden tube

(3) Strain guage

(4) Bellows.

The liquid: air diaphragm is a sensor unit using principle of balancing pressure across a diaphragm with a liquid on the side and air regulated by a nozzle-baffle system on the dry side. Due to change in liquid pressure the movement of diaphragm is balanced by build up of air pressure. Liquid system adopts a liquid instead of air.

The gauges should have minimum 100 mm size dial and should have range 1.5 to 2 times the normal operating pressure. The gauges are generally suitable for pressures up to 20.0 meters of water with accuracy of $\pm 1\%$ of maximum scale.

In the case of strain gauge, the sensor is used to measure dimensional change within or on the surface of a specimen when subjected to mechanical, thermal or combination of both inputs. The electrical type strain gauge is most frequently used and is based on measurement of capacitance, inductance or resistance change that is proportional to strain.

The Bellows element is a most conventional method of sensing pressure. It consists of a multiconvoluted bellows and its displacement by pressure change drives a mechanical linkage connected to indicator.

The Bourden tube works on a principle that a curved tube with a cross-sectional area that is not a circle will tend to uncurl as the area tries to become circular when subjected to pressure changes. The uncurling motion is calibrated to provide an indication of pressure. The pressure range for this type are from 0 to 7 and 0 to 350 kg/cm^2 with an accuracy of $\pm 1\%$ of full scale. This type is suitable where the liquid handled is noncorrosive type while for corrosive application, diaphragm type is preferable.

Wherever the pressure gauges are used in conjunction with reciprocating pumps pulsating dampener should be incorporated in the pressure gauge. All pressure gauges should have external zero setting mechanism and safety blow out mechanism. Pressure gauges installed in open should be of weather proof construction. Each and every pump and air blower should have pressure indication as an essential requirement.

12.10 WATER QUALITY

Water quality monitoring is essential for proper functioning of plant and to ensure desired quality of treated water. The water quality monitoring can be achieved in the plant laboratory by testing samples of raw and treated water at discrete intervals, using laboratory testing equipment. Reference may be made to Chapter 15 for details regarding recommended minimum laboratory tests and equipment.

12.11 OPTIONAL INSTRUMENTATION AND CONTROLS

12.11.1 LEVEL

(a) Raw Water Flow Control by level in clarified/filtered water tank

Where the flow to plant is controlled by the level in clarified/filtered water storage tank, a float operated electrically or pneumatically actuated inlet control valve may be used. The flow control of the inlet valve can be continuous or discrete using remote manual operation. In continuous control system, the level transmitter which transmits the level in the tank gives an input signal to a Pressure Internal Difference (PID) controller and the controlled output signal regulates the flow control valve on a continuous basis. In a discrete control system with remote manual operation both the raw water flow to plant and level of water in clarified/filtered water tank is observed on the control panel and the opening of the inlet control valve adjusted by remote manual inching operation from the control room. The push buttons, auto circuits, etc., are all mounted on the control panel. The raw water inlet control valve is also to be provided in both cases with a manual override.

(b) Level Annunciation and Auto Control Of Pumps

For ease of operation, it is advisable to provide high and low level annunciations for all the tanks at some centralised remote location where the control panel is housed. These alarms can be obtained from level switches similar to the one described at 12.5.1 (c). However, in chemical preparation tanks only low level annunciation serves its purpose. In tanks, where the draw-off of filling is by pumps, level switches can be used for auto starting/stopping of the pumps at high and low levels. Care should, however, be taken to have separate level switches for annunciation and for control of pumps.

(c) Remote Indication and Annunciation of Loss of Head Across Filters

Where the number of rapid gravity filters is large, it is sometimes preferable to have remote indication for loss of head across filters in individual filter consoles and alarm annunciation at high loss of head denoting need for backwash, for the filters at some centralised remote location. Differential pressure type electronic transmitters mounted in

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field near filters may be used for this purpose with pressure tapping from after the control valves and over sand media respectively.

Electronic transmitters should be suitable for high humidity atmosphere upto 95% RH and should give 4-20 m.a. signal. Remote indicator can be analog or digital type with facility for zero and span adjustment.

12.11.2 FLOW

(a) Remote Indication of Raw Water Flow

Raw water flow indication at a remote location may also be provided. This will facilitate operation of inlet valve as also in data-logging. The remote flow indicator may be of analog or digital type. It is also preferable to have an intergrator to know the cumulative flow to the plant. This integrator should be hand reset type only and the reset facility should be provided in such a way that accidental resetting of counter is avoided.

(b) Remote Indication of Rate of Flow Through Filters

Remote indication of rate of flow of individual filters may also be provided. For the purpose, float operated electronic type two wire transmitter with 4-20 m.a. output may serve the purpose using the filter outlet weir as the basic flow element. A differential pressure transmitter with an orifice in the filter outlet pipeline acting as the basic flow element may also be used. The remote indicator can be analog or digital type. Integrator similar to one described in Clause (a) above may also be used to know cumulative flow through the filter.

(c) Wash Water Flow Indicator

Wash water flow to filters may be measured locally by installing a rotameter in the main wash water header line to filters. The rotameter is usually a metal cased bypass rotameter with stainless steel float, stainless steel orifice and carrier ring assembly. In cases where remote indication of wash water flow to filters is desired, a differential pressure transmitter using a stainless steel orifice in the wash main header as basic flow element, may be used with a digital or analog remote indicator placed at a convenient location for the operator.

Repeat indicators of wash water flow may also be installed in individual filter consoles where such a system is adopted. However, the same need not be kept 'ON' at all times. The indicator is to facilitate backwashing and as such may be switched 'ON' for that period only.

(d) Chemical Flow

For regulating flow of chemicals solution, positive displacement metering pumps with 0-100% capacity mechanical stroke adjustment by means of a micrometer dial screw on the pump may be used.

The stroke adjustment may be manual/ remote by means of an electrical stroke positioner on the control panel.

12.11.3 PRESSURE SWITCH APPLICATIONS

In applications, where a minimum fluid pressure is required in a particular pipeline, a pressure switch may be incorporated for annunciation as well as auto trip of the connected

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equipment. For example, certain pumps requiring external water supply for cooling of the bearing should have a pressure switch on cooling water line to pumps.

12.11.4 Filter Console

Filter consoles for each individual filter can be provided when such an operational system is called for. The filter console table can be of FRP/M.S. sheet epoxy painted framed structure. All filter controls can be attended to by the operator from the individual filter console.

Open/close push buttons for filter inlet, outlet, wash water inlet, drain and air inlet valves are provided in the filter console along with their open/close indication. In such an operational arrangement all filter valves are to be pneumatically or electrically actuated. Control of air blowers for air scouring of filters are also incorporated in the filter console. If desired, wash water flow indicator, filter loss of head indicator and filter rate of flow indicator can also be incorporated in the filter console table.

It is also possible to have programmable logic based filter washing arrangement for the filters. The programmable controller should have required number of outputs each to be programmed independently and for pre-determined durations, to be decided at the time of commissioning.

12.11.5 Clarifier Desludging

A programmable logic based clarifier desludging arrangement may be provided to open the clarifier desludging valves at adjustable predetermined time intervals for an adjustable predetermined duration. In such a system of operation, the desludging valve will have to be electrically or pneumatically actuated. In case of pneumatic mode of operation, the solenoid valve used for the purpose should be of SS-316 or equivalent construction while the solenoid coil should be epoxy moulded, suitable for outdoor installation. The programmable controller may be located at a remote location preferably in the central control panel of the plant. Positive indications of valve operation by way of limit switches may also be provided near the programmable controller.

12 11.6 Water Quality

On-line instruments with annunciation for limit values may be provided for quality monitoring of the water treatment plant, for the following parameters.

(a) Turbidity

On-line turbidity meters working on the surface scatter principles may be provided for indication of raw water, clarified water and filtered water turbidities. Alarm annunciation can also be provided in case the turbidity of clarified water or filtered water is outside their respective acceptable values.

(b) pH

On-line pH sensors with preamplifier and two wire pH transmitters, if necessary, can be used for remote continuous pH indication. The pH transmitter should be housed in a weather-proof enclosure. Alarm annunciation can be provided in case pH of clarified/filtered water is outside acceptable limits.

(c) Residual Chlorine

On line residual chlorine sensor and residual chlorine two wir transmitter can be used for continuous remote indication of free residual chlorine after chlorination. The two wire transmitter should be housed in a weather-proof enclosure.

Free residual chlorine can also be continuously indicated by an amperometric chlorine residual analyser housed near main control panel. Sampling pumps at field are required for amperometric measurement and indication

In both types alarm annunciation can be provided in case residual chlorine in water is outside acceptable standards.

12.12 INSTRUMENT-CUM-CONTROL PANEL

To facilitate plant operation and monitoring, it is advisable to have a centrally located instrument-cum-control panel. This panel should have all the annunciations, status indication lamps, instruments, start/stop or open/close push buttons and logic wiring circuits. Chemical tank agitators, start/stop facility and status indication lamps may be on separate panel.

It is also advisable to have all logic circuit wiring in this control panel only so that Motor Control Centre (MCC) modules wiring can be standardised to advantage of draw-out features of MCC. Similarly, it is also advisable to route all control and instrumentation cables through this panel only. However, push button station cables, if so desired, may be taken directly to MCC.

For reasons of operators' safety, it is desirable to have control voltage not more than 100V, single phase, A.C.

The panel front sheet should not be less than 3 mm thick and single flap type doors with locking facility should be provided on back side. Panel height should in no case be more than 2400 mm and minimum operating height should not be less than 800 mm from floor level.

12.13 CONCLUSION

In the above chapter various methods of instrumentation and control system in water treatment plants have been discussed. As a general guideline the following should be considered while deciding the type of instrumentation & control required. Generally, for a plant of about 25 mld and below, simple and manually operated instrumentation should be preferred. This may include inlet flowmeters, level indicators for chemical tanks, overhead tanks, loss of head for filters, filter control equipment for filter, LOH glass tube graduated manometer for pressure measurement devices for pumps and blowers. For plant having capacity between 25 mld to 100 mld, semi-automatic equipment may be preferred.

However, valves may be manually or pneumatically operated. For a plant having more than 100 mld, it is advisable to go for electrically operated systems with instrumentation control panel or line quality measuring instruments with a remote indication. Flow measuring devices may be of ultrasonic type or electromagnetic type to have better accuracy. Tanks/sumps should be provided with automatic control for low and high level switches.

CHAPTER 13
OPERATION AND MAINTENANCE OF WATERWORKS

13.1 INTRODUCTION

Once the task of creating a water supply system is complete, the aspect which will have to be given the top most priority is the "operation and maintenance" of the system. Instances of poor operation and maintenance practices have on many occasions largely contributed to decreased utility, or even to an early failure, of newly constructed drinking water supply facilities. Thus the health and social benefits for which the facilities were designed have not been realised; capital investment have been wholly or partially lost, and scarce resources are expended on the premature replacement of equipment or for the rehabilitation of facilities before they have been in operation for the full span of their useful lives. Therefore proper operation and maintenance is the answer for deriving the benefit continually from the investments made.

Although each water works may have its own peculiar problems of operation and maintenance, there are many features of operation and maintenance which are common to all waterworks.

13.2 OPERATION AND MAINTENANCE

The maintenance of a water works may refer to up keeping the civil, mechanical and electrical components of a plant through normal repairs, so that they are able to function at designed capacity for their design period. It may further refer to such routine repairs as are necessary to prevent the components from malfunctioning.

The operation refers to the art of handling the plant and equipment optically so that the designed quantity and quality of water can be produced.

The operation of a water works refers to hourly and daily operation of certain component parts of the water works such as plants, equipments, valves, machinery etc., which are required to be attended to by an operator or his assistant. It is an important though routine work. Operators have to be trained properly before they are entrusted with the task of operation of specific plant, equipment, valves, machinery etc.

13.3 COMMON FEATURES OF OPERATION AND MAINTENANCE

In the operation and maintenance of a water works, there are certain common features to be considered by the authority controlling the water works. These are briefly stated below:

13.3.1 Availability of Detailed Plans, Drawings and Operation and Maintenance Manuals

When a water works is taken over for operation and maintenance, it must be ensured that atleast five to six sets of the detailed drawings, maps of each of the component of the water works along with all relevant 0 & M Manual are available with the operating authority. One of the sets may be preserved as a master set in apex office for reference. The other sets may be distributed to sub-offices in charge of relevant operation activity. All these sets must be corrected and updated whenever any additions/ alterations/ deletions are done to any of the structures and equipment.

13.3.2 Schedule of Daily Operations

For each of the activity where operators are employed, a detailed scheme and schedule of unit operations should be worked out and a copy of the same should be available with each operator. This schedule of unit operations may have to be altered to suit changes in raw-water quality, hours of availability of power, break-downs and up-set conditions etc.

13.3.3 Schedule of Inspection of Machinery

A regular schedule of inspection of machinery, equipment their lubrication and servicing programme must be prepared and circulated. Appropriate supervisory control should be exercised to see that these inspections, lubrications and servicing are being regularly carried out.

13.3.4 Records

For each piece of equipment and machinery a record register should be maintained in which all records of the equipment such as servicing, lubricating, replacement of parts, operating hours (including cumulative) and other important data is entered.

13.3.5 Records of Quality of Water

Complete records of bacteriological and chemical analysis of water from source to the consumers tap should be maintained and reviewed. Charts could also be prepared for the important characteristics of the water and any changes in these characteristics as compared to the standards must be taken note of.

13.3.6 Records of Key Activities of O & M

For planning future augmentations and improvements of a water works in operation, it is advisable to maintain certain key records such as daily and cumulative supply over the years, number of connections of various sizes given and cumulative number of connections each month, water treated and the supply billed.

13.3.7 Staff Position

Appropriate charts indicating the standard staff for each of the unit of operation and maintenance and the staff actually in position (by names if possible) shall be maintained at each office for review.

13.3.8 Inventory of Stores

A reasonable assessment of the stores and spare parts of machinery required over a period of time say, one year or half a year can be made and an inventory of the same prepared. Issues and replacement of store articles could be watched and procurement procedures laid down and supervised. The aim should be that any material required for replacement is available at any time for the maintenance.

13.4 FEATURES OF OPERATION AND MAINTENANCE OF INDIVIDUAL COMPONENTS OF WATER WORKS

13.4.1 Source and Intake Works

(a) Sanitary Survey

Sanitary Surveys at regular intervals at field management levels and inspections at supervisory management level should be conducted. The catchment area of the source should be located on the maps. Potential sources of pollution observed in the catchment should be marked. The type of pollution e.g. industrial/domestic waste discharges, wastes of animal origin and agricultural run-offs should be determined.

The quality of such discharges has to be ascertained and its likely effect on water being drawn at source should be mentioned. Reports of such surveys should be promptly sent to the Pollution Control Authorities as well as water works authorities to promote corrective action. Procedure for monitoring of preventive action taken should be laid down and observed. An instant action plan for providing chlorination of raw-water should be available and brought into effect under such circumstances.

(b) Measurement of Flow

In cases of sources such as springs, rivers, canals, etc., there should be a permanent arrangement for recording daily flows near the intake works. Appropriate records in the form of graphs showing variation of flows in the source for each month in a year and for each year shall be maintained. Rain gauge stations should be established to record daily rainfall in the reservoir catchment and appropriate rainfall records should be built up and compared with discharges/storages available. In cases of reservoirs, the regime tables for filling and emptying of storages should be maintained for each year.

13.4.2 Maintenance of Dams

(a) Pre, during and post monsoon inspection of dams should be undertaken to observe settlement, longitudinal/transverse cracks in the embankment/masonry structures.

(b) Behaviour of spillways should be observed during floods. Procedures for fool proof operation of spillway gates should be prescribed and observed.

(c) In case of earthen dams, special attention should be paid to slipping of slopes, damages and water seepage. The functioning of sand galleries, drains, relief wells should be watched carefully. In case of masonry dams, sweating, leakages, leaching of mortar of appreciable magnitude from masonry should be immediately attended to. Pointing of damaged faces of masonry should be attended to promptly.

13.4.3 Maintenance of Intakes

(a) It should be ensured that sufficient water level is maintained at headworks in order to ensure drawal of required quantity of water into intake works without vortex formations.

(b) All intake strainers should be cleaned at frequent intervals particularly during monsoon to prevent entry of fish or floating matter into intake works.

(c) All damages to structural components of intake works particularly during floods should be promptly repaired.

(d) Sufficient stocks of rubble should be maintained at intake works site for use to temporarily overcome the problems of scours at spillways and other places.

(e) A schedule of painting of steel and other structural parts of the intake works should be prepared and followed scrupulously to avoid damages to the structure.

(f) All raw-water holding structures such as intake wells, jackwells and inspection wells should be desilted during and immediately after monsoon to remove settled silt.

13.4.4 Maintenance of Pumps & Pumping Machinery

(Details are given in Chapter 11)

13.4.5 Maintenance of Transmission Systems

The transmission mains would include raw/treated water pumping as well as gravity mains from source to treatment works, treatment works to master balancing reservoirs (M.B.R.) and from M.B.Rs to service reservoirs in the distribution system. The maintenance problems to be attended to for various types of pipes used in the system could be briefly summarised as follows:

(a) M.S. Pipes Laid Above Ground

(i) Pipes should be painted atleast once in five years to prevent corrosion.

(ii) Appurtenances such as sluice valves, air valves, expansion joints, rollers should be checked, cleaned atleast twice a year and worn-out parts replaced. The cleaning and lubrication of rollers should also be done twice a year, preferably pre and post monsoon.

(iii) Expansion joints should be inspected every month.

(iv) The catch drains provided for the portion of water mains laid in cutting should be cleaned before onset of monsoon so that no water accumulates in the cutting portion, resulting in uplift pressure on pipes.

(b) All Pipes

(i) Sufficient stock of spare pipes and specials should be maintained for replacement of damaged ones.

(ii) Regular leak detection surveys should be undertaken particularly for bursting of pipes and leaky joints.

(iii) A detailed record of break downs and leaks observed and repaired should be maintained section-wise so that more vulnerable lengths could be identified and special measures to repair/replace them could be undertaken.

(iv) A regular schedule of inspection and attendance to all valves including air and scour valves should be drawn up and the same followed scrupulously. Special attention should be given to air valves

13.5 OPERATION AND MAINTENANCE OF WATER TREATMENT PLANTS

13.5.1 PROBLEMS

The person in charge of the maintenance and operation of water treatment plants should have a thorough knowledge of the functions of the several units under his control. The problems that may be posed before him may relate to those arising from (a) poor design, (b) faulty execution, or (c) special situation during operation.

A resourceful operator should be in a position to bring to the notice of the concerned person, any faults in design and execution giving rise to problems during the course of operation and rectify them immediately. The other problems which are to be tackled at the operational stage are mainly those which arise out of

(a) Fluctuation in the quality of the water;

(b) Fluctuation in the quantity and changes in the flow pattern;

(c) Malfunctioning of the unit(s); and

(d) Mechanical and electrical equipment.

13.5.2 REQUIREMENTS

Maintenance should be carried out in a manner which prevents emergencies and unscheduled shutdowns. An efficient maintenance requires considerable skill which can only be acquired by experience, study and practice. Basically, any maintenance programme should observe the following general rules:

(a) Keep a set of plans giving details of the several units and indicating the layout and position of all pipelines and appurtenances;

(b) Establish a systematic plan of daily operations;

(c) Establish a routine schedule for inspection of machinery and lubrication and maintain records thereof. Instructions for lubrication, the type of lubricant suggested and the frequency of lubrication should be drawn out;

(d) Main data and record of each piece of equipment giving details of cleaning and replacement of worn parts and other data of importance such as unusual incidents on faulty operating conditions. Details for any special equipment should be obtained from the manufacturers;

(e) Keep a record of analysis of water collected at various points from the source to the distribution system and observation on the effect of such quality on the several units of operation; and

(f) List out safety measures including good house-keeping.

13.5.3 RAW WATER

The problem will mainly relate to the change in the quality of the raw water due to natural causes and by inadvertent pollution of the source.

In the case of a river source, a sanitary survey of the catchment area should be underta:ken at regular intervals and water samples taken at significant points where pollution is likely to take place. The analysis of these samples will reveal the degree and nature of pollution and thus help in taking the necessary measures to check or control the pollution. If the fluctuations in quality are rapid, the surveys should be undertaken at shorter intervals. Turbidity is not a special problem as the dosage of the coagulant is adjusted on a daily routine. On the other hand a sudden rise in chlorides content will indicate pollution most probably due to sewage. In such cases, more confirmatory test should be undertaken such as for nitrogen in its various forms, dissolved oxygen, oxygen absorbed and chlorine demand to help the operator to decide whether pollution has taken place and to fix the dose of prechlorination needed.

In the case of a lake as a source the periodical biological and physical examination of the samples will indicate if there is any need for control of algae which may lead to taste and odour problems or clogging of filters. Samples taken at different depths in the lake will indicate the level at which water should be drawn to get the best quality of the water.

13.5.4 FLOW MEASURING DEVICES

Float sump should be periodically cleaned to see that silt does not accumulate which may affect the proper functioning of the float. Charts and pen recorders should be stocked adequately. Annual or more frequent calibration of these devices is necessary. Annual servicing and checking of the instruments is imperative.

13.5.5 CHEMICAL FEEDING UNIT

Alum preparation tank is to be painted annually by anti-corrosive paint. V-notch weirs and floats and floating arrangements should be cleaned daily. Enough spares for the mixing device in the chemical preparation should be stocked. Setting of the V-notch should be checked periodically.

Sometimes, if the alum dosing equipment is not in order, the alum slabs are just dumped in the raw water channel. This is bad practice and should not be adopted as it will mean wastage of alum and imprope dosing of alum. Alum should be made into a solution and dispensed until the dosing equipment is rectified. The optimum dosing of alum and coagulant aids should be based on a proper and detailed laboratory study including Jar Test. The chemical feeding rate should be controlled, depending upon the needs from time to time.

13.5.6 RAPID MIXER

Adequate spares should be kept ready in stock for timely replacement when necessary. Life of the equipment could be prolonged by periodical painting with anticorrosive paints.

13.5.7 SLOW MIXER

Slow Mixer should be operated continuously for avoiding sludge build-up. All equipment should be painted with anti-corrosive paints every year. Mechanical devices should be properly lubricated and worn out parts replaced. In non-mechanical type of flocculators like baffle and tangential flow tanks, desludging atleast once in six months is necessary.

13.5.8 CLARIFIER OR SEDIMENTATION TANK

Annual overhauling and repainting of the unit should be done a month or two prior to monsoon.

Sludge lines should be kept free of chokages. The lines should be flushed with high pressure water if chokages are noticed. The telescopic sludge discharge device, when provided, should be checked for free vertical movement and O-rings replaced when leaky.

The traction wheels should be checked for alignment and rubber wheels replaced, if required.

The unit should be worked continuously to protect the mechanical parts from ill-effects of corrosion, malfunctioning etc., as well as problems from sludge build-up. Outlet weirs should be kept cleaned at all times. Algicide or bleaching powder may be used for controlling biological growth on weirs.

The important features in the operation of a clarifier are:

(a) The introduction of water into the tank with a minimum turbulence;

(b) The prevention of short-circuiting between inlet and outlet; and

(c) The removal of the effluent with the minimum of disturbance to avoid settled material being carried out of the tank.

Very often, a basin which is not functioning properly can be modified by making changes to the inlet and outlet devices by installing stilling baffles so as to improve any or all of the important features mentioned above. Algal growth, if any, should be controlled.

13.5.9 RAPID GRAVITY FILTERS

The common problems encountered are:

(a) Defective Gauges

Rate of flow gauges and loss of head gauges frequently get out of order. The operator should be conversant with the working of gauges and should be able to handle minor repairs. Necessary spares should be stocked.

However, even if the rate of filtration gauge is under repair, the filtration rate can be checked whenever desired by closing the inlet valve and observing the time during which the level of water in the filter falls by a measured distance.

For knowing the loss of head when the gauges are out of operation, a temporary arrangement consisting of two glass tubes on each side of a calibrated scale could be provided. One tube is to be connected to the effluent pipe between filter and controller and the other tube to the filter structure above the sand. The relative elevation of the water surfaces in these tubes indicates the prevailing hydraulic gradient or loss of head through the filter.

(b) Inadequate Media on the Filter Bed

Expansion of sand bed during backwashing should be kept within the limits to avoid carry over of sand to wash water trough which would lead to appreciable depletion of sand depth over a period of time. Sand depth should never be depleted by more than 10 cm, when the media has to be replenished. The entire bed should be taken out and additional sand mixed to give the required effective size and uniformity coefficient. Before starting the filter, the sand has to be backwashed to stratify the bed.

(c) Air Binding

This is caused due to the development of negative head and formation of air bubbles in the filter sand. This could be overcome by more frequent backwashing during these periods. Provision should also be made wherever feasible for increasing the depth of water over the bed by about 15 to 30 cm. There are chances of air being released if back wash is carried out by direct pumping. Air release valves should be provided on the pumping mains in such cases.

The solution lies in providing adequate depth of water atleast 1.5 meters over sand. If air binding persists, loss of heads may be limited to 1.5 meters instead of normal 2 meters. This will discourage air binding and will ensure reasonable length of filter runs.

(d) Incrustation of Media

This problem may arise as in the case of water softening with lime soda when sand gets coated with material that is difficult to remove by normal backwash. The remedy lies in washing the filter occasionally with sodium hydroxide (10 kg/m^2 area of bed) or bleaching power (20 kg/m^2 area of bed).

(e) Cracking of Sand Beds.

This occurs mostly when the water is lowered below the surface of the sand. Cracks in a sand bed under water may also arise due to the cementing of the grains by some material in the applied water. The vulnerable portion is near the filter walls, since the sand is drawn away from the walls. The rate of flow increases through such cracks allowing a heavier deposit of solids at these points, which in turn, intensifies the forces compacting the sand until a dense mass is formed. The degree of this mass may be limited, creating a dead area, resulting in an unequal distribution of the wash water. This can be overcome by the use of hand rake or by draining the bed and removing the clogged sand.

(f) Bumping of Filter Beds

Sometimes careless and indifferent operation may lead to "bumping" or "lifting" of the filter beds when switching on the back-wash for a minute to dislodge the sand bed and recommending filtration without going through the full back-wash cycle is adopted. This practice should be discouraged as the filtrate quality deteriorates considerably.

(g) Mud Balls

These are caused by the general buildup of materials not removed in back-wash. Mud balls accumulate at or near surface and in course of time clog the entire media.

By proper coagulation and settling of applied water, mud ball formation could be considerably reduced. Surface wash or surface raking, or shovelling at intervals helps reduce mud ball formation. Also compressed air scouring during backwash for periods of three minutes, instead of 1 to 2 minutes, effectively decreases mud ball concentration.

(h) Sand Boils

These are caused when disproportionately large discharges of wash water rush towards expanding the sand and displacing the gravel. The situation is encountered mostly due to the poor distribution of wash-water from the underdrain.

(i) Slime Growths

When slime growths are noticed on filters, the bed is cleared in the normal way and the water is lowered to the level of the sand bed. Then common salt is distributed evenly over the surface of the sand, using 7 kg/m^2 of filter area, after which the wash water valve is opened until water rises about 15 cm. above the sand level. The water is allowed to remain for 2 hours to dissolve the salt and then lowered to the bed level to be retained for 24 hours after which it is thoroughly backwashed before placing into service. If this procedure does not produce effectives results, it may be necessary to replace the media.

(j) Backwash Requirements

The waste water drains carrying filter backwash should be kept free of clogging or sediment. If the backwash water is led away quickly, there will be no backing up in water channels or into the filter bed. Incidentally, it may be worthwhile to consider setting up a plain sedimentation tank to recover the supernatant from the backwash water. For the small investment, the water recovery could be appreciable.

The requisite upflow velocity of backwash water should be maintained at the design rate for proper cleaning of the sand. The practice of backwash at reduced rate for longer periods should be avoided as it leads to wastage of water and washing becoming ineffective.

Backwashing of filters should not be based on arbitrarily fixed time schedules but the frequency should be in accordance with the filtrate quality and head loss measurement. Duration should be dependent upon the turbidity of the wasted water.

13.5.10 Slow Sand Filters

The inlet float valve should be periodically checked with a view to maintain the desired level in the bed.

The outlet weir arrangement should be checked periodically with a view to ensure the design rate of filtration. Where there is telescopic arrangement, it should be functioning smoothly and without drawing in water through the sides. Where manual adjustment is to be done with increasing filter heads, this should be done at specified intervals.

The filter head indicator should always be kept in working condition. When a filter is clogged, most of the head loss is restricted to the top layer of sand and if the filter head exceeds 1m, pressures below atmospheric can occur in sand gravel and in the under drains, leading to air binding or dissolved air coming out of solution. Occurrence of negative head can be avoided by placing the sill of the outlet weir in level with the top of the sand bed.

It is most important to avoid rapid fluctuations in filtering rates. Cleaned or resanded filters should be brought up gradually to the maximum filtering rate and maintained as far as possible at a constant rate until the head reaches the maximum of 1m when the bed should be taken up for cleaning.

On no account the filter bed be allowed to get reduced by disturbing the top of the sand as this will impair the bacterial efficiency of the filter.

13.5.11 Chlorinators

The chlorine demand of filtered water is to be satisfied and a free chlorine residual maintained to make it completely safe. Hence the operator should be careful in administering calculated doses accurately.

Bubbling the chlorine gas through the filtered water stored in the clear water reservoir by dipping rubber tubes connected to chlorine cylinder must be avoided. Chlorine application should be done through a chlorinator only. The chlorinator should be maintained properly. If the unit is out of order, the same should be repaired quickly and recommissioned.

A complete understanding of the principles of operation of chlorine gas feeders and familiarity with tests for pinpointing leakages are essential. Low capacity units require frequent cleaning of the rotameter and rate setter. Large capacity chlorinators must have vaporisers. The gas piping and feeders should be completely dismantled every one or two years to clean out accumulated impurities.

13.5.12 Clear Water Sump & Reservoir

Roofing should be periodically checked to ensure that no leakages are there so that pollution can be prevented. Ventilator outlets should be regularly checked and cleaned to guard against mosquito breeding and bird droppings. Cleaning of the sump and reservoir should be done regularly. Level recorder should be kept in working order at all times.

The total capacity of clear water reservoirs should be adequate for storage of treated water, especially during low supply periods at night when reservoirs become full. Instances are reported, where water from the filters have backed up into the inspecting galleries, thus reducing the rate of filtration. The remedy lies in having additional clear water reservoir in the plant, or arrangement for the final water to be automatically pumped to the balancing reservoirs in the town.

13.5.13 TREATED WATER

The quality of the water before distribution may be controlled by adjusting the calcium carbonate balance in the water to safeguard against corrosion or excessive scale formation in pipes. The periodical analysis of the water can also indicate if there is any biological growth in the main and if any further chlorination is needed to check it. The samples of water collected from several points should be routinely examined for residual chlorine and other chemical and bacteriological parameters.

13.5.14 PROBLEMS RELATED TO THE QUALITY & FLOW PATTERN

When flow gets reduced, it may not be desirable to cut out certain units but it is preferable to operate all the units with reduced flow conditions. In any case, the flow-through condition in the several units should be periodically studied using appropriate tracers. This will help to locate if there is any short-circuiting so that corrective measures can be adopted.

The flow conditions in open channels should be examined periodically to avoid obstructions and heading up which will affect the unit process especially the efficiency of the clarification units.

13.6 AERATORS

Aerators are required to be maintained in a clean condition so that maximum water surface and agitation are provided.

Slime and algae growth on the surface would require cleaning and periodic treatment with copper sulphate with or without lime to kill growth. The porous plates or tubes used with diffusion areators may become partly clogged either from dust in the compressed air or from the collection of sediment on the outside surfaces. When areators are shutdown, appropriate cleaning with detergents or acid and brush should be attempted. Cloggging of diffuser plates could be minimised by (i) maintaining air filters in effective operation, (ii) not over-lubricating air compressors and blowers, (iii) maintaining air pressure on diffusers, when compressors are shut down.

13.7 MASTER BALANCING RESERVOIRS AND ELEVATED RESERVOIRS

Important aspects to be considered during maintenance are:

(i) Measurement of inflows & outflows : Whenever measuring devices are provided, it should be seen that discharge at inlets and outlets fairly tally. It should be seen that water level indicators and recorders are in proper working order.

(ii) Structural Leakages : All structural damages and leakages should be promptly repaired.

(iii) Preventing External Pollution : The manhole opening, ventilating shafts and overflow pipes should be properly closed and protected with wire gauge from external pollution.

(iv) General cleanliness in and around the reservoirs should be maintained and observed. A garden around the reservoir structure may be provided.

(v) A programme for periodical cleaning of the reservoirs atleast once in a year should be undertaken. During such cleaning process there should be facility to bye pass the supply to distribution system.

(vi) Appropriate safety measures to prevent climbing of unauthorised persons should be provided. All the railings provided shall be maintained in a safe and firm condition.

13.8 DISTRIBUTION SYSTEM

Important aspects of operation and maintenance of distribution system are detection and prevention of wastage due to leakage. The object is to control the waste within reasonable limits. Further in case of intermittent supply, possibility of pollution of empty pipelines cannot be ruled out. Special inspection of pipelines through marshy or high water table areas, crossings across waste channels, pipes, etc., and in the vicinity of sewers should be carried out at regular intervals. Such areas should be identified on plans and bacteriological tests of tap water in such areas need to be done more frequently and results compared.

A regular programme of leak detection should be undertaken for the entire distribution system such that each section of the system comes up for leak detection atleast once in three years. Leaks and damages detected should be promptly repaired. The causes of wastage through leakages such as (i) high pressures in distribution, (ii) corrosive soils, (iii) corrosive water, (iv) inferior quality of pipes and fittings, (v) age of pipes, (vi) gland packings of valves etc. should also be ascertained. The repair work should tackle those causes as well

In a distribution system complaints are received frequently from consumers about

(a) Non-availability of required quantity of water

(b) Low pressure at the supply point

(c) Leakages & wastages through valves & pipelines

(d) Unauthorised connections.

One of the major causes of wastage is unauthorised connections. Procedures for granting connections need to be streamlined. The officer incharge of operation & maintenance of distribution system should have powers to inspect any household for water supply to know as from where that household is taking water.

The entire distribution system could be divided into sub-zones served preferably from one elevated service reservoir. The maintenance and operation of each zone of distribution system should be entrusted to atleast a junior engineer who should be made the authorised official of the controlling authority to receive and deal with the complaints. Appropriate registers should be maintained by him to record the complaints and to note in it the follow-up action till the complaint is redressed. If the complaint is such that it cannot be dealt with at his level, he should at once refer the matter to higher authorities under intimation to the complainant. Frequent vigilance checks in the areas having maximum complaints should be made a part of the duty of the supervisory staff.

It is preferable to have meters provided by the water works controlling agency after charging appropriate monthly rentals to the consumer. This enables effective control over defective meters. Meter repair workshops should be established to attend to repairs of meters promptly. Surface boxes and chamber covers of valves should be frequently inspected and kept in proper condition. Billing for an out of order meter for more than three times consecutively should be avoided. All attempts should be made to repair/replace out of order meters once these are detected.

Sufficient stock of meters and spares should be available at hand to keep almost every meter in the field in working order.

Comprehensive water rules should be framed to make the maintenance operation most effective.

The consumers should be made aware of difficulties and shortcomings in the maintenance and operation of water supply system. Adequate publicity and public relations are required to be developed for this purpose.

13.9 CONTROL OF QUALITY OF WATER

For a waterworks industry, ensuring an appropriate quality of water to the consumer is its primary responsibility. Quality control is, therefore, required at every step in the water supply process. The physical, chemical and bacteriological tests of water samples need to be carried out at as frequent intervals as required. Reference may be made to Chapter 15 for more details. The results of these tests should be studied and remedial measures taken promptly as and when required.

These tests are usually needed at:

(i) Source-to determine the raw water quality;

(ii) Treatment Plants-to determine whether the treatment is in conformity with raw-water quality; and

(iii) Distribution system-to determine whether adequate residual chlorine is present in the water supply to consumers.

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13.10 TASTE & ODOUR CONTROL

The following measures are applicable in taste and odour control:

(a) Routine examination of samples of raw, settled and filtered water and samples from distribution system for taste & odour;

(b) Periodic Treatment with copper sulphate and by chlorine;

(c) Routine maintenance by flushing distribution system, especially at hydrants served by dead-end mains; and

(d) Maintenance of records of consumers' complaints and corrective action taken so that it can serve as guide for future.

13.11 STAFF PATTERN

Recommended Staffing Pattern for Operations & Maintenance of Waterworks for various capacities is given in Appendices 13.1 to 13.7.

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CHAPTER 14
WATER WORKS MANAGEMENT

14.1 LEVELS OF MANAGEMENT

In India, 'Community Water Supply Systems' are normally managed by local bodies. In a few specific cases these are managed by State Government Departments, where the system is supplying water to more than two local body areas, the bulk supply component of the system is some times managed by statutory Water Supply Boards set up by State Governments. This service facility falls under the water supply and sanitation sector. The development of this sector is assisted at three levels.

14.1.1 GOVERNMENT OF INDIA (G.O.I.) LEVEL

Broad policies on sector development of water supply system in urban and rural areas are formulated and circulated to State Governments and Union Territories as guide lines. Technical manuals are drafted and published for use by the Water Works Industry. General progress in providing these services in the urban and rural areas is monitored. External or G.O.I. assistance as required to needy areas is offered for capital investment and implementation of water supply schemes. Certain in service training programmes for the employees of the Water Works Industry in the states are sponsored. Financial assistance for specific inservice training programmes of the states is offered.

14.1.2 STATE GOVERNMENT LEVEL

The State Governments offer to assist the local bodies in planning and implementation of water supply schemes of individual or a group of local bodies. Financial assistance is also given for these local body schemes in the form of Grant-In-Aid (GIA) and loan etc. for capital investment. In certain special circumstances, the State Governments assist the local bodies in operating and maintaining their water supply schemes wholly or upto bulk supply level through its own departments or through the statutory boards of the state governments. Trained engineers and skilled workmen are sometimes deputed to local bodies on request, to plan, implement and operate the water supply systems. The state governments monitor general progress of water supply schemes of local bodies in respect of planning, implementation, operation and maintenance.

14.1.3 LOCAL BODY LEVEL

It is the obligatory responsibility of every local body (municipality, village panchayat etc.) to provide potable water supply to the residents of the area under their respective jurisdictions. Depending upon financial status of each local body, the State/Central Governments come to the help of these local bodies to meet a part/whole of their capital investment cost on water supply augmentation/improvement schemes in the form of GIA, and/or loan. The expenditure on annual operation and maintenance of these schemes has, however, to be met by the local body out of its own revenue to be generated from water charges and water tax. As per the respective acts of local bodies, they have been empowered to levy and recover water charges and tax from the consumer to whom water facility is created by the local body.

14.2 COMMON ASPECTS OF WATER WORKS MANAGEMENT

The aspects considered in this chapter refer to management of operation and maintenance of water supply systems. There are five important aspects of management that could be considered, namely, (i) General Administration, (ii) Personnel Administration, (iii) Inventory Control, (iv) Financial Control and (v) Public Relation. The system has to work as a unit management organisation and as a business enterprise. The management in general should aim at the following achievements.

(a) The quality of water supplied should be safe.

(b) Service to consumers should be satisfactory.

(c) Operations should be safe and self supporting.

(d) Financial management should be sound.

An efficient and effective management of water supply systems is most essential for their proper functioning.

14.3 GENERAL ADMINISTRATION

This could be further sub-divided into two categories, viz; (a) Supervisory and (b) Operational. The operational level is to be subordinate to supervisory level..

The supervisory administration is expected to control all the functions of management. Water works is an engineering service. Hence it is a general practice to set up an Engineering Supervisory Organisation on the considerations of annual work load expenditure to be handled by the organisation. These units are (i) an Engineering Division Unit and (ii) an Engineering Sub-Division Unit. The 1988 norms for establishing these units and the staffing pattern could be considered approx. as shown in Appendix 14.1. Annual work load per unit could be enhanced or reduced depending upon local conditions such as high cost of power consumption or wider area to be covered e.g. for regional schemes etc. The works expenditure considered in the norms is inclusive of all expenses incurred during annual O & M of water works. It is suggested that the functions of sanction to water connections and revenue collection due to sale of water on meter/flat rate/stand post basis should be wholly entrusted to these Engineering Units for better management and control. If the annual work load of water works or a group of water works controlled by one local body/agency is higher than that justified for one Division, then additional divisions together with apex supervisory units such as a Superintending Engineer's Unit/Chief Engineer's Unit, etc. could be established. These Engineering Units would be administratively controlled by the head of engineering department and/or the elected local bodies of the town or village and their Committees.

14.3.1 Duties and Responsibilities

The duties and responsibilities of these supervisory units could be listed as under

(a) To supervise and manage the water works.

(b) To develop annual operation and maintenance (A.O.M.) programme and the budget.

(c) To implement A.O.M. Programme using appropriate planning and scheduling techniques.

(d) To keep accounts, records of the materials and tools, work performance and money spent on work establishment.

(e) Periodically (say monthly/quarterly) inform the owner about the status of O&M programme and budget.

(f) Prepare special reports as required to ensure economical and efficient use of resources.

(g) Schedule, assign and monitor work being done by personnel in the organisation.

(h) Purchase equipment, tools and supplies required to carry out O&M programme.

(i) Provide inservice training.

In addition to the above, it should also look into the following aspects:

(a) That there are adequate maintenance facilities.

(b) That the operations are smooth.

(c) That the maintenance is efficient and economical.

(d) That the administration is efficient and responsive (task assigned to the manager).

(e) That the equipment and supplies are controlled properly.

(f) That good public relations are established.

(g) That appropriate plans for future expansions are drafted.

Some of the additional tasks that these supervisory units are required to handle could be briefly stated as under:

(a) The entire work of O& M could be grouped into logical tasks or functions. Each function may be assigned to a group of workers.

(b) Wherever found necessary and in the interest of work powers could be delegated to subordinates.

(c) The organisation could be flexible in order to enable it to respond to changing work load and work conditions.

(d) Organisation manual and charts could be developed containing (i) Role of Organisation, (ii) job descriptions, (iii) Statements, etc.

(e) O&M schedules could be prepared assigning works to individuals.

(f) Works could be checked to see that these are being done as required/ expected.

(g) O & M manual could be developed to include (i) Description of system. (ii) System operation, (iii) Special items to be considered, (iv) Lubrication and maintenance, and (v) Repairs etc.

(h) Office operations include answering telephone calls, handling correspondence, records, typing letters/statements, standardising work forms for transmission of information etc.

(i) Compilation of statistical information: The task would include (i) Quantity of water pumped/gravitated into system, (ii) Quantity of water billed/ sold to consumers, (iii) Consumer patterns, (iv) Rate of increase in the number of consumers, (v) System losses, (vi) System maps including location of connections, (vii) Delivery capacity of the system at different stages, (viii) Relation of supply to demand in a tabular or charted form.

(j) Number and nature of complaints received

14.3.2 General Administration at Operating Level

The establishment required at operating level of a water works is determined on the basis of physical work output to be expected from each individual. A general guide line for the creation of some of the categories of staff at operating level is indicated in Appendix 14.2. The requirements are expected to vary according to individual circumstances, like topography and geographical locations etc.

For optimum output from each of the operating staff certain modern business principles could be introduced such as:

(a) Unity of Command - Each worker should report to only one person incharge. One person incharge may not have more than 8 to 10 person for direct control.

(b) Each worker must have a clear understanding as to the expectations of the job from him by the supervisory units.

(c) The worker should be given the relevant extract of the operating manual.

(d) Regular work forms should be maintained by each worker and submitted to controlling person incharge.

(e) Service records of each worker should be kept uptodate by supervisory section and all dues paid to him on time.

(f) All possible service facilities should be provided to the operating staff so that he can devote his full attention to work entrusted to him.

(g) Personal grievances of workers should be attended to promptly.

14.3.3 Personnel Administration

The personnel administration can be classified into four categories, namely:

(a) Describing and classifying work by developing job descriptions, establishing qualifications and goals for each position and developing wage and salary structure.

(b) Recruiting and selecting employees by evaluation.

(c) Evaluating the work of the employee by a system of evaluation norms such as confidential reports etc. The tasks should be identified and achievements mentioned against each task. General assessment made on these basis and report prepared. The evaluation may refer to (i) Knowledge, (ii) Punctuality, (iii) Quality of work, (iv) Dependability, (v) Initiative, and (vi) Tolerance of criticism.

(d) Inservice training of employee (described separately hereafter).

14.4 INVENTORY CONTROL

Inventory control is the process of managing supplies required for day-to-day O&M of water works. It involves (a) deciding what supplies to be stocked, (b) keeping a record of supplies and their locations, and (c) accounting for all receipts and issues of supplies.

Many of the water works failures require spare parts or supplies available instantly to put the system back in working condition. These supplies have got to be ready at hand any time the failure occurs and repairs are to be carried out. Materials of stock would pertain to items which have frequent usage and items of emergency repairs.

Inventory control cards are vital documents to serve the purpose of accountability and stock demand by reflecting usage pattern. They enable stock control and record purchasing information.

Inventory control would include tools required for O& M of the system, although new purchases for these may not be as frequent as for stock materials for repairs and replacement. Requirements have to be checked at intervals.

14.5 ACCOUNTING & BUDGETING

Accounting is the process of recording and summarising business transactions that affect the financial status of the O & M oganisation of the water works. It is an important tool for monitoring revenue and expenditure activities and for interpreting the financial results of the organisation.

Budgeting is the art of interpreting the goal of O & M organisation in meaningful monetary terms. It should be used to control the financial activities of the organisation.

Accounting system would involve the following functions.

(a) A basic chart of accounts for the organisation.

(b) Accounting reports such as income and expenditure statements, balance sheets, cash flow statements and debt servicing, etc.

(c) Annual O & M budget.

(d) A frequent review say quarterly, of income analysis from customer class is desirable.

This would enable the supervisory unit and the authorities of the water works to decide at what level, a review of water tax structure is called for. It would also review ways and means of effecting recovery of outstanding dues from consumers. Legal powers of the authorities to effect full arrears recovery from consumers may have also to be examined periodically and enhanced if required by legislation. A review of expenditure pattern on the basis of revenue realised could also be simultaneously done.

It would be desirable to keep financial records of the system to include:

(a) Updated valuation of the system.

(b) Depreciation.

(c) Operating expenses.

(d) Investments in new capital improvements.

(e) Long term debts, their servicing,

(f) Appropriate schedules of water rates.

Development and implementation of appropriate water rates would go a long way in helping to generate adequate annual revenues of the water works.

14.6 INSERVICE TRAINING

The object of well founded short term in-service training for the employees of water works undertaking is:

(a) To improve group level of operational efficiency.

(b) To acquaint the group with the new developments.

(c) To develop amongst the members of the group a better understanding of human relations and concept of their individual responsibility to the community.

(d) To bring about and increase community awareness of water works operation.

The training could include:

(a) Orientation courses to describe duties and responsibilities of individuals in the organisation.

(b) Providing an employee with a handbook.

(c) On the job training to work with experienced employee for some time.

(d) Work shops, short courses and seminars on concerned subjects.

The subjects to be included in the training could be :

(a) How to perform a number of specific jobs well.

(b) Lectures on practical aspects of subjects covered under O&M of water works,

(c) Laboratory control tests.

(d) Physical, chemical and bacteriological examination of water and interpretation of results.

(e) Disinfection.

(f) Design of component works of scheme.

(g) Supervisory control.

(h) Systems management and Administration.

(i) Accounting, budgeting and financial management.

Each one of the supervisory and operating staff on the water works should be subjected to appropriate training course depending upon work to be handled by him atleast once in three to five years of his service period.

14.7 LONGTERM PLANNIING

One of the important functions of a water works management is to develop technical and financial plans for future expansion of the water works. For this purpose, the management should review periodically, present adequacy and future requirements. Some of the aspects to be reviewed could be:

(a) Analyze the ability of the system to deliver water of acceptable quality, adequate quantity and under sufficient pressure at times of max. demand.

(b) Forecast future requirements, determine the areas and population to be served and the future likely consumption.

(c) Co-ordinating construction and financing.

It is much better to keep up and improve the system through small construction programmes undertaken yearly than to allow deficiencies to accumulate. The yearly improvement should be planned to fit in with the prospective objectives and requirements.

(d) The planning for future expansions require knowledge of original designs and basis for present water system.

There is no harm for the local bodies in soliciting assistance from external agencies such as Governments and consultants for development of future plans and implementation programmes as required.

14.8 PUBLIC RELATION

The object of public relations is to develop:

(a) Consumer satisfaction

(b) Opportunity for the community to know how works are managed.

(c) Frequent dialogue between the community, owner and management.

(d) Art of keeping owners informed about day to day working of the system, shortfalls, if any, and assistance required.

(e) Interpretation of articles in the news papers about O&M situation, deficiencies, deviations, etc., based on facts and figures.

Sufficient publicity needs to be given to O&M work being done by the management, difficulties experienced and cooperation required from public to make good the deficiencies, if any. Information could be given in news papers. Appropriate talks could be given on T.V., A.I.R. etc. All criticism in the press about O&M of the system could be promptly attended to and appropriate replies published, preferably in the same news papers in which criticism appeared.

In addition to the above activities, publicity of O&M work is automatically enhanced if;

(a) Every employee of the management who makes public contacts adopts a helpful and courteous attitude towards consumers and public.

(b) Personal interest is shown in consumer's complaints and problems and these are dealt with promptly with courtesy and commonsense.

(c) Consumers are encouraged to visit water works which should be kept clean, tidy and in good repairs.

(d) Good relations are established with local press by providing fullest possible information on the O&M of water works.

(e) Contacts are established with benevolent, social, health and educational bodies.

(f) Small pamphlets on water works are periodically published and distributed.

CHAPTER 15
LABORATORY TESTS AND PROCEDURES

15.1 GENERAL

Laboratories with adequate facilities and manned by qualified personnel are essential for inspection and evaluation of the suitability of water supplies for public use as well as for controlling the water treatment processes. The ultimate aim of laboratory examination of water is to ensure that potable water conforming to the drinking water standards is supplied to the consumers.

Tests carried out in the laboratory are intended to assess the quality and classify the raw water to be treated; to determine the need and extent of treatment; to check that water has been properly prepared for each phase of treatment process; to ensure that each phase of treatment proceeds according to plan and to examine the finished water to ascertain that it conforms to the standard. Other objectives that could be served by a regular testing programme include: (i) determination of trends in drinking water quality over time, (ii) provision of information to public health authorities for general public health protection purpose and (iii) identification of sources of contamination.

Laboratory facilities are thus indispensable for controlling plant operation and to record and improve plant performance which help research and development.

15.2 TYPES OF EXAMINATIONS

The laboratory examination comprises of physical, chemical, bacteriological and biological analyses.

Physical analysis determines the aesthetic quality and assess the performance of various treatment units.

Chemical analysis determines concentrations of chemical substances which may affect the quality of water and be indicative of pollution and which reflect variations due to treatment – a requirement for control of water treatment processes.

Bacteriological examination indicates the presence of bacteria characteristic of pollution and hence the safety of water for consumption.

Biological examinations will find application in providing information on causes of objectionable tastes and odours in water or clogging of filters and dictating remedial measures.

15.3 SAMPLING

The value of any laboratory analysis and test depends upon the method of sampling. Failure to observe proper precautions in securing a representative sample may result in an analysis which is of little use since it may unnecessarily condemn a good water supply or more frequently it may certify a bad water as satisfactory. Physical, chemical and bacteriological analysis are necessary for drinking water, while physical analysis may be adequate for industrial waters excepting in food or beverage industries. Biological analysis will be required for limnological work or where taste and odour problems are encountered.

All samples of water should be properly labelled and should be accompanied by complete and accurate identifying and descriptive data. Data should include date and time of collection, type of source of the sample and temperature of water at the time of collection. When samples are being collected from the same sampling point for different analysis, it is essential that the sample for bacteriological examinations be taken first. The particulars to be supplied with the sample are enumerated in the Appendixes 15.2, 15.3, 15.4, 15.5, 15.6 and 15.7.

For transport, bottles may be packed in wooden, metal, plastic or heavy fibreboard cases, with a separate compartment for each bottle. Boxes may be lined with corrugated fibre paper, felt or other resilient material or may be provided with spring-loaded corner strip to prevent breakage. Polythene bottles do not need such elaborate care.

15.3.1 SAMPLING FOR PHYSICAL AND CHEMICAL ANALYSIS

Samples should be collected in containers of Pyrex glass or other inert material like polythene.

Sample bottles must be carefully cleaned before use. Glass bottles may be rinsed with a chromic acid cleaning mixture by adding one litre of concentrated sulphuric acid slowly with stirring to 35 ml saturated sodium dichromate solution or with an alkaline permanganate solution followed by an oxalic acid solution. After having been cleaned, bottles must be rinsed thoroughly with tap water and then with distilled water.

About 2.5 litres of the sample is required for analysis. Prior to filling, the sample bottle should be rinsed out two or three times with water to be collected. Care should be taken to obtain a sample that is truly representative of existing conditions and to handle it in such a way that it does not deteriorate or become contaminated before it reaches the laboratory.

The sample should reach the place of analysis as quickly as possible within 72 hours of collection. The time elapsed between collection and analysis should be recorded in the laboratory report.

Some determinations are likely to be affected by storage of samples. Walls of glass containers are likely to absorb cations like aluminium, cadmium, chromium, copper, iron, lead, manganese, silver or zinc which are best collected in a separate bottle and acidified by concentrated hydrochloric or nitric acid to a pH approximately 3.5 to minimise precipitation and adsorption on the walls of the container.

Certain parameters like temperature, pH dissolved gases like carbon dioxide, hydrogen sulphide, chlorine and oxygen may change significantly during transport. For this reason, determinations of pH, carbon dioxide, ferrous iron, dissolved oxygen and chlorine should be carried out on the spot. Hydrogen sulphide can be preserved by fixing it with zinc acetate until the sample is ready for analysis.

Hot samples collected under pressure should be cooled while under pressure. Sample from wells should be collected only after the well has been pumped for a sufficient time to ensure that the sample will be representative of the ground water.

15.3.2 SAMPLING FOR BACTERIOLOGICAL ANALYSIS

15.3.2.1 Sampling Bottles

Sterilized glass bottles provided with ground glass stopper having an overlapping rim should be used. The stopper and the neck of the bottle should be protected by brown paper. The sterilization is carried out in an autoclave at 1 kg/cm^2 pressure for 15 minutes or by dry heat at 160^0C for 1 hr.

15.3.2.2 Dechlorination

Dechlorination is necessary for chlorinated water samples. For this, sodium thiosulphate should be added to the clean, dry sampling bottles before sterilization in an amount to provide an approximate concentration of 100 mg/l in the sample. This can be done by adding 0.2 ml of 10% thiosulphate solution to a 250 ml bottle and the bottle is then sterilized.

15.3.2.3 Sample Collection

The sample should be representative of the water to be tested and they should be collected with utmost care to ensure that no contamination occurs at the time of collection or prior to examination. The sample bottle should not be opened till the time of filling. The stopper with the cap should be removed with care to eliminate soiling. During sampling, the stopper and the neck of the bottle should not be touched by hand and they should be protected from contamination. The bottle should be held near the base, filled without rinsing and the stopper replaced immediately. The bottle should not be filled completely but sufficient air space left for shaking before analysis. Then the brown paper wrapping should be tied to protect the sample from contamination.

(a) Sampling from Taps

The tap should be opened fully and the water allowed to run to waste for two to three minutes or for a sufficient time to permit clearing of the service line. The flow from the tap should then be restricted to permit filling the bottle without splashing. Leaking taps, which allow water to flow over the outer surface of the bottle, must be avoided as sampling points. If it becomes necessary to collect from that point, the leak should be attended to before sampling. When a tap is not in continuous service, it is advisable to wipe the tap free of any grease or preferably flamed before collection of the sample. It should be ascertained whether the tap from where the sample is collected is supplying water from a service pipe directly connected with the main or with a cistern or a storage tank. This information should be sent along with sample.

(b) Sampling Direct from a Source

When the sample is to be collected directly from a stream, river, lake, reservoir, spring or a shallow well, it should be representative of the water that will be taken for treatment. Hence, a sample should not be taken from a point which is too near the bank or too far from the point of draw-off or at a depth above or below the point of draw-off. Areas of relative stagnation in a stream should be avoided.

Sample from a river, stream, lake, or a reservoir can often be taken by holding the bottle in the hand near its base and plunging its neck downward, below the surface. The bottle should then be turned until the neck points slightly upward, the mouth being directed against the current. If no current exists, as in a reservoir, a current should be artificially created by pushing the bottle horizontally forward in a direction away from the hand. If it is not possible to collect samples from this situation, in this way, a weight may be attached to the base of the bottle which can then be lowered into the water. In any case, damage to the bank must be guarded against, as otherwise fouling of the water can occur. Special apparatus which permits mechanical removal of the stopper of the bottle below the surface is required to collect samples from the depths of a lake or a reservoir. If the sample is to be taken from a well, fitted with a hand-pump, water should be pumped to waste for four to five minutes before the sample is collected. If the well is fitted with a mechanical pump, the sample should be collected from a tap on the discharge end. If there is no pumping machinery, the sample can be collected directly from the well by means of a sterilized bottle attached with a weight at the base. In this case, care should be taken to avoid contamination of the sample by any surface scum. Where it is not possible to collect the sample directly into the bottles as for example where there is a high bank, the sample may be obtained by means of suitable metal jug. The jug is sterilized by pouring into it 3 to 5 ml of methylated spirit and tilting the jug in such a way that the spirit comes in contact with the entire inner surface of the jug and igniting. The jug should be lowered to the required depth and then drawn up and down two or three times before it is brought to the surface. It should be rinsed out atleast twice before the sample is taken. Should the jug come in contact with the bottom or skid along the surface so that it may have collected the surface film, the sample should be discarded, the jug resterilized and another sample drawn. The water from the jug should be poured into the bottle and the glass stopper of the bottle be replaced, care being taken to avoid the cover being caught between the stopper and the neck of the bottle.

15.3.2.4 Size Of The Sample

The volume of the sample should be sufficient for carrying out all the tests required and in no case, it should be less than 250 ml.

15.3.2.5 Preservation And Storage

Water samples should be examined immediately after collection. However, this is seldom practical and hence it is recommended that the samples should be preferably analyzed within one hour after collection and in no case this time should exceed 24 hours. During transit, the temperature of the sample should be maintained as close as possible to that of the source of the sample, at the time of sampling. The time and temperature of storage of all samples should be recorded since they will be considered in the interpretation of the laboratory results. If they can not be analyzed within 24 hours, the samples must be preserved in ice until analysis. No sample is fit for bacteriological analysis after 72 hours.

15.3.3 SAMPLING FOR BIOLOGICAL ANALYSIS

For this purpose, two samples should be collected in clean two litre wide mouthed bottles with a glass stopper or a bakelite screw cap.

In making this collection, the bottle, after the stopper is removed, is thrust as far as possible mouth downward into the water. It is then inverted and allowed to fill.

One bottle is to be stoppered as such. To another bottle, add 5 ml of commercial formalin for every 100 ml of water sample immediately after collection. Both the bottles would be despatched with the label on the sample stating the one with formalin.

If two litres of samples could not be collected, even 200 ml of the sample may be collected as above and formalin added to one sample (10 ml of formalin added to 200 ml of water.)

15.3.4 FREQUENCY OF SAMPLING

The frequency of collection of samples for chemical analysis depends on the variability of the quality of tested water, the types of treatment processes used and other local factors.

Samples for general systematic chemical examination should be collected atleast once every three months in supplies serving more than 50,000 inhabitants and atleast twice a year on supplies upto 50,000 inhabitants. More frequent sampling for chemical examination may be required for the control of water treatment processes.

It is necessary to collect samples of both raw and treated water for examination of toxic substances atleast every three months and more frequently when subtolerance levels of toxic substances are known to be generally present in the source of supply or where such potential pollution exists.

For bacteriological sampling, which controls the safety of supply to the consumer, the frequency of sampling and the location of sampling points at pumping stations, treatment plants, reservoirs and booster pumping stations, as well as the distribution system, should be such as to enable a proper evaluation of the bacteriological quality of the entire water supply.

The minimum number of samples to be collected from a distribution system should be as prescribed in Table 15.1.

The samples should be taken from the different points on each occasion to enable overall assessment.

In the event of an epidemic or immediate danger of pollution, it should be borne in mind that much more frequent bacteriological examination will be required than the recommended minimum frequencies for routine bacteriological examination.

For biological examinations, where seasonal growth of plankton are known to be a regular occurrence, samples may need to be taken at weekly or even shorter intervals, in order to determine the type of treatment. During treatment operations, samples for examination would need to be taken at short intervals, probably daily. When growth of plankton is not anticipated, samples should be drawn on a monthly or less frequent basis. Greater frequencies, determined by experience may be needed in tracing possible entrance of pollution into water sources or more particularly into distribution systems.

TABLE 15.1

MINIMUM SAMPLING FRE-QUENCY AND NUMBERS FROM DISTRIBUTION SYSTEM

Population Served	Maximum Intervals between successive sampling	Minimum No. of samples to be taken from entire distribution system
Upto 20,000	One month	One sample per 5,000 of population per month
20,000-50,000	Two weeks	
50,001-100,000	Four days	
More than 100,000	One day	One sample per 10,000 of population per month.

15.4 STANDARD TESTS

The standard tests that are employed in the analysis of water are as follows:

15.4.1 Physical Examination

The parameters tested are temperature turbidity, colour, taste and odour,

15.4.2 Chemical Examination

(a) This includes tests for consistency and characteristics of water that affect the health of the consumers and the potability of water, viz. pH, acidity, alkalinity, hardness, calcium, magnesium, iron, manganese, copper, zinc, aluminium, sulphates, fluorides, chlorides, nitrates, total dissolved, and suspended solids.

(b) Tests for efficacy of treatment, viz., chlorine demand, free and combined residual chlorine, coagulant dosage.

(c) Tests for chemical parameters which are indicators of pollution such as total nitrogen and nitrogen in various forms like ammonia, nitrite and nitrate, phosphate, dissolved oxygen and BOD.

(d) Tests for toxic chemical substances-lead, arsenic, mercury, selenium, chromium, cyanide, phenolics, pesticides and hydrocarbons and

(e) Test for radio-activity

15.4.3 BACTERIOLOGICAL EXAMINATION

Microscopic tests for identification and enumeration of microorganisms other than bacteria are included in this category.

15.4.4 SCHEDULE OF TESTS

The schedule of laboratory tests followed by a particular undertaking will vary with the size of the plant and character of water treated, though for ordinary plants the minimum schedule should include turbidity, colour, alkalinity, pH, hardness, residual chlorine, bacterial count at 37^0C and coilform bacterial numbers, both presumptive and confirmed.

Occasionally special tests may be necessary such as residual alum, iron and manganese, taste and colour and other undesirable constituents of finished water. Where prechlorination is practised, residual chlorine should be tested at each major stage of treatment. Chlorine demand tests should be carried out in raw water.

15.5 METHODS OF EXAMINATION

The physical, chemical, bacteriological and biological procedures for the analytical laboratory examinations given in the Manual of methods for the Examination of Water, Sewage and industrial Wastes published by the Indian Council of Medical Research, are to be followed. For procedures regarding trace and other elements not covered by the ICMR, the procedures recommended in Standard Methods for the Examination of Water and Waste water prepared and published by American Public Health Association, American Water Works Association and Water Pollution Control Fedration are to be followed.

Conformity to standard analytical methods is important if the results of tests carried out by different laboratories are to be meaningful.

15.5.1 REPORTING OF RESULTS

Specimen forms for reporting results of a short chemical examination, a complete chemical examination and bacteriological examination of water are given in Appendices 15.5 and 15.6. For purposes of uniformity, standard expressions should be used and this should be clearly stated in the report.

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15.6 LABORATORY EQUIPMENT AND FACILITIES

A well equipped laboratory is a prerequisite for efficient analytical control. The size and equipment of the laboratory depends more upon the nature of the processes to be controlled and to a lesser extent on the size of the plant. The laboratory could be divided into several units; viz. a physical and chemical laboratory, a bacteriological laboratory, a biological laboratory, a preparation room and a store. For a small plant, the various units could be combined into one laboratory.

15.6.1 RECOMMENDED MINIMUM TESTS AND EQUIPMENT

It is necessary that all waterworks should be provided with the equipment and facilities for tests mentioned in Appendices 15.7, 15.8 and 15.9.

The lists provide for the categories of water works:

Category I is applicable to all State laboratories and for large water works with an output greater than 7.5 mld or serving a population larger than 100,000 and dealing with polluted surface water and practising coagulation, filtration and post chlorination. Such laboratories would be equipped for conducting complete chemical, bacteriological and biological tests. It is expected that such laboratories may also undertake simple research problems, stream sanitation studies and other investigations and will assist the smaller laboratories in their vicinity by supplying standard solutions and providing guidance.

Category II is applicable to the water works with an out put upto 7.5 mld or serving a population upto 100,000 and when the water is coagulated, filtered and chlorinated. The tests laid down are routine chemical and bacteriological examinations only. Thus, the bacteriological procedure would consist of the presumptive test for coilform organisms followed by confirmation (in the case of finished water only) by the use of liquid broth. Isolations on solid media and differentiation of coliforms into feacal and non feacal forms, if is felt necessary, may be carried out by sending samples to a suitable laboratory with this facility.

Category III is applicable to all other water works, mostly with the only treatment of storage or settlement followed by chlorination. They should be equipped for routine chemical tests included under category II. The bacteriological examinations necessary may be undertaken by the nearest available large laboratory.

The expression, parts per million (ppm), still used to express chemical concentrations, should be replaced by milligrams per litre (mg/l), which is much more appropriate, unless there is a special need to use some other chemical concentration unit like 'millequivalents per litre' (me/l) or microgram per litre (µ/l). The expression me/l facilitates the summation of several anions or cations responsible for imparting a particular characteristic to the water like hardness.

Volumes are expressed in milliliters (ml) and temperatures in degree centigrade (0^0C). The total number of micro-organisms developing on solid media should be given in significant numbers per ml of water, the medium, time and temperature of incubation being stated. The number of coliform organisms and other organisms indicative of pollution should be expressed in terms of "Most Probable Number" (MPN) per 100 ml or as a determined number obtained by direct plating procedures. In biological examinations, the concentration of organisms per ml of sample is expressed in many instances as a simple numerical count. Occasionally the results are expressed in mg/l , but more usually in terms of area standard units or volumetric standard units.

Reporting analytical results of a particular examination should include the proper use of significant digits and the expression of confidence limits, where appropriate.

15.6.2 FACILITIES

The working benches should be of suitable height (o.75 to 1 m) with acid resistant tops. Adequate gas, electric power and water points must be provided along the benches and services for gas, electricity and water can be fitted against the walls, under the bench work, as much cupboard space as possible should be built-in, finishing flush to the bench work, thus providing unobstructed floor space throughout. There should be ample sinks and drain lines. The analytical work in the laboratory requires provision of ample window space and fluorescent artificial lighting. A minimum area of 150 m^2 is required for category I laboratories, a minimum of 50 m^2 being sufficient for other categories.

15.6.3 EQUIPMENT

The equipment in the laboratory must be adequate to permit proper analytical laboratory control of purification processes. Careful planning is necessary while equipping the laboratory to effect proper utilization of the equipment. Proper maintenance of equipment and storage of chemicals must be in the hands of responsible analysts. A needbased planning to acquire consumable materials like glassware, chemicals and reagent is in general more important than the procurement of various special equipments. Calibrated instruments should frequently be checked using standards.

15.7 RECORDS

A continuing programme of examination of water and controlling its quality to determine its conformity with established water quality standards calls for proper maintenance of accurate and complete records. These records are essential for a review of the working of the plant and also for adequate and intelligent operation of water works processes and for laboratory activities.

All details of actual specific determinations, burette readings, weights and calculations should be recorded. These information should, of course, remain as laboratory records and only the final result should be reported. This makes all laboratory data available at any time for review any important factor when unusual findings are called to attention.

Depending upon the specific needs of the laboratory forms and cards could be designed providing spaces for entering the data and for calculations. Monthly reports may be in single sheets and annual reports may be furnished in two sheets grouping physical and chemical data on one sheet and bacteriological and biological data on the other.

Representation of data collected over a period of time by means of charts and graphs makes it an easy and useful study for the staff and visitors.

15.8 LABORATORY PERSONNEL

Laboratory personnel must be qualified and suitably trained in laboratory control. Water analysts with sufficient experience in treatments and quality control may be kept in charge of the laboratory.

The minimum staff required for water works laboratories is given in Appendix 15.1. The recommended minimum staff required for water works laboratories for groundwater source is presented in Appendices 13.6 and 13.7

CHAPTER 16
COMPUTER AIDED OPTIMAL DESIGN OF WATER TREATMENT SYSTEM

16.1 GENERAL

The unit processes in conventional water treatment include coagulation-flocculation, gravity separation, sand filtration and disinfection. The individual units in the treatment train are usually designed based on the norms recommended in the Manual. These design, when implemented, may give satisfactory level of performance but not necessarily be optimal, both functionally and costwise, The performance of each treatment unit affects the efficiency of the subsequent units. However, decisions are often made with no regard to the interacting nature of the various unit operations and the treatment systems are designed on individual unit basis. This is largely due to the non-availability of appropriate Operations Research (OR) tools for total system analysis to enable development of designs which will produce potable water of specified quality at minimum cost. This chapter presents an approach to the computer aided functional and minimal cost design of water treatment systems.

16.2 DYNAMIC PROGRAMMING

16.2.1 CONCEPT

A conventional water treatment system shown schematically in Fig. 16.1 favours application of Dynamic Programming (DP) for minimal cost design and it is a technique useful in solving sequential decision problems, each decision influencing the subsequent decision (s). The main advantage of dynamic programming is the reduction of efforts required to find optimum. Dynamic Programming is a simple procedure from computational point of view, and one which can treat non-convex non-linear, discontinuous objective and constraint functions. Since it is an iterative procedure, a relatively small number of computer instructions is required. Further constraints imposed on system reduce the number of feasible solutions and therefore time required to establish the optimal policy. Other advantages of DP are with respect to availability of feasible solutions with costs and hence selection of most acceptable optimal solution based on site conditions.

Dynamic Programming can generally be applied to any system with multi-decision problems. The system is broken into stages. The stages may be unit processes with inter relationships and each stage having only a few variables. In such an analysis, each stage is characterized in terms of four factors as depicted in Fig. 16.2.

1. The input state 'Sn' which depends on decisions made in the previous stages and/or on fixed external conditions.

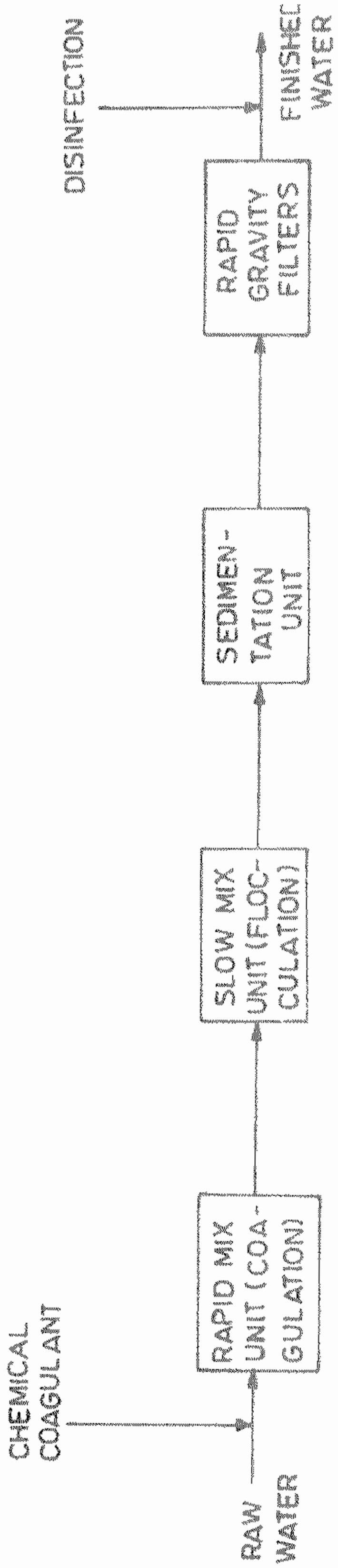


FIGURE 16.1: CONVENTIONAL WATER TREATMENT SYSTEM-SCHEMATIC

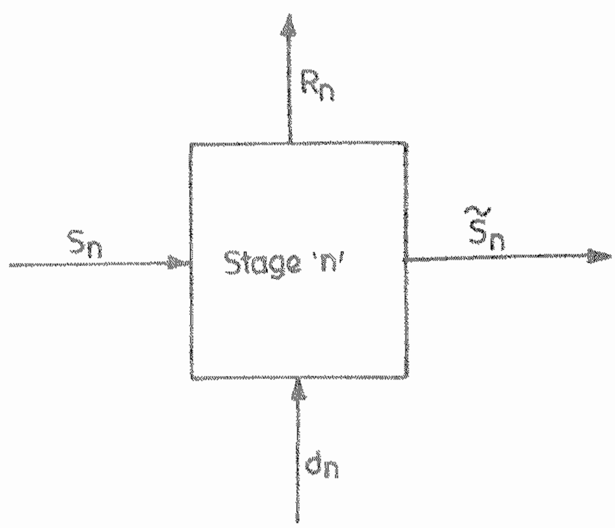


FIGURE 16.2: FUNCTIONAL DIAGRAM OF DYNAMIC PROGRAMMING

2. Decision 'dn' which fixes the design and/or operating conditions of the stage.

3. The output state '$\overline{Sn}$' which depends on Sn and dn, i.e., '$\overline{Sn}$' = ϕn (Sn,dn), and

4. The stage return, 'Rn' which is dependent on Sn, dn and '$\overline{Sn}$' i.e., Rn = gn (Sn, dn, '$\overline{Sn}$')

16.3 APPLICATION TO WATER TREATMENT SYSTEM DESIGN

Water treatment system can be considered as a multistage process with stages represented by various unit processes, viz. coagulation-flocculation, sedimentation, and rapid gravity filtration; and states represented by the levels of water quality parameters like turbidity or suspended solids. The decision variables would be the design parameters depending on the type of unit process, i.e. stage. The information flow diagram showing the sequence of stages; input, decision and output variables for each stage and the stage return for each input-decision combination is shown in Fig. 16.3. The range of design variables, for conventional treatment units, as summarised from the earlier chapters of the Manual are given in Table 16.1.

In order to optimize the system logically and methodically, a thorough knowledge of the major variables of all the unit processes and their influence on performance and cost of the subsequent units is necessary. Further, information on the process models relating the design variables to the behaviour of the systems and cost models for the individual treatment units is essential. These enable the formulation of an objective function and constraints enabling solution of optimization problems through Dynamic Programming.

16.4 PERFORMANCE MODELS

16.4.1 RAPID MIX UNIT

The rapid mix unit is an an adjunct to the flocculator and hence is not modelled separately for its functionality and should be designed as per the norms recommended in the Manual. Its performance is expected to be satisfactory when the appropriate coagulant dose is applied.

TABLE 16.1
RANGE OF MAJOR DESIGN VARIABLES FOR WATER TREATMENT PLANTS

Sl. No.	System Component	Design Variable	Range
1	Rapid Mix Unit	Velocity Gradient	300 to 900 Sec^{-1}
		Detention Time	20 to 60 Sec
2.	Slow Mix Unit	Velocity Gradient	10 to 75 Sec^{-1}
		Detention Time	10 to 40 min
3.	Sedimentation Unit	Surface Overflow Rate	1.25 to 1.66 $m.hr^{-1}$
4.	Rapid Sand Filter	Filtration Rate	4.8 to 6.0 $m.hr^{-1}$

Assuming that SS_4 is the raw suspended solids concentration, pH and alkalinity of raw water are within the desirable range for effective coagulation and that all Al(III) is precipitated as Al $(OH)_3$, the mass balance, then leads to:

$$SS_4 + KA = '\overline{SS_4}' \tag{16.1}$$

Where KA = suspended solids in mg/l due to the addition of A mg/1 of coagulant;

(K = 0.247 for $Al_2(SO)_3$. 16 H_2O based on stoichiometry)

$'\overline{SS_4}'$ = suspended solids concentration in mg/1 in the effluent from rapid mix unit.

It is generally accepted that the principle design parameters of rapid mix are velocity gradient (Gr) and duration of mixing (Tr), although chemical factors such as pH and alkalinity of water to be treated also influence the process of coagulation-flocculation. The intensity of agitation is expressed in terms of power input or the velocity gradient. The value of G_r. T_r, has been assumed as 1. 8 x 10^4.

16.4.2 Slow Mix (Flocculation Unit)

The flocculation should be designed to generate particle aggregates such that the settleability and filterability of the suspension are improved. The important attributes in the settling are the floc size, density and viscosity of water. The effluent from the rapid mix unit is the influent to the flocculation unit. Assuming that no settlement of floc particles occurs in the flocculation basin, then the concentration of suspended solids in the effluent would remain unchanged.

Hence,

$$'\overline{SS_4}' = SS_3 = '\overline{SS_3}' \tag{16.2}$$

Where SS_3 = suspended solids concentration in the influent to the flocculation unit.

$'\overline{SS_3}'$ = suspended solids concentration in the effluent from the flocculation unit.

Although the mass of suspended solids remains unchanged in the process of flocculation, the size of floc particles is increased due to interparticle contact brought about by the applied velocity gradient. The size of floc aggregates thus formed is related to the velocity gradient as under:

$$d = a\ G^b \tag{16.3}$$

Where, d = volume average diameter of the floc, mm

G = applied velocity gradient, S^{-1}

'a' is a constant and 'b' is an exponent, the values of which can be determined experimentally.

For this recommended range of velocity gradient, the following relationship has been developed for alum floc.

$$d = 26.88\ G^{-0.91} \tag{16.4}$$

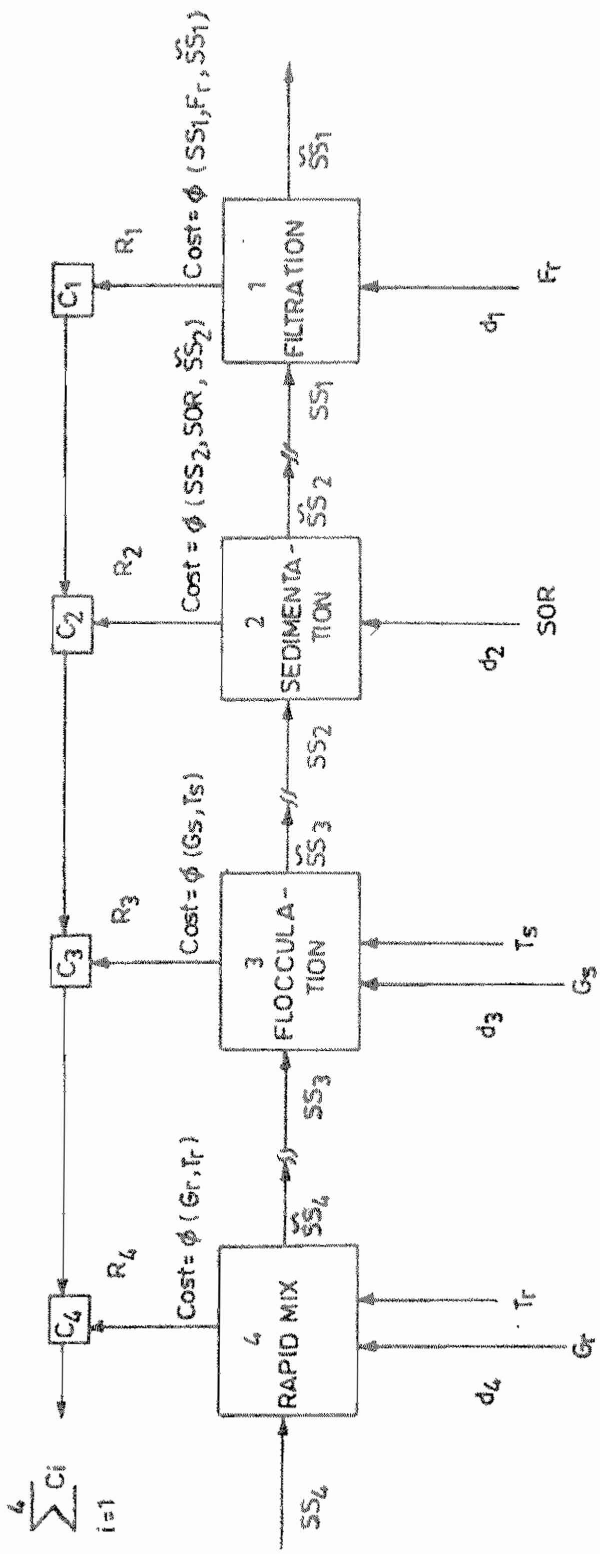


FIGURE 16.3: INFORMATION FLOW DIAGRAM FOR WATER TREATMENT PROCESS

In the derivation of the above relationship, it is assumed that the velocity gradient is uniform and constant in the flocculation unit.

The currently accepted criteria for design of slow mix unit are based on the concept that performance is a linear function of "Gs" and "Ts" which are independent and hence their dimensionless product "Gs.Ts" is regarded as the most significant parameter. The value of Gs . Ts in the range 2 to 4 x 10^4 has been assumed.

16.4.3 SEDIMENTATION UNIT

Efficiency of settling is primarily governed by the size and density of floc particles and the settling basins are designed on the basis of surface overflow rate which is related to the settling velocity of the suspended particles. It is presumed that floc aggregation is complete in the flocculator itself and that any further agglomeration during settling is insignificant. For Reynolds number less than 1, the settling velocity can be estimated using well known Stock's equation given below:

$$V_s = \frac{g}{18}(S_s - 1)\frac{d^2}{\nu} \tag{16.5}$$

Where,

V_s = settling velocity of floc particles, m/s

g = acceleration due to gravity, m/s^2

S_s = specific gravity of floc particles

d = volume average diameter of floc, m

ν = Kinetic Viscosity, m^2/s

For floc particles of size larger than 1mm, the Reynold's number exceeds 1 and the Stoke's law is not applicable. In such cases and for Reynold's number upto 50, the settling velocity can be estimated using Hazen's Equation:

$$V_s = \frac{g^{0.8}}{10}(S_s - 1)^{0.8}\frac{d^{1.4}}{\upsilon^{0.6}} \tag{16.6}$$

For an ideal sedimentation tank, surface overflow rate represents the settling velocity of particles which are removed 100 percent. However, in practice, the efficiency of the basin is reduced due to various factors such as currents induced by inertia of the incoming water, turbulent flow, wind, density and temperature gradients, etc. which result in short circuiting of the flow. Mathematically the efficiency of suspended particles removal is expressed as:

$$\frac{SS_2 - \overline{SS_2}}{SS_2} = 1 - \left[1 + n\frac{V_s}{\left(Q/A\right)}\right]^{-1/n} \tag{16.7}$$

where,

$\frac{SS_2 - \overline{SS_2}}{SS_2}$ = suspended solids removal efficiency

n = coefficient that identifies basin performance

V_s = surface overflow rate for ideal basin

Q/A = required surface overflow rate to achieve the desired efficiency.

the value of n is assumed '0' for best possible performance, '1/8' for very good performance, '1/4' for good performance, '1/2' for poor performance and '1' for very poor performance.

A well designed sedimentation basin, irrespective of the influent suspension concentration, should produce a settled water of turbidity less than 20 NTU or suspended solids less than 50 mg/l.

16.4.4 Rapid Sand Filtration

Filtration is an important step in the solids removal chain. Mathematical formulations presented by most of the researchers for predicting filter performance have limitation. In the development of models, some idealised assumptions are made with regard to the nature of suspension which often deviate significantly from real life situations. Also these models do not eliminate the need for some empirical constants. A method has been proposed for prediction of filter performance and demonstrated its usefulness for a variety of suspensions using different chemical coagulants and filter media. It was observed that the removal of particles per unit depth through a filter bed is quite similar to the Chi-square probability distribution. The variate 'U' of this distribution is considered a measure of the clogging process and is related to the filtration data as follows:

The ratio of concentration at any time 't' and sand depth 'L' to the influent concentration is equated to the cumulative probability 'P,' in the Chi-square distribution, i.e.

$$\frac{SS_1 - \overline{SS_1}}{SS_1} = P_c \qquad (16.8)$$

The filtration time 't' in hours is equated to the degrees of freedom 'v', i.e. 1 hr = 1 degree of freedom.

The variables such as filtration rate, diameter of sand grain and the filter run time are grouped into a single term 'G' as under:

$$G = 0.725\, F_r^{\,0.29}\, d^{\,0.62}\, t \qquad (16.9)$$

Where,

F_r = filtration rate, m/hr

d = 0.5 (ES) (1 + UC), mm

t = filter run time, hrs.

Similarly the variables headloss at time 't', sand size, filtration rate and the filter influent suspension concentration have been grouped in to a single term 'R' :

$$R = \frac{4.55 d^{2.5} H}{F_r^{\,1.2} SS_1} \tag{16.10}$$

Where,

H = increase in headloss at the end of `t`, m

SS_1 = influent suspension concentration , mg/1

From the above two group terms, the performance prediction models developed as under:

$$\log\left(\frac{U}{13.3L}\right) = -0.208 + 1.950\log\left[\frac{G}{(13.3L)^{1.2}}\right] - 0.645\left[\log\frac{G}{(13.3L)^{1.2}}\right]^2 \tag{16.11}$$

$$\log\left(\frac{R}{(13.3L)^{1.6}}\right) = -3.250 + 1.013\log\left[\frac{G}{(13.3L)^{1.2}}\right] - 0.036\left[\log\frac{G}{(13.3L)^{1.2}}\right]^2 \tag{16.12}$$

From the value of variable 'U' obtained from above relationships, the probability `$P_C$` (which can be expressed as removal efficiency $\left(\frac{SS_1 - \overline{SS_1}}{SS_1}\right) = P_c$ could be read from the cumulative table of Chi-square distribution or computed mathematically using the expression:

$$P_C = \sum_{J=0}^{(t/2)-1} \frac{e(-U/2)(U/2)^J}{LJ} \tag{16.13}$$

Using the above functional relationships, the filter performance can be predicted for various combinations of influent suspended solids concentrations (SS_1), size of filter sand (d), filtration rate (Fr), depth of filter bed (L), length of filter run (t), filtrate quality $\overline{SS_1}$ and headloss (H). A properly designed rapid sand filter should be capable of producing a filtered water with turbidity of less than 1 NTU or SS concentration less than 2 mg/l.

16.4.5 DISINFECTION

Treatment processes such as coagulation flocculation, sedimentation and rapid sand filtration reduce to varying degrees the bacterial content of water. However, they do not necessarily always produce a water safe from bacteriological point of view. Terminal disinfection is, therefore, essential to ensure bacteriological safety of the finished water. Chlorine and chlorine compounds are commonly used for disinfection in India. The dose of chlorine depends on the quality of the filtered water. If the filtrate turbidity is consistently less than 1 NTU, the chlorine dose required may remain more or less uniform.

16.5 COST MODELS

The cost of the water treatment system includes costs of rapid mix unit, slow mix unit, sedimentation tank and rapid sand filters. These costs (civil, mechanical and electrical) depend on the size(s) of the individual treatment units adopted. While civil cost mainly includes cost of construction, the mechanical and electrical costs, relate to the equipments and accessories necessary for effective operation of the treatment units. These costs include the costs of turbine agitator/flocculating propeller, motor and gear assembly etc. for rapid and slow mix (flocculation) units, costs of scraper bridge, end carriage drive, traction drive unit for circular clarifier and the costs of appurtenances such as rate setter, rate of flow controller, flow indicator, headloss indicator, air-blower, backwash water pump etc. for rapid gravity filters. The costs can be modeled separately for individual treatment units and expressed in the functional form as under:

Cost = f (surface area or volume or diameter)

For a given design flow, the costs of other components of water works such as raw water pumps, transmission mains, clear water reservoir, disinfection, clear water pumps etc., as also the manpower component would remain the same irrespective of variation in the size(s) of the treatment units and therefore are not considered in the economic analysis.

16.6 PROBLEM FORMULATION

There could be a number of designs which would satisfy the product quality standards prescribed in the Manual. The objective, therefore, should be to minimize the system cost satisfying all the constraints. A rational comparison of various feasible designs should be based on the capitalized cost or total annual cost of the system. Hence, the objective function will be:

$$\text{minimize } Z, \quad Z = \sum_{i=1}^{4} Ci \qquad (16.14)$$

Where,

Ci = annual cost (AC) of individual treatment unit

AC = ACC + DCW + DMEQ + ENERGY

Where,

ACC = annualized capital cost

DCW = annual maintenance cost of civil works

DMEQ = annual maintenance cost of mechanical equipments/ machinery

energy = annual energy cost

$$ACC = CC\frac{(1+r)^n r}{(1+r)^{n-1}}$$

Where,

CC = Capital cost, i.e. cost (both civil and mechanical) of the treatment unit

r = rate of interest

n = number of years over which the capital cost is to be repaid

CONSTRAINTS

- Suspended solids concentration in the effluent from clarifier $\leq$ 50 mg/l
- Suspend solids concentration in the filtered water $\leq$ 2 mg/l
- Diameter of clarifier $\leq$ 60 m
- Detention time, DT in clarifier $2 < DT < 4$ hrs
- Weir loading rate < 600 m^3/m/d
- Length of filter run $\geq$ 24 hrs.
- Maximum headloss in the filter bed $\leq$ 2m.

SOLUTION

A flow chart for computer aided functional and minimum cost design of water treatment system is presented in Fig. 16.4. The major inputs required are:

(i) Design data on input, decision and state variables and step length

(ii) Data for formulating the cost models for treatment units.

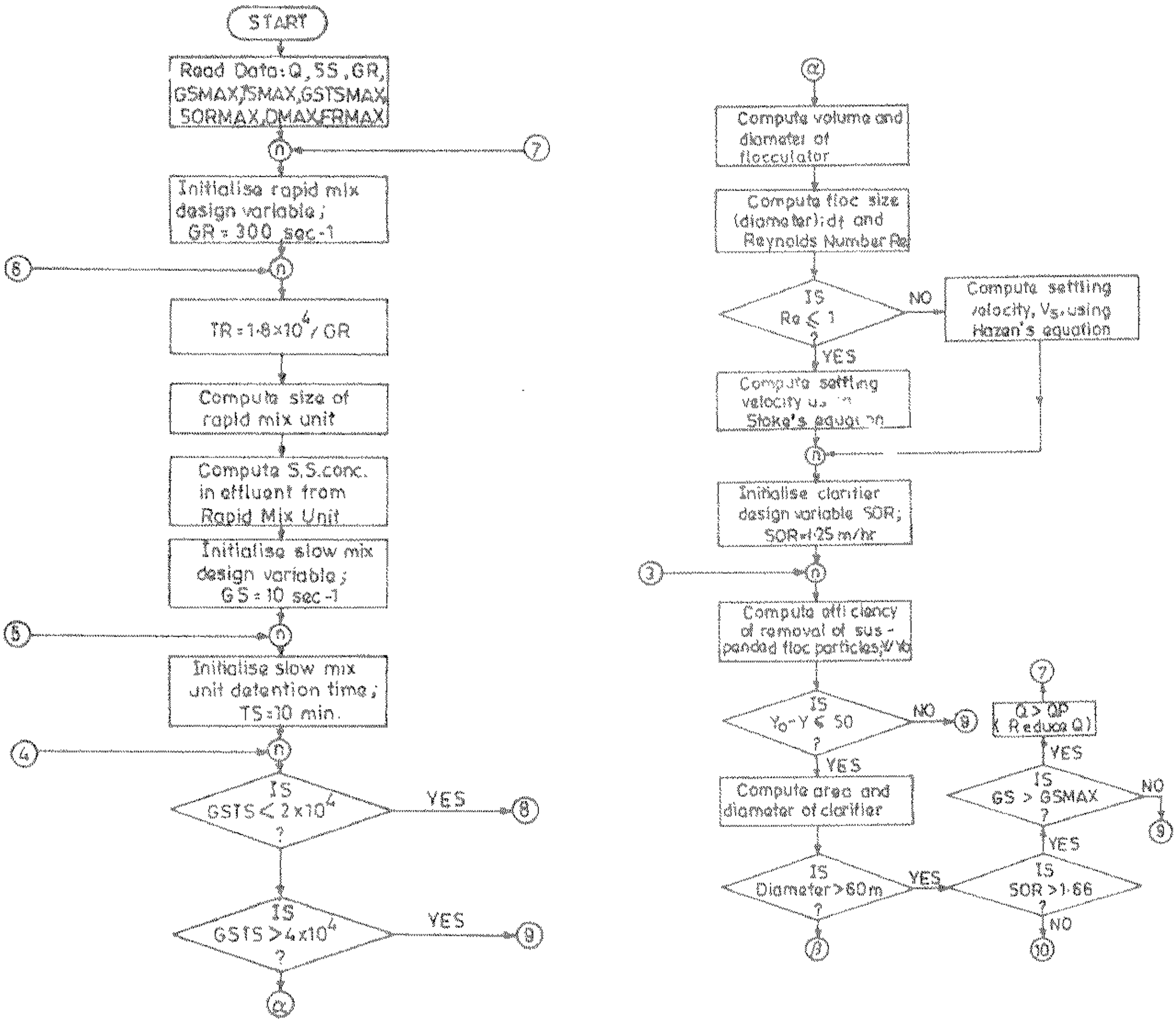
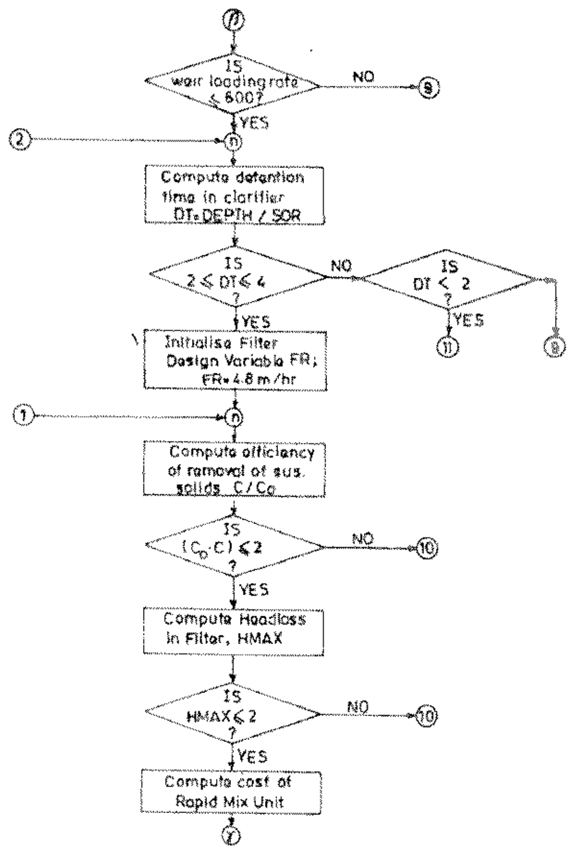
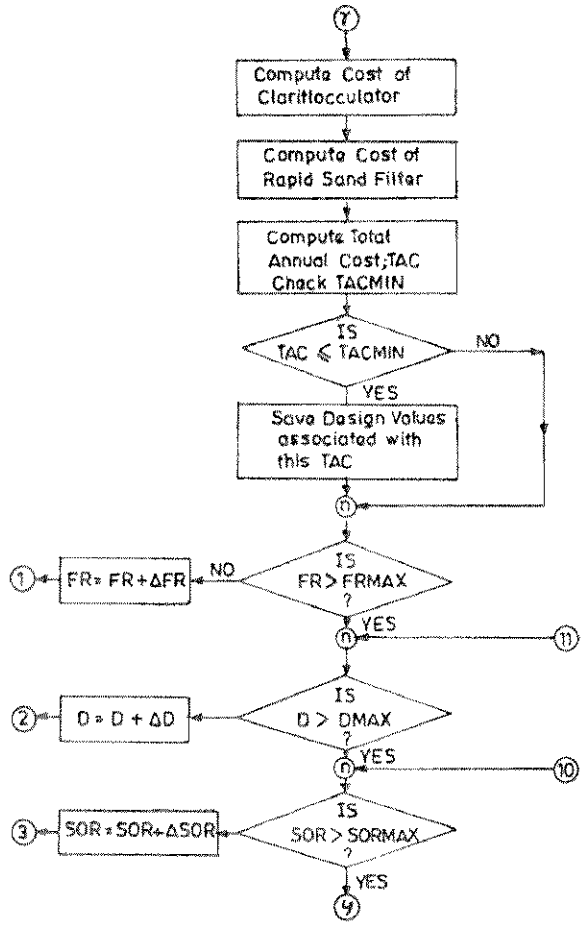
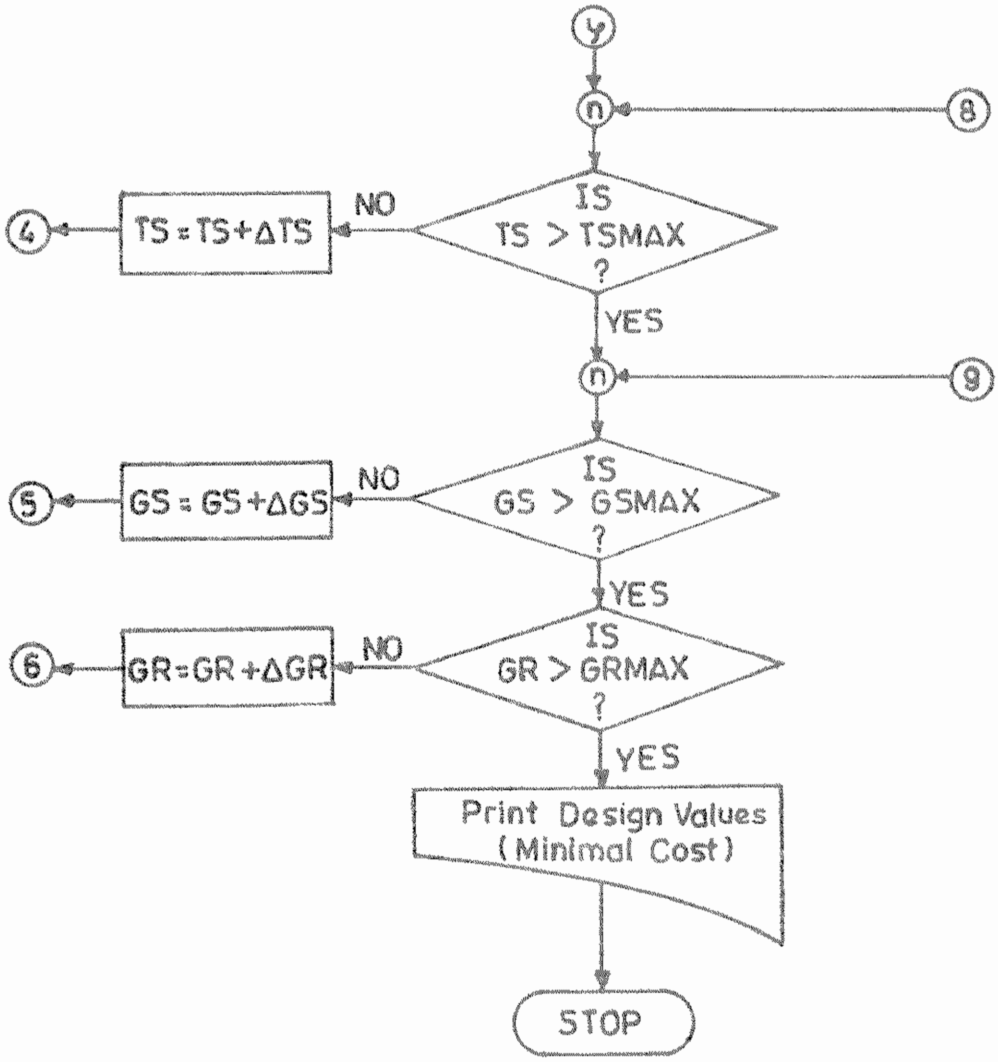


FIGURE 16.4: ALGORITHM FOR COMPUTER AIDED FUNCTIONAL AND MINIMAL COST DESIGN OF CONVENTIONAL WATER TREATMENT SYSTEM


IS
weir loading rate
≤ 600?
NO
YES
Compute detention time in clarifier DT=DEPTH / SOR
IS
2 ≤ DT ≤ 4
?
NO
IS
DT < 2
?
YES
YES
Initialise Filter Design Variable FR; FR=4.8 m/hr
Compute efficiency of removal of sus. solids C/Co
IS
(Co-C) ≤ 2
?
NO
YES
Compute Headloss in Filter, HMAX
IS
HMAX ≤ 2
?
NO
YES
Compute cost of Rapid Mix Unit


Compute Cost of Clariflocculator
Compute Cost of Rapid Sand Filter
Compute Total Annual Cost;TAC Check TACMIN
IS
TAC ≤ TACMIN
NO
YES
Save Design Values associated with this TAC
IS
FR > FRMAX
?
NO
FR= FR+ΔFR
YES
IS
D > DMAX
?
D = D + ΔD
YES
IS
SOR > SORMAX
?
SOR = SOR+ΔSOR
YES



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CHAPTER 17
FINANCING AND MANAGEMENT OF WATER SUPPLY PROJECTS

17.1 WATER SUPPLY FINANCING

The aim of any water supply undertaking should be to provide safe and adequate supplies of potable water at the lowest practicable cost. This demands in addition to the knowledge of water works planning, design, construction and administration, a sound understanding of the elements of financial policy, viz.,

(i) The equitable spreading of the cost of water supply by means of appropriate scales of charges and a potable water rate; and

(ii) The economic aspects of development and execution of the schemes, the methods of providing the capital needed to finance such schemes and the manner of providing for the repayment of such capital.

Apart from the above, financing in the water supply sector requires consideration of expanding requirements of this natural resource due to increase in population, changes in living habits and also increasing requirements due to technological advancements in agriculture, industry, etc.

17.1.1 Scope

The salient features of water supply financing are:

- Methods of raising capital for the installation of the system and provision for repayment of loans where needed;
- Methods of raising revenue to meet the annual expenses of water supply including the determination of tariffs as well as the collection/recovery of charges;
- The application of revenue derived from water charges;
- The formation and use of reserve and contingency funds;
- Accounting in connections with income and expenditure;
- Wages, store and cost accounting; and
- Financial organisation and control such as ordering of goods, authorisation of payments, internal audit, budgeting, insurance etc.

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Note:

1. The possible need for expansion of the system with reference to requirements for drinking purposes and also for sewage disposal should be considered.
2. In the rural sector, the methodology of raising capital should be with reference to change level of service, i.e. replacement of handpump schemes/power pump schemes with point distribution/public standposts by piped water supply with house service connections for domestic purpose and metering which should be the ultimate aim.

17.2 CAPITAL AND REVENUE

The transactions of any water undertaking fall into two broad classes as under:

(a) Capital

Capital from an economical and engineering point of view means the amount invested by an agency from the beginning to the time the works are placed in operation. It includes:

(i) the purchase of property and rights of way;

(ii) payments for the structures equipment and the engineering services.

(b) Revenue

Revenue denotes income sources such as from water charges and government grants. The revenue account is a summary of the expenditure and income of the undertaking during a financial year. Revenue expenditure includes the cost of operating and maintaining the undertaking as well as other charges (loan charges) which have to be met from the income of the undertaking.

17.3 SOURCES FOR RAISING CAPITAL

The various sources available for raising capital are:

(i) Accumulated funds such as water funds with the local body.

(ii) Grant from government,

(iii) Internal borrowing which means investing the surplus funds of the authority itself from its various accumulations such as provident fund and other funds which is the cheapest source as the rate of interest would be the lowest; and

(iv) external borrowings from

- Government with stipulated terms of payment;
- Public through Government bonds;
- Life Insurance Corporation, Nationalised Banks and such other financing institutions;
- Potential buyers who may be desirous of establishing industries in the area;
- Direct beneficiaries; and

- International agencies such as the World Bank, Asian Bank, International Development Authority (I.D.A.).

17.3.1 AUTHORITY RESPONSIBLE.

It is the first charge of the local body or any authority responsible for water supply to provide potable and adequate water supply in the area. Its responsibility starts from the time it is mooted through the various stages of preliminary and detailed investigation, design, administrative, financial and technical sanctions, construction, operation and maintenance and upto the repayment of the loans drawn for the project. The role of other agencies such as the Government in the entire programme is only to help the local body to realise its objective by furnishing the technical and administrative services necessary for the purpose. In regard to the financial commitments, the local body's responsibility is undivided, except in those cases where the State Government agrees to meet a part of the initial capital burden according to its accepted policy.

The problems facing the local body, however is more to find the initial capital to meet the heavy investment of the schemes rather than to device ways and means of repaying the loan over a period of years. The local body has to mortgage the repaying capacity of the beneficiaries against the capital loan secured for the project. The soundness of the project is guaranteed by the assurances of a safe and sufficient water supply to every consumer. (The function of the local body is to sponsor the project on behalf of the consumer). The State and Central Governments generally provide the necessary technical and administrative services to implement the project and arrange for the capital funds necessary therefor. When loans are to be raised by the local body, the State Government acts as a security for the local body to underwrite its financial capacity. It is in recognition of these basic principles of financing and management of water supply utilities for communities that the World Bank and specifically the I.D.A. have come out with loan assistance for community water supplies in the different countries as part of a global programme.

Financial autonomy can be built into the operation of water supply systems, through revolving funds, reliance on bonds and debenture sales in the open market and to public investment institutions as well as assistance from international/bilateral financing agencies. The use of each method or some combination of them, has proven effective elsewhere in launching water systems on the way to self support relieving the State and Central Governments of some part of their burden of support. The local bodies/water supply undertakings should realise that the schemes have to be self supporting.

No proposal for supplying water to a community should be considered complete until adequate arrangements for the disposal of the community waste water are included. A proper monetary assessment of the direct and indirect effects of water supply and sewage facilities for urban communities has to be made to present the financial implications in proper perspective. It will be seen that the consequences of postponing these facilities will be greater than the cost of providing them.

17.3.2 The Relative Merits of the Various Methods

The obtaining of a loan from the Government is beset with the difficulty that the Government will not be able to oblige all local bodies interested in the provision of water supply facility due to its own financial limitations. The local body will have to claim priority for its scheme, advancing its own reasons. However, the State Government may go in for a public loan on behalf of the local body acting as a borrowing agency from LIC and other agencies. It should consider the issue of serial bonds for the purpose since it is believed that this would attract a wide class of investors on account of their varying lengths of repayment.

Direct beneficiaries of the project would willingly contribute substantial part on the initial investment to be adjusted later against the house service connection fee and water charges. This potential source, which has not been tapped so far, should be taken full advantage of.

Industries which are interested to buy water may give loans at reasonable rates of interest and it would be worthwhile to exploit this source fully.

The International Agencies insist on certain preconditions relating to the preparation of the project, its execution, operation and maintenance including revenue collection.

17.4 METHOD OF RAISING REVENUE

The sources of revenue are the funds received by general taxation such as water tax or a portion of the general property tax which is realised by assessment on all taxable property and water rates paid by those who use the water, more or less in proportion to the amount consumed.

17.4.1 Water Tax

Since the provision of a water supply to a town enhances the value of the property, a water tax is justifiable on the annual rental value of the property. This may be a separate tax or included in the general property tax but it is desirable that the revenue under this head is earmarked for water supply purpose.

For supply of water for general purposes such as the supply through public fountains, places of public resort, for fire protection etc., for which no charge is leviable on the public individually or collectively, the local body has to apportion a part of its income from general taxes for meeting this obligation. This will help the water wing of the local body to manage its affairs on a sound business footing. This revenue is assured for the local body as it is independent of the actual quantity consumed and the party catered to. This revenue is preferably utilised to pay annuity charges on the loan obtained for the installation of the water supply system and can be adjusted to meet this commitment fully.

17.4.2 Water Rates

The revenue from the sale of water or water rates recoverable from parties actually consuming the water such as for domestic purposes or for commercial and industrial purposes is utilised to meet the annual recurring cost of operation and maintenance and to provide for a reserve for meeting the capital expenses for future improvement to the system.

Any major augmentation of the system should, however, be dealt with as a new scheme for which the capital is to be raised in the usual manner.

The simplest form of a water rate is a flat rate payable monthly or quarterly by the customer regardless of this quantity consumed, the services being not metered. Many local bodies also adopt the system of a fixed tap rate charged per tap irrespective of the quantity used. This rate is easily fixed by dividing the total revenue required by the total number of customers or taps so as to make the local body meet its obligation. But this system leads to waste of water and is therefore not satisfactory. Since the charges for water bear no direct relation to the cost of service rendered, such a rate is discriminatory.

The most equitable method will be based on metering of all the supplies. The quantity actually accounted for by the meters is invariably less than the quantity produced since there is a considerable wastage as 'unaccounted water', which should also be considered in fixing the water rates. This will mean that even the supplies to places of public resort for which the general wing of the local body has to bear the charges should also be metered. This is all the more necessary as substantial wastage occurs at these supply points and the consumption figures will point out to the need for greater vigilance.

Some local bodies allow a free allowance for the metered supplies, based on the water tax collected and charge only for the excess. This is also not desirable as the revenue collected by water rates is to finance the operation and maintenance costs fully. A worthwhile alternative is to collect a fixed charge called the service charge per consumer in addition to the charge for the water consumed. This fixed charge is to provide for the meter rent where the meters are supplied by the department and for the overhead charges for the billing and collection, meter reading, maintenance of meters and prorated general expenses. The entire supply as measured by the meter is to be charged for, at either a uniform rate or by graded rates. Consumption above a reasonable quantity may be charged at a higher rate to discourage such drawl. This will enable all consumers to get an equitable supply. Again there may be separate meters for measuring the supply for domestic and non-domestic uses. The rates for non-domestic and industrial purposes may be fixed higher. The water rates are to be carefully fixed taking into account the following:

(i) The rate should be high enough to fetch the necessary revenue and not excessive as to discourage consumers from making needed use of the water for domestic needs and for personnel hygiene in particular.

(ii) The rate should be such as to make the amenity more or less self paying and worked on a no-profit-no-loss basis.

(iii) The rate should be such as to provide for generating source for expanding the system to take care of increasing requirements.

It is desirable that water supplies at least to all cities having a population of one lakh or more are metered.

17.5 WATER SUPPLY MANAGEMENT

Efficient and effective management of water supply systems is most essential for their proper functioning. A water supply organisation should be treated as a business enterprise involving managerial skills and engineering knowledge to make it successful in service, in safety and in financial considerations. The quality of water supplied should be the prime consideration for any water supply organisation as the safety and health of the people depend upon it.

The technical and engineering problems involved in the running of a water supply organisation call for a qualified Public Health Engineer as the head of the management.

17.5.1 SCOPE

A good management of a water supply system includes a number of functions such as;

(i) Provision and maintenance of adequate facilities;

(ii) Good and smooth operation;

(iii) Efficient and economical maintenance;

(iv) Efficient administration;

(v) Establishment of sound fiscal methods;

(vi) Development of equitable water tax and water rates;

(vii) Efficient control of equipment and supplies;

(viii) Keeping the wastage of water to a minimum;

(ix) Good public relations and a satisfactory service to consumers; and

(x) Development of technical and financial plans for future expansion.

17.5.2 TASKS

A successful management involves;

(i) A detailed knowledge of the components of the system, the basis of their design and the assumptions made;

(ii) Objective plans with charts to indicate the work, past, present and future and the time schedules;

(iii) Promptness in changing plans to meet any contingencies and unforeseen conditions;

(iv) Detailing job specifications;

(v) Prescription of the duties, powers and responsibilities of each employee of the organisation for routine maintenance and in emergencies and to prepare a list of "do's and dont's" for the operational staff;

(vi) They should be drafted in a form which can be easily understood by the operators, preferably in the languages, they are accustomed to;

(vii) Evaluation of a good record systems of manuals, drawings, lubrication charts for machineries etc., so that they are available even after a decade or two when there is every likelihood that they cannot be obtained from manufactures etc. due to possible change of design or other reasons;

(viii) Search for an elimination of unnecessary job inefficiencies in the working of the system;

(ix) Recruitment and inservice training of the personnel, technical and nontechnical;

(x) Recognition of merit and choosing efficient men to occupy position of higher responsibility. Persons associated at the construction stage should be preferred as key personnel since they have had the opportunity of understanding how the system had been put together and the mode of its working;

(xi) A thorough knowledge of the business methods including financing, budgeting, billing and revenue collection work and investment of funds; and

(xii) Carrying out health education programmes to get the full cooperation of the public, in not only preventing contamination of the water supply but also making them appreciate the value of protected water, with a view to prevent wastage.

17.6 FINANCIAL APPRAISAL OF WATER SUPPLY PROJECTS

17.6.1 INTRODUCTION

Project appraisal is the analysis of costs and benefits of a proposed project with an aim of obtaining a rational allocation of scarce resources among alternative investment opportunities in view of achieving certain specified goals in the National Development Programme. As the number of projects to satisfy the identified needs exceed the resources available, project appraisal becomes necessary to choose the best alternative one from a package of projects. Moreover, careful project analysis will point out unrealistic or questionable assumptions and indicate ways in which a project can be modified to improve its wealth generating capacity. A project carefully analyzed and revised in the light of this analysis has a much improved chance of being implemented on time and of yielding the benefits aimed at.

In projects analysis, there is a critically important distinction to be kept in mind between two complementary points of view viz.

(i) Economic analysis; and

(ii) Financial analysis.

Economic analysis is concerned with the total return or productivity or profitability to the whole economy of all the resources committed to the project regardless of who in the society contributes them and regardless of who in the society receives the benefits.

The social cost benefit or economic analysis aims at evaluating the profitability according to the impact on the society as a whole, while the financial cost benefit analysis tries to assess the profitability to the operating entity.

Accounting prices are used in social cost benefit analysis to establish the undistorted basic relationship between world prices and domestic prices. There are two ways of doing this. The first method known as Little and Wireless method contemplates valuing all goods and services in terms of world prices without the use of exchange rate. The second method known as UNIDO method envisages valuation of goods and services in terms of their contribution to consumption at domestic prices. Export and import prices, under this method, are converted into domestic price relatives by the use of Shadow Exchange Rate, the purpose of this adjustment being to allow for tariff induced distortions between domestic and border prices.

On the other hand, financial analysis is concerned with the individual financial entities which participate in a project, viz. entrepreneurs, businessmen, farmers, public agencies, etc., each is interested in the return to the equity capital one contributes. Project appraisal is very important for the developing countries which are in the process of achieving stupendous task of recycling of financial and other resources for productive purposes and welfare of the poor people.

The analytical techniques employed for Economic and Financial appraisal comprise deriving values for the net present worth (NPW), internal rate of return (IRR) and the benefit cost ratio (B/C). These are defined as follows:

Net present worth (or Net present value): (NPW/NPV)

This is defined as the present worth of the net benefits of a project discounted at the opportunity cost of capital.

i.e. Net present worth = (Present worth of benefits - Present worth of costs)

Internal rate of return: IRR

This is defined as that discounted rate at which the present worth of benefits, is equal to the present worth of costs. This measure represents the return over the life of the project to the resources engaged in the project.

To determine IRR the NPW is first calculated at two different discounting rates (r_1 and r_2 being the higher and lower discounting rates)

$$IRR = r_2 + \frac{(r_1 - r_2)NPW_2}{NPW_2 - NPW_1}$$

Benefit Cost Ratio: (B.C. Ratio)

This is defined as the present worth of benefit divided by the present worth of costs.

17.6.2 Project Cycle

Any project has to under go the following project cycle:

(i) Identification

The first phase of the cycle is concerned with identifying projects that have a high priority with reference to the set objectives and needs of the country.

(ii) Preparation

The next stage is project preparation which should cover the full range of technical, institutional, economic and financial conditions necessary to achieve the project objectives:

A critical element of preparation is identifying and comparing technical and institutional alternatives for achieving the project objectives. This has to be followed by a more detailed investigation of the most promising alternative and the most satisfactory solution is finally worked out.

(iii) Appraisal

As the project takes shape and studies are nearing completion, the project is scheduled for appraisal. It is a critical stage of the project cycle because it is the culmination of the preparatory work, provides a comprehensive review of all aspects of the project, and lays the foundation for implementing the project and evaluating it when completed.

Appraisal consists of four parts viz.,

(a) technical

(b) institutional

(c) economic

(d) financial

Technical appraisal is necessary to ensure that the project is designed in a sound manner as least-cost solution following all the accepted engineering norms. The various technical alternatives considered and the solution proposed are part of technical appraisal. This also includes appropriateness of technical standards adopted, reality of the implementation schedule, likely hood of achieving the expected results, review of capital cost and operating cost estimates and engineering and other data, proposed procurement arrangements etc.

Second part is the appraisal of the institutional aspects of the project which also includes recognition of the need for a continuous re-examination of the institutional arrangements with an open mind to accept new ideas and adopt a long term approach that may extend over several projects.

Third is the economic appraisal which aims at assessing the contribution of the project to the development objective of the country and this remains the basic criterion for project selection and appraisal.

The fourth and the last one is financial appraisal which has several purposes viz, to find out whether the project is financially viable to meet all its financial obligations including debt servicing, to generate adequate working capital, to generate funds from internal sources, to earn a reasonable return on its assets in operation and make a satisfactory contribution to its future capital requirements. The financial review often highlights the need to adjust the level and structure of prices charged to the project.

It is the objective of economic analysis to identify whether projects have Net Present Worth which will be a positive quantity and whether, Benefit-Cost ratio will be atleast unity. In the absence of these conditions being fulfilled, they also provide criteria based on which projects could be arranged in the order of preference.

(iv) Negotiations with the Financing institutions

Negotiations is the stage at which the lending institution and the borrower endeavour to agree on the measures necessary to assure the success of the project. These agreements are then converted into legal obligations, set out in the loan documents. All the principal issues that have been raised prior to and during appraisal are dealt with in the loan documents. The drafting and negotiation of the legal documents are an essential part of the process of ensuring that the borrower and the lender are in agreement not only on the broad objectives of the project, but also on the specific actions necessary to achieve them and the detailed schedule for project implementation.

(v) Implementation and Supervision

Implementation by the borrower and supervision by the lender form the next stage. Progress reports followed by field visits constitute part of supervision.

(vi) Evaluation and Feed back

This is the last stage of the project cycle and provides lesson of experience which are built into subsequent project identification, preparation, and appraisal work.

17.6.3 FINANCIAL APPRAISAL

Financial Appraisal of Water Supply Scheme is necessary:

(i) To ensure that the project is financially viable; whether the project will meet all its financial obligations including debt servicing; whether there will be adequate working capital; whether the project can generate funds from its internal resources to earn a reasonable return on its assets in operation and make satisfactory contribution to its future capital requirements;

(ii) To adjust the level and structure of prices charged, when need arises; and

(iii) To ensure recovery of investment and operating costs from the project beneficiaries.

The finances of a project are closely reviewed through projections of the balance sheet, income/expenditure statement, and cash flow. Where financial accounts are inadequate a new accounting system has to be established with technical assistance financed out of the loan.

The economic appraisal of a project aims at assessing the contribution of the project to the development objective of the country whereas the financial appraisal aims at ensuring the financial viability of the project.

Two important factors which lead to the distinction between financial analysis and economic analysis are:

(a) exclusion / inclusion of some costs and benefits in the appraisal of a project; and

(b) valuation of costs and benefits and market prices or some other prices.

In the Project Appraisal Technique, the costs and benefits of the project in financial/ economic terms are evaluated. It is easy to identify costs and benefits in financial terms whereas it is difficult to identify in economic terms. The project incurs expenses on capital investment, such as machinery and equipment, operation and maintenance cost, purchase of raw materials, payment of wages and import of goods and services etc. In addition, the project has to pay taxes, import duties, fees, repay the loan with interest and allow for the depreciation of fixed assets. The project gets its return from the sale of goods and services and also receives subsidy, if allowed by the Government, which reduces the costs or add to the income.

Two types of costs and benefits are encountered in the appraisal of a project-one involves the use of resources, and the other which does not involve use of resources, but it is a transfer of resources from the project to the Government or any other institution/ individual (taxes, fees, duties, loan repayment and interest) or viceversa (subsidies). Thus in the identification of costs and benefits, it is useful to deal with each individual item, briefly described below.

(i) Transfer Payments

(a) Depreciation

It is a provision of funds over the life time of the project for its replacement. Depreciation is excluded from the economic appraisal of a project as it is only an accounting concept.

(b) Interest rate

Similarly, in the economic analysis no allowance for interest on the capital employed is made as the analytical technique automatically takes care of the return of capital (interest) in determining the worth of a project.

(c) Opportunity cost

In the economic analysis, estimated income foregone would feature as cost, while in the financial analysis, it would not feature as cost.

(d) Taxes

Taxes are also transfer payments. In the financial analysis where analysis is done from the point of view of the individual entity of a project, all taxes are treated as financial costs and benefits is done from the point of view of society, taxes are transfer payments. Taxes are not included as cost in economic appraisal as they are in the nature of transfer payments which do not involve the use of resources. But in the case of financial appraisal, taxes are included on the cost side as it is a financial cost to the project. This would apply to all types of taxes-income tax, import duties, local taxes etc.

(e) Subsidies

In financial analysis, subsidy reduces cost and adds to the income of the project. In the case of economic analysis, it is a transfer payment and increase or decrease in it does not add or substract income from the point of view of society.

(f) Social Costs and Benefits

In financial analysis, social costs such as air pollution, noise, wear and tear of road etc., would not enter as costs in the calculations as these are no costs to the individual project. But social costs would be included in an economic analysis when the project is appraised from the point of view of society.

(ii) Solution of proper prices

In financial analysis, costs and benefits are calculated at market prices. But in economic analysis costs and benefits are calculated after making certain adjustments in market prices. The rational, of course, is the efficient use of available resources which have alternative uses. An economy should utilise more intensively that resource whose process is lower compared to a resource whose process is higher. However, since markets of the factors of production are no perfect in reality, the price of an item may not correctly reflect the scarcity or abundance of the factors of production.

However, the prevailing market price do not reflect the intrinsic value of goods as they are distorted in many developing countries due to the following factors:

(a) Inflation

(b) Currency overvaluation

(c) Wage rate and unemployment

(d) Imperfect capital markets

(e) Tariffs, import quotas

(f) Inequality in distribution of wealth.

For example, in a labour surplus economy, given the supply of and demand for labour, market wage would be higher than the wage that should be operated, based on the equilibrium of demand and supply.

Similarly the official foreign exchange rate may not correctly show the scarcity or abundance of foreign exchange. In the economic analysis, costs of items are calculated not on the basis of prevailing prices in all cases, but on modified prices assumed on the basis of their supply and demand position. These assumed prices are termed as shadow prices or accounting prices.

Also the prices charged for the product of a project may be lower for various socio economic considerations. In such cases, the modifications of the selling price of an item is done in economic analysis.

17.6.4 FINANCIAL ANALYSIS STATEMENTS

The consultancy study on tariff conducted in a town reveals that the average monthly household income of the persons who are likely to obtain house service connection is about Rs. 630. It is considered that such a household will not be able to pay more than 2% of its income. Hence the monthly water charges are fixed at Rs. 12.60 per connection or 95 paise/1000 litres for domestic purposes based on the tariff study conducted in the town. The tariff for non domestic consumption of water is fixed for commercial and industrial requirements respectively.

The Average Incremental cost of water per 1000 liters works out to Rs. 1.34 (Appendix 17.1). The tariff for domestic consumption works out to about 70% of the average incremental cost.

The Economic Analysis has been carried out using a discount rate of 8.5%. The Net Present Worth is worked out at two different discount rates of 8.5% and 2.0%. The results are as follows (Appendix 17.2).

Benefit cost ratio	:	0.62
Net Present Worth	:	Rs. (-) 0.7933
Internal rate of return	:	2.34%

The assumptions made for the financial forecast are appended (Appendix 17.3)

Three important financial statements namely, Income and Expenditure statement, Source and Applications of funds (Cash Flow) and Balance Sheet relating to the financial analysis of a water supply project are prepared and furnished (Appendices 17.4, 17.6 and 17.9).

The funding pattern is given in Appendix 17.5. Interest calculations during the moratorium period are furnished in Appendix 17.7. Working sheet for arriving at the Annuity is given in Appendix 17.8.

The financial analysis of project has been done with the following lending terms and conditions. The loan period is 25 years with a moratorium of 5 years from the year of receipt of first loan.

The results of the financial analysis indicate that the project can meet its commitment (Annual operation and maintenance cost and Annuity) throughout its life if an element of subsidy is given.

The financial analysis of the project reveals that it is necessary to have a grant element to an extent of 75% of the project cost to make the scheme viable as explained below:

The total cost of the project is Rs. 1.768 million. The monthly water charges are fixed at Rs. 12.60 per month per connection from 1988-89 for domestic purposes taking into account the paying capacity of the users. At this low rate, the project is likely to incur loss financially and 75% of grant towards capital cost is necessary to make this project a financially viable one. If 75% grant is not given, there will be cash deficit.

The modus operandi for the economic and financial analysis as well as the evaluation of benefit cost ratio in respect of different funding agencies will be the same. However, the discount factors adopted by various funding agencies differ which should be taken into account for appraisal of projects.

17.7 STATUTORY WATER AND SANITATION BOARDS

Most of the local bodies at present face serious handicaps in the promotional stages of a project, in its prefinancing stage and in the fund raising stage as well. Saddled as they are with responsibilities beyond their capacity and circumscribed by limitation of finance and procedure, any attempts by them individually to raise loans in the open market to finance a local water supply project may not attract encouraging response. This problem could be satisfactorily solved by the creation of autonomous water and sanitation boards.

These boards are devices by which State Governments will be able to establish corporate public entities to construct, manage and operate water and sanitary services on a fully commercial basis in large metropolitan areas as well as in smaller urban communities. Not only provision of safe drinking water is important to health but so are sewage disposal and related sanitary measures. These are all interconnected in their effects on health as well as technically the failure of any one service endangers other and consequently affect the health of the community. These boards should be empowered and equipped to raise such capital from local resources and the open market to supplement the provisions which the government at the State and Central level could not provide for the purpose. Such boards will have the advantage of;

(i) Increased efficiency resulting from financial autonomy;

(ii) Improved ability to raise capital with confidence;

(iii) Better opportunities for small municipalities grouped together to finance and operate their schemes as a business proposition;

(iv) Economies implicit in a common source of water which may be made to serve several undertakings;

(v) Better realisation of water revenues when this duty is divorced from local politics;

(vi) Economies possible by pooling technical and administrative staff to serve a number of municipalities; and

(vii) Opportunities for equalising the rates in every region.

A statutory water and sanitation board may be set up at State level with regional boards if and to the extent necessary within the State, to provide water and sanitation services and to collect revenues to meet such services, to raise the capital needed to provide the facilities and to exercise all other corporate powers necessary to act on behalf of the local bodies within its jurisdiction.

Normally, such boards would encompass all activities, including production, conveyance and distribution of water within their statutory areas and also for the collection, treatment and disposal of sewage from that area as well as other sanitation services. It is, however, possible that some local bodies may prefer to purchase water in bulk from the statutory boards and arrange for the internal distribution themselves and may also prefer to have the statutory board take over sewage in bulk from the local area and arrange for its treatment and disposal. This should be avoided as far possible as the supply and distribution of water as also collection and disposal of sewage are two inter-dependent functions and the divisions of such functions amongst two independent agencies might lead to inefficiency and avoidable difficulties for both parties. Any local body managing its systems satisfactorily need not necessarily come under such a board.

17.8 CONCLUSION

In India, water is considered a "free gift of nature" and therefore charging for water, by any agency, may it be a Municipal Corporation/Council/Panchayat or a Water Board, is not liked by the people. Providing water supply was not considered to be a commercial or even no-profit-no-loss activity.

The situation has since changed considerably. Per capita use of water has increased, sizes of human settlements are increasing at faster rates and industrial and agricultural uses of water have increased considerably. As the reliable sources of water are getting exhausted, we have to go in for additional sources of supplies. Irregular and inadequate rains, as also the pollution of surface and ground waters being caused by discharges of industrial and domestic wastes have rendered the problem of meeting rising drinking water demands, increasingly difficult and expensive.

No doubt, water is still the free gift of nature but it is so in "as it is where it is" condition. When it is desired that water, as is available in nature, should be made safe for drinking and transported to the points of consumption, it becomes a "Commodity" i.e. it acquires economic value. A water works must therefore be treated as an industry and be built and operated as a " Commercial enterprise, with professional approach where the aim is not only to meet the debt servicing and operational costs, but also to earn a fair return on the investments made, so that future expansion of the water works can be financed atleast partly and the undertaking can attract funding from outside sources. It is in this context that the financing of water works, water pricing policy, and overall financial management of water supply undertakings have been receiving serious attention of the planners and administrators at National and State levels.

India had launched the International Drinking Water Supply and Sanitation Decade with effect from 1981. Hence both the Central and State Governments have been providing appreciable financial assistance to the local bodies for construction of new facilities and for augmentation/ rehabilitation of existing facilities. Almost in all the States and Union Territories capital expenditures on rural water supply are fully funded by the States/ Union Territories and the Central Government also provides funds to the States and Union Territories, under Accelerated Rural Water Supply Programme.

For financing Urban Water Supply Projects, normally the following sources of funds are available.

(i) Internal borrowing;

(ii) Government grants and loans;

(iii) LIC loans;

(iv) Open market borrowings;

(v) Loan from financing institutions; and

(vi) International/Bilateral aids

If a water supply undertaking has to function in the long run on a self-reliant basis, it must charge for supply of water and collect revenues adequate for meeting debt servicing, operation and maintenance charges and also generate surplus for future investment.

If water could be sold to all consumers at the same rate, like any other commodity in a free market, the water tariff structure could be simple. In that case it would be necessary only to fix water charges at suitable intervals of time and charge for sale of water accordingly, depending upon the basis for charging of water i.e. metered supply, non-metered supply etc. However, tariff based on uniform rates of water cannot be adopted in a country like India, where a large percentage of population is below poverty line. Water is to be made available to all in quantities sufficient to meet atleast the minimum needs.

Therefore, appreciable quantities of water may have to be supplied to poor section of the society either free of cost or at adequately subsidised rates, which would be much less than the unit cost of water. The loss thus incurred will have to be made good by charging higher rates to consumers who can afford to pay those rates such as industries, commercial establishments, traders, professionals as well as owners of high value properties, etc. Therefore, it is necessary to identify different categories of consumers as stated above including poor section in a city or town and estimate the likely consumption of water by each of these categories of consumers. Graded rates of water will have to be fixed for these consumer categories, considering their paying capacities, such that the total annual revenue receipt would be equal to or more than the total annual burden.

Water tariff structure also depends upon the methods of charging for sale of water. Generally these are based on:

- Percentage of rateable value of a property;
- Flat rate depending upon size of a connection; and
- Metered supply.

Charging on the basis of volume as measured by meters is the most equitable and rational method, as a consumer pays directly in proportion to the water consumed. Moreover metering helps in accurately estimating the consumption of water by various categories and in locating wastages and leakages. However, this method of charging has the following disadvantages;

- Metering increases unit cost of water;

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- Meters often go out of order, requiring frequent removal, repairs and reinstallation; and hence accurate measurement of water is not possible;
- Large skilled staff is required for installations, repairs, testing, reading and billing;
- Fixing of a meter reduces pressure;
- Where unfiltered supply is made, meters often choke, requiring frequent cleaning;
- Where water supply is intermittent, meters may record more reading than the actual consumption of water;
- During temporary absence of meter (when removed for repairs or testing) or when it is not in working order, billing on the basis of average consumption in the past, is often disputed by consumers and this situation affects recovery of bill.

For the above reasons, universal metering of water is not being practised. Generally only bulk consumers, like industries, institutions, commercial establishments and large premises like co-operative housing societies, etc. are metered, where as individual domestic consumers are charged on the basis of either flat rates depending upon the size of connections or as percentage of rateable value of a property served.

From the foregoing paragraphs it will be clear that selection of a suitable tariff structure needs consideration of aspects such as income distributions, the possible mix of service levels and the systems of charging. In short the social objectives and systems constraints would influence the tariff structure. Generally the tariff structure should aim at:

- Collecting target revenue;
- Sharing out the burden fairly between users of different income groups (by providing different levels of services); and
- Administrative simplicity and efficiency.

To these aims must be added the one for influencing consumer behavior. In other words pricing policy must be such that it would induce consumers to economise use of water. Considered from this angle, charging on the basis of rateable value of a property or collective metering of an apartment block are the systems which provide little incentive to economise on use of water.

Annual burden imposed by a water supply scheme consists of two components, viz.,

- Fixed charges comprising debt servicing and such staff and minimum maintenance charges as are necessary to be incurred.
- Variable charges comprising power, chemicals and raw water bills which are proportional to the quantity of water produced.

When a facility like a water supply scheme is constructed and services are made available to a community, it imposes financial burden as stated above. On account of the services made available the property value goes up. Therefore, it is justifiable for a local body to levy

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betterment tax on all premises and properties which can avail of the services though the facility may not be actually used by such premises and properties. Such a betterment tax could be related to the fixed charge component of the financial burden caused by the scheme.

For recovery of variable charges, rates based on consumption of water may be charged and these rates can be different for various categories and slabs of consumption. These charges would be payable by only those who actually consume water.

Authorities such as water supply boards generally do not own water works. The functions of these boards are generally restricted to planning, designing and constructing facilities on behalf of local bodies and then to transfer the works to the owners who have the responsibility to operate the works and also to collect water charges. The boards receive only the agency charges to cover the cost of their establishment, these agency charges being treated as a part of the capital cost of work, planned and constructed by the boards.

There are, however, a few boards, who besides carrying out the functions of planning, designing and execution of works also own water works. These boards operate the water works and also collect water charges directly from the consumers they serve.

While concluding, it is to be stated that a water supply system has to be created since it is essentially required for sustenance of life. It may be initially uneconomical but the water supply project may be evaluated on social cost-benefit analysis method. It is difficult to quantify the social benefits and relate them to the capital cost. The following factors which are likely to get developmental impetus due to creation of water supply system and incidentally a waste water disposal system should be identified:

- Industrial and agricultural development;
- Improvement in living habits, health and hygiene; and
- Increased productivity.

Water supply being a community service, the economical analysis and the financial analysis should be done prudently and judiciously.

CHAPTER 18
LEGAL ASPECTS

18.1 GENERAL

In India, laws related to use of water date back to the period when the CODE OF MANU was prescribed, over 3000 years ago. Water was considered public property, subject to public administration, several penalties were prescribed for unauthorised use and for causing harm to water holding structures and for causing pollution of water. Upstream points along a river were reserved for drawl of drinking water and in-situ uses of water such as washing clothes, bathing etc., were permitted only at the downstream.

The establishment of priorities in the use of water for multiple purposes and among several users for the same purpose is one of the longest established features of water law.

18.2 SYSTEM OF ACQUISITION OF WATER USE RIGHTS

There are currently three major systems of acquisition of water use rights. These are:

(i) The riparian rights system;

(ii) The prior appropriation system; and

(iii) Administrative disposition of water use rights.

18.2.1 RIPARIAN RIGHTS SYSTEM

The riparian rights belong only and equally to those who possess access to water through ownership of land abutting on a stream. A person having riparian right can initiate use of water at any time and insist that his right be accommodated with other user, or that a share of the water be allotted to him. Riparian right is a form of real property, and is a part of land law. Thus this right is appurtenant to the land, in the sense that a person who purchases or inherits riparian land automatically acquires the water right, although it may not be specifically mentioned. The riparian does not own the water, but owns only the right to use it on his riparian land, and to have it flow to his land so that it may be used.

As a rule only the natural flow of a stream is subject to riparian rights. Water added artificially to a stream i.e. the so called "developed" water is not subject to riparian rights. It belongs to whoever developed it, unless the increased flow was caused by mere clearing of obstacles. Riparian rights do not attach either to waste water which seeps or escapes from ditches or reservoirs, or to foreign waters drained artificially from a different water shed. They do attach to a spring when it is the source of the stream and also to the under flow of a stream.

Under this system there are two operating doctrines, viz. (i) Natural flow Doctrine, and (ii) Reasonable use Doctrine.

18.2.1.1 Natural Flow Doctrine

Under the natural flow doctrine the riparians have the right to use water on riparian lands, in as much quantity as they need, without consideration of the needs of their downstream users, if their use is confined to so called "Natural" or domestic purposes, i.e. drinking, washing. cleaning and the watering of live stock. However., when they make use of the water for other than domestic purposes even though still within riparian land, they may become subject to action by the lower riparian if he sustains harm in the use of water to which he is entitled; since he has the right to expect the water to flow to him, in its natural and undiminished state. Also any use not connected with riparian land, which affect the flow of water, even though it does not cause any harm, is considered subject to action.

18.2.1.2 Reasonable Use Doctrine

Because of the limitation of the natural flow doctrine in the use of water law as a tool for purposes of social engineering, the trend is away from, "Natural Flow Doctrine" and towards acceptance of the "Reasonable Use Doctrine".

As a rule, in determining reasonableness, such factors as social utility, capacity of the stream, benefit to the use and suitability to the purpose of the stream are taken into account, mostly retaining the fundamental right of the riparian to the reasonable use of the water of the stream, but free from unreasonable interference with other uses.

A number of uses have received judicial approval and their limits have been defined to some extent. Domestic use includes water for drinking, cooking, laundry, sanitation and other household purposes. A substantial quantity of water may be necessary to fulfil domestic uses where people gathered in hotels, apartment houses or resorts. Even military camps are given the privilage of taking water for domestic use. But domestic use does not include municipal uses in nonriparian areas of cities. A city situated on the banks of a stream is not a riparian right holder in any sense that would permit it to divert water and sell it to inhabitants who live on lands not adjacent to the stream.

The reasonableness of a particular use of water by a riparian is a question of fact and each case must be determined with reference to its own facts and circumstances. The use of water by one riparian that causes substantial harm to another, can generally be said to be unreasonable unless the utility of the use out weights the gravity of the harm. Wasteful uses or wasteful method of use may be unreasonable.

A prescriptive right may be described as a power to take water without reference to the rights of riparian owners. The right obtained by the prescription is absolute there being no corrective rights between the riparian and the prescriptive user.

18.2.1.3 Loss Of Riparian Rights

Generally a riparian right cannot be lost by abandonment or simply by non-use of water. Since use does not create the right, non-use cannot destroy it. However, there are some exceptions to this in some places when a riparian may lose his right.

(i) When a non- riparian or excessive use has been made continuously and adversely for the period of the status of limitations.

(ii) When prescriptive rights to the use of water have been acquired for such adverse use.

(iii) When the legal doctrine known as "estopple" is operative [e.g. when a riparian has permitted a non-riparian to construct a dam on his land at great expenses he is "estopped" (prevented) from revoking the license and destroying the value of the irrigated non-riparian land].

(iv) When there has been silent acquiescence by a riparian in respect of an upstream use of water, for which large sums of money have been spent for the public benefit; though he may still have the remedy for damages to compensate him for the rights he has lost.

(v) When a public or quasi-public agency needs water, it has the power to take it as long as it pays just compensation for the use it causes. (Any government authority, has this "right of eminent domain", and quasi-governmental bodies such a Water Supply Boards, may be given a similar power by grant from the state that creates them.)

18.2.2 PRIOR APPROPRIATION SYSTEM

The two cardinal principles of the doctrine of prior appropriation are:

(i) That beneficial use of water and not land ownership gives the basis of the right to use water; and

(ii) That priority of use and not equality of right is the basis of the division of water between appropriators when there is not enough for all.

18.2.2.1 Elements Of An Appropriation

An appropriation is the right to use a specific quantity from water from a public source of supply for a beneficial purpose, if that quantity is available free from the claims of prior appropriators. An appropriation requires:

(i) The diversion of water from a stream or other source;

(ii) The intent to appropriate;

(iii) Notice of appropriation to others;

(iv) Compliance with state procedural requirements; and

(v) The application of water to a beneficial use.

Once the appropriation has been established, prior appropriator has the right to exclusive use of the amount of water of his appropriation and all subsequent junior users take subject to his right. The appropriation may be obtained only for beneficial uses, which include domestic, agricultural and industrial uses. It lasts as long as water is beneficially used and is limited to the amount that can be so used.

18.2.2.2 Beneficial Uses

A number of uses of water have been approved as beneficial by courts and legislatures. Domestic use is everywhere recognised such. Cities and towns may appropriate water for municipal purposes. A city may appropriate more water than it presently needs in order to provide for future growth.

18.2.2.3 Quantity Of Water

An appropriation is always stated in terms of the rights to take a definite amount of water. Direct flow rights are stated in terms of the maximum current or flow that may be diverted from the stream. Storage rights are expressed in terms of the total volume of water that may be stored.

An appropriation acquired, by building a reservoir and storing water in it, is measured by the storage capacity of the reservoir, that it will hold as a result of a single filling each year. If the reservoir is to be filled more than one time, it can be done only after paying the compensation for the additional quantity of water stored.

18.2.2.4 Place Of Use

With few exceptions, an appropriation can be made in order to use the water at any place where it is needed. Diversions out of water-shed have been permitted, but not between interstate.

18.2.2.5 Preferences

Preferences are exceptions to the rule of priority. A preference allocates the water to what has been legislatively deemed to be a higher or better use regardless of the time of initiations of use. There is wide variation as to what uses shall be preferred. There is general agreement that man's personal needs come first so that domestic and municipal water supply head every list. A true preference exists when a junior right to a preferred use is placed at the top of the priority list, so that in times when water is short, senior non-preferred rights are cut-off while the preferred uses still draw water. Stated another way, a true preference exists when the preferred use may be initiated without regard to the fact that the supply is already fully appropriated for other purposes. The authorities have to prefer some uses over others when several applications for appropriation of water are pending and the available water is insufficient for all. These preferences should go first for domestic and municipal water supply, then to agriculture, then to power.

18.2.2.6 Changes In Appropriation

A water right is private property and, in most cases, it can be sold or used by its owner at any-place of use, but in the case of diversion type of use, at any time of use, or place of storage also. But the privilege of making such changes is subject to the rule that a change must not injure the vested water rights of the other appropriators. The agencies and courts that regulate the appropriation and distribution of water are given the power to approve or forbid changes on this ground, after proceeding at which all interested parties are represented.

The restriction on changes that cause damage is not merely on application of the rule of priority; it is applicable to any person senior or junior who will suffer as a result of the change. A change from non-consumptive use to consumptive one will obviously injure downstream appropriators. The loss of benefits from return flows is the most common type of damage that will prevent a change, but the appropriator may be permitted to change the place of use or the amount of this consumptive use, though not of his total diversion and other conditions may be imposed to permit a change to as great an extent as possible, and yet prevent infliction of damage.

18.2.2.7 Transfers Of Appropriation

An appropriation is regarded as real property and where it can be sold to a person who will use it at a different place or for different use, the transfer is ordinarily made by a deed. Water rights for the irrigation of land are generally regarded as appurtenant to the land, hence a sale of the land will carry the water right with it, although the water right was not specifically mentioned in the deed.

18.2.2.8 Loss Of Appropriation

An appropriation is a property right and its ownership, like that of land, is held in perpetuity although same may be granted for a limited period. However, it may be terminated if it is not used. It has been recognized that the non-use of water, coupled with an intent not to resume the use, amounts to an "abandonment" that terminates the water right and makes the water available for use and appropriation by others. No particular period of time is required for an abandonment, but long un-explained nonuse will often cause a court to say that the right is abandoned although there is no direct evidence of the intent of the appropriator.

18.2.3 System of Administrative Disposition of Water

The riparian rights doctrine and the prior appropriation systems, as a rule, are appropriate either in humid countries in which there is an abundance of water, or in circumstances in which the government organisation is weak and under-developed. As water becomes scarce, government tends to assume a more active role in the disposition of the available supply. This trend can be plainly seen in arid regions of the world where demand outstrips supply even at a primitive level of economy. When supply exceeds demand there is little need for desire for control; but where demand outgrows supply administrative control intensifies. The administrative authorisation system has become the main feature of the water codes of new countries, such as Israel. These systems envisage authorisation by government for using any water declared public. Usually two kinds of authorisation are given:

(i) A permit which is less permanent and easily revoked; and

(ii) A concession which sets up reciprocal rights and obligations between granter and grantee.

In administrative law, "permits" are distinguished from "concessions", in as much as the former are revocable and create obligations only for the grantee, where as concessions are for a fixed period or perpetual, create reciprocal obligations and their revocation is governed by law. Consequently procedure for obtaining them is different, since a concession has a certain condition of stability which a permit lacks.

18.3 SURFACE WATER

18.3.1 POWER OF LEGISLATION REGARDING WATER

According to the Constitution of India water is in the "State list". Therefore, the States can enact any legislation regarding water that is to say, water supplies, irrigation and canals, drainage, embankments, water storage and water power excepting the regulation and development of inter-state rivers and river vallies. The parliament thus has no legislative competence in the matter.

18.3.2 NATIONAL WATER POLICY

Water is a prime natural resource, a basic human need and precious national asset. Therefore, planning and development of water resources need to be governed by national perspectives.

The Government of India have therefore formulated a National Water Policy in 1987; according to which, in the planning and operation of systems water allocation priorities shall be broadly as follows:

- Drinking
- Irrigation
- Hydro-power
- Navigation
- Industrial and other uses

However these priorities can be modified, if necessary, in particular regions with reference to area specific consideration. The National Water Policy has directed that adequate drinking water facilities should be provided to the entire population both in urban and rural areas by 1991; that irrigation and multipurpose projects should invariably include a drinking water component wherever there is no alternative source of drinking water; and that drinking water needs of human beings and animals should be the first charge on any available water.

In order to provide for use and control by the state, the water of all rivers and streams flowing in natural channels and of all lakes, and to that end to amend and consolidate the existing laws relating to irrigation and drainage and assessment and levy of water rates and betterment contributions, a Model Canal Irrigation and Drainage Bill is being formulated by Union Government for the guidance of the States.

18.4 GROUND WATER

The existing Irrigation Acts or any other Acts do not define the ownership of such surface or ground water which is considered as belonging to the owners of the land. But in view of the vital importance of ground water to the nation; for water supply and irrigation it is essential for government to extend control over it and to provide for the methodical and systematic regulation in conjunctive use with surface water. The National Water Policy has directed that exploitation of ground water resources should be so regulated as not to exceed the recharging possibilities, as also to ensure social equity; and that ground water recharge projects should be developed and implemented for augmenting the available supplies.

The Union Government has prepared and circulated to the State a Model Ground Water (Control and Regulation) Bill to regulate and control the development there with. The salient features of the Bill are as under:

- Ground water has been defined as the water which exists below the surface of the ground at any particular location.
- Ground Water Authority shall be constituted by the State Government.
- The State Government, on a report received from the Ground Water Authority may declare areas as notified areas; where, extraction and use of ground water will be regulated in the public interest.
- Any person desiring to sink a well in the notified area for any purpose other than exclusively domestic use, either on personal or community basis, shall apply to the Ground Water Authority for the grant of a permit for the purpose and shall not proceed with any activity connected with sinking unless a permit has been granted by the Ground Water Authority.
- In granting or refusing a permit the Ground Water Authority shall have regard to:

    (a) the purpose or purposes for which water is to be used;

    (b) the existence of other competitive users;

    (c) the availability of water; and

    (d) any other relevant factor.

- Every existing user of ground water in the notified area, shall apply to the Ground Water Authority for the grant of a certificate of registration recognising his existing use in such forms and in such manner as may be prescribed.
- No person shall himself or by any person on his behalf, carry on the business of sinking wells or any other activity connected with the sinking of wells in any notified area except under and in accordance with a licence granted in this behalf.
- Any person desiring to carry on the business of sinking of wells in the notified area may make an application to the Ground Water Authority for the purpose.
- The Ground Water Authority or any person authorised by it in writing in this behalf shall have power to enter on any property with the right to investigate and make any measurements concerning the land or the water located on the surface or underground, inspect the well, sunk or being sunk, take specimens of such solid, or other materials or of water extracted from such wells, and obtain such information and record as may be required.
- Any user of ground water who contravenes or fails to comply with any of the provision of the Act, will be penalised and/or punished according to the provision of the Act.

18.5 PREVENTION AND CONTROL OF POLLUTION

Though the conservation of available water sources free from pollution is of paramount importance now, even the early law regulating pollution says that the riparian owner may make such reasonable use of the water as he can while it passes his land; but he cannot make such use of water as to pollute it unreasonably or so as to create nuisance. The early law regulating pollution was enforced almost entirely through the process of individual suits for what was termed a private nuisance.

The concept of public nuisance has also been used to some degree to control pollution. A public nuisance is an act which causes inconvenience or damage to the public as distinguished from one or a few individuals and includes any interference with the public health, safety, or inconvenience. Thus the pollution of a stream which merely inconveniences several riparian owners is a private nuisance only, but may become public one, if it kills fish or creates a menace to the health of the community. A public nuisance is subject to abatement at the behest of state officials. It may also constitute a crime.

In our country until recently the pollution was regulated through state factory acts and rules, and also by some sections (section 28) of the Indian Easement Act. As the scope of these acts is limited in its extent and does not provide much guidance in respect of water pollution prevention, the Union Government enacted the Water (Prevention and Control of Pollution) Act, in 1974; which is applicable to all Union territories, and has been adopted by all the States, by resolution passed in that behalf under clause (i) of Article 252 of the Constitution. Under the provision of this Act no discharge of waste water can be made in the environment without obtaining consent from the State Pollution Control Board (from the Central Pollution Control Board, in respect of Union Territories). A consent prescribes the volume and quality of waste water in terms of concentration of various pollutants, which can be permitted for discharge in the environment. In 1986, the Union Government enacted the Environment (Protection) Act, 1986, for protection and improvement of environment, and the prevention of hazards to human beings, other living creatures, plants and properties. The Act empowers the Union Government to make rules providing standards in excess of which environmental pollutants shall not be discharged or emitted in the environment.

APPENDIX A
ABBREVIATIONS AND SYMBOLS

atm	Atmosphere	emf	Electromotive force
BOD	Biochemical oxygen demand	Eq	Equation
ci	Curie	Fig	Figure
°C	Degrees centigrade	g	Gram
cal	Calorie	ha	Hectare
cc	Cubic centimetre	I.D	Internal diameter
CCE	Carbon-chloroform extract	JTU	Jackson turbidity unit
cgs	Centimetre gram second	k cal/kg	Kilocalorie per kilogram
C.I	Cast iron	kg/cm^2	Kilogram per square centimetre
cm	Centimetre	kg/m^2	Kilogram per square metre
cm/min	Centimetres per minute	kL	Kilolitres
cm/sec	Centimetres per second	kLd	Kilolitres per day
cm^2	Square centimetres	km	Kilometre
COD	Chemical oxygen demand	kw	Kilowatt
Col	Column	kwh	Kilowatt hour
cum	Cubic metres	L	Litre
cumec	Cubic metre per second	Lpcd	Litre per capita per day
		lpd	Litres per day
deg	Degree	lph	Litre per hour
DO	Dissolved oxygen	lph/m^2	Litres per hour per square metre
EDTA	Ethylenediaminetetraacetic acid	lpm	Litre per minute

lpm/m^2	Litres per minute per square metre
m	Metre
m^3	Cubic metre
m^3/hr	Cubic metres per hour
me	Milliequivalent
mg	Milligram
mg/l	Milligram per litre
ml	Millilitre
mL	Million litres
mLd or mld	Million litres per day
mm	Millimetre
mps.or m/s	Metre per second
min	Minute
mole	Gram molecular weight
mol wt	Molecular weight
mph	Metres per hour
$m^3/d/m$	Cubic metres per day per metre
$m^3/d/m^2$	Cubic metres per day per square metre of area
m^3/mL	Cubic metres per million litre
MPN	Most probable number
mμ	Millimicron
μ	Micron
μc_i	Microcurie
μg	Microgram
N	Newton
NPSH	Net positive suction head
No	Number
NTU	Naphelometric turbidity units
OTA	Orthotolidine arsenite
N_R	Reynold's number
P	Page
PP	Pages
pCi	Picocurie
ppb	Part per billion
ppm	Part per million
rpm	Revolution per minute
s	Second
sq	Square
Vol	Volume
wt	Weight

APPENDIX B
CONVERSION FACTORS

LENGTH

1 In	=	25.4 mm	1 mm	=	0.0394 in
1 ft	=	0.3048 m	1 cm	=	0.3934 in
					0.0328 ft
1 yd	=	0.9144 m	1 m	=	3.2808 ft
					1.0936 yd
1 mile	=	1.6093 km	1 km	=	0.6214 mile

AREA

1 sq in	=	645.163 sq mm	1 sq mm	=	0.00155 sq in
	=	6.4516 sq cm	1 sq cm	=	0.1550 sq in
1 sq ft	=	0.0929 sq m		=	0.00108 sq ft
1 sq yd	=	0.8361 sq m	1 sq m	=	10.7639 sq ft
1 sq mile	=	2.59 sq km		=	1.1960 sq yd
1 acre	=	0.4047 ha	1 ha	=	2.4710 acre
		4046.86 sq m		=	0.00386 sq mile
			1 sq km	=	0.3861 sq mile
				=	247.105 acre

CAPACITY

1 gal (UK)	=	4.54609 l	1 l	=	0.0353147 cu ft
	=	0.00454609 cum		=	0.001308 cu yd
	=	0.160544 cu ft	1 l	=	0.2200 gal(UK)
1 gal (US)	=	0.00378541 cu m	1 l	=	0.264172 gal(US)
	=	3.78533 l			
	=	0.832675 UK gal			
	=	0.133681 cu ft			

1 US Pint (Liquid)	=	0.4732 l			
1 fluid oz (US)	=	29.5729 ml			
1 fluid oz (UK)	=	28.4123 ml			

VOLUME

1 cu in	=	16.8871 cu cm	1 cu cm	=	0.061024 cu in
1 cu ft	=	0.0283 cu m	1 cu m	=	35. 815 cu ft
1 cu yd	=	0.7646 cu m		=	1.60795 cu yd
1 acre ft	=	1233.48 cu m		=	0.00081071 acre ft

WEIGHT

1 grain	=	0.0648 g	1 g	=	15.45254 grains
1 oz	=	28.3495 g		=	0.0352740 oz
1 lb	=	0.4536 kg	1 kg	=	2.20462 lb
1 ton	=	1.01605 tonnes	1 tonne	=	0.98421 ton

DENSITY

1 lb/ft^3	=	16.0185 kg/m^3 or g/L	Kg/m^3	=	0.0624 lb/ft^3

PRESSURE AND STRESS

1 lb/in^2	=	0.0703 kg/cm^2			
1 lb/ft^2	=	4.88243 kg/m^2	1 kg/cm^2	=	14.223 lb/in^2
1 ton/in^2	=	1.5749 kg/mm^2		=	10 m H_2O
				=	0.96784 atm
1 atm	=	101325.0 N/m^2	1 kg/m^2	=	0.204816 lb/ft^2
	=	760.0 mm Hg	1kg/mm^2	=	0.6850 ton/in^2
	=	1.01325 bar	1 atm	=	68087.0 pdl/ft^2

	=	14.6959 lbf/in^2			(Where 1 pdl = 0.138255N)
	=	33.8984 ft H_2O			
	=	29.9213 in Hg			
	=	10332.2 kg/m^2			
	=	1.03322 kg/cm^2			
	=	10.3322 m H_2O	1 mm Hg	=	2.78450 Lb/ft^2

FORCE

1 lbf	=	4.44822 N	1N (or 10^5 dynes)	=	0.101972 kgf
	=	0.453592 kgf		=	0.224809 lbf
1 tonf	=	9.96402 kN			
1 pdl	=	0.138255 N	1 kgf	=	2.20462 lbf

g(acceleration due to gravity)	=	32.1740	ft/sec^2
	=	980.665	cm/sec^2

ENERGY AND POWER

1horse-power	=	0.745700 kW	1kW	=	1.34102 Horse-power
1 ft.lb f/s	=	1.35582 W	1kWh	=	3.6 MJ
1 b.t.u	=	1.05506 KJ	1 J	=	0.737562 ft lbf
1 therm	=	105.506 MJ	1 kJ	=	0.277778 Wh
1 ft lbf	=	1.35582 J			

VELOCITY

1 fps	=	0.0348 m/s	1 m/s	=	3.2808 fps
	=	1.0973 km/h		=	2.2369 mile/h
1 mile/h	=	0.4470 m/s	1 km/h	=	0.9113 fps
	=	1.6093 km/h		=	0.6214 mile/h

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TREATMENT LOADING RATES

1 in/h	=	0.00705555 mm/s	1 mm/s	=	141.732 in/h
1 UK gal/ft^2/h	=	0.0135927 mm/s		=	73.5689 UK/gal/ft^2/h
1 UK gal/ft^2/h	=	1.17441 m^3/m^2/d		=	76.9130 Million UK gal/acre/d
1 million UK gal/acre/d	=	0.0130016 mm/s	1m^3/m^2/d	=	0.851491 UK gal/ft^2/h
	=	1.12336 m^3/m^2/d		=	0.890187 million UK gal/acre/d
1 UK gal/day/ft	=	14.915 lpd/m	1m^3/day/m	=	67.466 UK/gal/day/ft
	=	0.014915 m^3/day/m			
1 ft^3/s/1000 acres	=	6.99724 l/s/km^2	l/s/km^2	=	0.142915 ft^3/s/1,000 acres
1 ft^2/s/mile^2	=	10.9332 l/s/km^2		=	0.0914645 ft^3/s/mile^2

HARDNESS

mg/l $CaCO_3$	Grains per UK gal $CaCO_3$ (Clark scale-British degrees)	Grains per US gal $CaCO_3$ (American degrees)	Parts per 100,000 $CaCO_3$ (French degrees)	Parts per 100,000 CaO (German degrees)	Parts per million Ca (Russian degrees)
1.00	0.07	0.058	0.10	0.056	0.40
14.29	1.00	0.83	1.43	0.80	5.72
17.15	1.20	1.00	1.72	0.96	8.86
10.00	0.70	0.58	1.00	0.56	4.00
17.86	1.25	1.04	1.79	1.00	7.14
2.57	0.18	0.15	0.26	0.14	1.03

APPENDIX C

LIST OF INDIAN STANDARDS RELATING TO WATER SUPPLY

Sl No.	IS No.	Title
I. GENERAL		
1.	SP 7 (Part 9 Selection 1): 1983	National building code of India 1983 Part 9 Plumbing services: Section 1:Water Supply
2.	SP 35: 1987	Handbook on water supply and drainage with special emphasis on plumbing
3.	1172: 1983	Code of basic requirements for water supply drainage and sanitation (third revision)
4.	2065:1983	Code of practice for water supply in buildings(second revision)
5.	456:1978	Code of practice for plain and reinforced concrete (third revision)
6.	457:1957	Code of practice for general construction of plain and reinforced concrete for dams and other massive structures.
7.	1343:1980	Code of practice for prestressed concrete (first revision).
8.	3103:1975	Code of practice for industrial ventilation.
9.	3370	Code of practice for concrete structure for the storage of liquids.
(a)	Part 1 : 1965	General requirements.
(b)	Part 2 : 1965	Reinforced concrete structures.
(c)	Part 3 : 1967	Prestressed concrete structures.
(d)	Part 4 : 1967	Design tables.
10.	6518 : 1972	Code of practice for control of sediment in Reservoirs.
11.	5330 : 1984	Criteria for design of anchor block for penstocks with expansion joints (first revision).

Sl No.	IS No.	Title
12.	6748 :	Recommendations for watershed management relating to soil conservation.
(a)	Part 1 : 1973	Agronomic aspects.
13.	7357 : 1974	Code of practice for structural design of surge tanks.
14.	3913 : 1966	Suspended sediment load samplers.
15.	3917 : 1966	Scoop type bed material samplers.
16.	4890 : 1968	Methods for measurement of suspended sediment in open channels.
17.	4926 : 1976	Ready-mixed concrete (first revision).
18.	6295 : 1986	Code of practice for water supply and drainage in high altitudes and/or sub-zero temperature regions(first revision).
19.	4880	Code of practice for design of tunnels conveying water.
(a)	Part 1 : 1975	General design.
(b)	Part 2 : 1976	Geometric design(first revision).
(c)	Part 3 : 1976	Hydraulic design(first revision).
(d)	Part 4 : 1971	Structural design of concrete lining in rock.
(e)	Part 5 : 1972	Structural design of concrete lining in soft strata and soils.
(f)	Part 6 : 1971	Tunnel support
20.	5477	Methods for fixing the capacities of reservoirs
(a)	Part 1 : 1969	General requirements
(b)	Part 2 : 1969	Dead storage
(c)	Part 3 : 1969	Live storage
(d)	Part 4 : 1971	Flood Storage
21.	9668 : 1980	Code of practice for provision and maintenance of water supply for fire fighting
22.	8062	Code of practice for cathodic protection for steel structures

Sl No.	IS No.	Title
(a)	Part 1 : 1976	General principles
(b)	Part 2 : 1976	Underground pipelines
23.	10221: 1982	Code of practice for coating and wrapping of underground steel pipelines
24.	12183 : 1987	Code of practice for plumbing in multi-storeyed buildings Part 1 water supply

II. Pipe And Pipe Laying

Cast Iron

1	1536 : 1976	Centrifugally cast(spun) iron pressure pipes for water, gas and sewage(second revision)
2	1537 : 1976	Vertically cast iron pressure pipes for water, gas and sewage (first revision)
3	1538 (Parts 1 to 24)	Cast iron fittings for pressure pipes for water, gas and sewage(second revision)
(a)	Part 1 : 1976	General requirements
(b)	Part 2 : 1976	Specific requirements for sockets and spigots of pipes
(c)	Part 3 : 1976	Specific requirements for sockets of fittings
(d)	Part 4 : 1976	Specific requirements for flanges of pipes and fittings
(e)	Part 5 : 1976	Specific requirements for raised flanges
(f)	6 : 1976	Specific requirements for standard flange drilling of flanged pipes and fittings
(g)	Part 7 : 1976	Specific requirements for flanged sockets
(h)	Part 8 : 1976	Specific requirements for flanged spigots
(j)	Part 9 : 1976	Specific requirements for double socket bends
(k)	Part 10 : 1976	Specific requirements for double socket bends
(l)	Part 11 : 1976	FIC REQUIREMENTS FOR TEES, ALL SOCKETS
(m)	Part 12 : 1976	Specific requirements for double socket tee with flanged branch
(n)	Part 13 : 1976	Specific requirements for crosses, all sockets
(o)	Part 14 : 1976	Specific requirements for double socket tapers (third revision)
(p)	Part 15 : 1976	Specific requirements for caps
(q)	Part 16 : 1976	Specific requirements for plugs
(r)	Part 17 : 1976	Specific requirements for bell mouth pieces
(s)	Part 18 : 1976	Specific requirements for double flanged bends

Sl No.	IS No.	Title
(t)	Part 19 : 1976	Specific requirements for all flanged tees
(u)	Part 20 : 1976	Specific requirements for all flanged crosses
(v)	Part 21 : 1976	Specific requirements for double flanged taper
(w)	Part 22 : 1976	Specific requirements for split puddle or body flanges
(y)	Part 23 : 1976	Specific requirements for blank flanges
(z)	Part 24 : 1984	Specific requirements for all flanged radial tees(second revision)
4.	1879 : 1975: Pipe Part 1 to 10	malleable cast iron pipe fittings (first revision)
5.	3114 : 1985	Code of practice for laying of cast iron pipes (third revision)
6.	782 : 1978	Caulking lead(third revision)
7.	6163 : 1978	Centrifugally cast(spun) iron low pressure pipes for water, gas and sewage(first revision)
8.	7181 : 1986	Horizontally cast iron double flanged pipes for water, gas and sewage(first revision)
9.	8329 : 1977	Centrifugally cast (spun) ductile iron pressure pipes for water, gas and sewage
10.	9523 : 1980	Ductile iron fittings for pressure pipes for water, gas and sewage
11.	11606 : 1986	Methods of sampling cast iron pipes and fittings
12.	11906 : 1986	Recommendations for cement mortar lining cast iron, mild steel and ductile iron pipes and fittings for transportation of water .
13.	12288 : 1987	Code of practice for laying of ductile iron pipes
CONCRETE		
14.	458 : 1971	Concrete pipes(with and without reinforcements) (second revision)
15.	784 : 1978	Pre-stressed concrete pipes(including fittings)(first revision)
16.	1916 : 1963	Steel cylinder reinforced concrete pipes
17.	3597 : 1985	Methods of test for concrete pipes(first revision)
18.	783 : 1985	Code of practice for laying of concrete pipes(first revision)
19.	4350 : 1967	Concrete porous pipes for under drainage.

Sl No.	IS No.	Title
ASBESTOS CEMENT PIPES		
20.	1592 : 1980	Asbestos cement pressure pipes(second revision)
21.	6530 : 1972	Code of practice for laying of asbestos cement pressure pipes
22.	5531 : 1977	Cast iron specials for asbestos cement pressure pipes for water, gas and sewage(first revision)
23	9627 : 1980	Asbestos cement pressure pipes(light duty)
MILD STEEL TUBES AND PIPES		
24.	1239	Mild Steel tubes, tubulars and other wrought steel fittings
(a)	Part 1: 1979	Mild Steel tubes (fourth revision)
(b)	Part 2 : 1982	Mild steel tubulars and other wrought steel pipe fittings (third revision)
25.	1978 : 1982	Line pipe
26.	3589 : 1981	Electrically welded steel pipes for water, gas and sewage (150 to 2000 mm nominal size) (first revision)
27.	4270 :1983	Steel tubes used for water wells (first revision)
28.	4516 : 1968	Elliptical mild steel tubes
29	5504 : 1969	Spiral welded pipes.
30.	5822 : 1986	Code of practice for laying of welded steel pipes for water supply (first revision)
31.	4711 : 1974	Method for sampling of steel pipes, tubes and fittings(first revision)
32.	4736 : 1986	Hot-dip zinc coatings on mild steel tubes (first revision)
33.	6286 : 1971	Seamless and welded steel pipes for sub zero temperature services.
34.	6631 : 1972	Steel pipes for hydraulic purposes .
35.	11722 : 1986	Thin welded flexible quick coupling pipes.

Sl No.	IS No.	Title
PLASTIC PIPES		
36.	3076 : 1985	Low density polyethylene pipes for potable water supplies(second revision)
37.	4984 : 1987	High density polyethylene pipes for potable water supplies, sewage and industrial effluents(third revision)
38.	4985 : 1988	Unplasticized PVC pipes for potable water supplies (second revision)
39	12818 : 1989	UPVC ribbed and casing pipes for potable water supply.
40	7634	Code of practice for plastic pipe work for potable water supplies.
(a)	Part 1: 1975	Choice of materials and general recommendation
(b)	Part 2: 1975	Laying and jointing polyethylene (PE) pipes.
(c)	Part 3: 1975	Laying and jointing of unplasticized PVC pipes.
41.	7834	Injection moulded PVC fittings with solvent cement joints for water supplies.
(a)	Part 1 : 1975	General requirements
(b)	Part 2 : 1975	Specific requirements of 45° elbows
(c)	Part 3 : 1975	Specific requirements for 90° elbows
(d)	Part 4 : 1975	Specific requirements for 90° tees.
(e)	Part 5 : 1975	Specific requirements for 45° tees
(f)	Part 6 : 1975	Specific requirements for sockets.
(g)	Part 7 : 1975	Specific requirements for unions.
(h)	Part 8 : 1975	Specific requirements for caps.
42.	8008	Injection moulded HDPE fittings for potable water supplies.
(a)	Part 1 : 1976	General requirements.
(b)	Part 2 : 1976	Specific requirements for 90° bends

Sl No.	IS No.	Title
(c)	Part 3 : 1976	Specific requirements for 90° tees
(d)	Part 4 : 1976	Specific requirements for reducers
(e)	Part 5 : 1976	Specific requirements for ferrule
(f)	Part 6 : 1976	Specific requirements for pipe ends
(g)	Part 7 : 1976	Specific requirements for sandwich flange
43.	8360	Fabricated high density polyethylene (HDPE) fittings for potable water supplies.
(a)	Part 1 : 1977	General requirements.
(b)	Part 2 : 1977	Specific requirements for 90° tees
(c)	Part 3 : 1977	Specific requirements for 90° bends
44.	10124	Fabricated PVC fittings for potable water supplies
(a)	Part 1 : 1988	General requirements.
(b)	Part 2 : 1988	Specific requirements for sockets(first revision)
(c)	Part 3 : 1988	Specific requirements for straight reducers (first revision)
(d)	Part 4 : 1988	Specific requirements for caps (first revision)
(e)	Part 5 : 1988	Specific requirements for equal tees (first revision)
(f)	Part 6 : 1988	Specific requirements for flanged into pieces with metallic flanges (first revision)
(g)	Part 7 : 1988	Specific requirements for threaded adaptors (first revision)
(h)	Part 8 : 1988	Specific requirements for 90 degree bends (first revision)
(j)	Part 9 : 1988	Specific requirements for 60 degree bends (first revision)
(k)	Part 10 : 1988	Specific requirements for 45 degree bends (first revision)
(m)	Part 11 : 1988	Specific requirements for 30 degree bends (first revision)

Sl No.	IS No.	Title
(n)	Part 12 : 1988	Specific requirements for 22 1/2 degree bends (first revision)
(o)	Part 13 : 1988	Specific requirements for 11 ¼ degree bends (first revision)
44.	12231 : 1988	UPVC pipes for use in suction and delivery lines of agriculture pump.
45.	12235	Methods of test for unplasticized PVC pipes for potable water supplies.
(a)	Part 1 : 1986	Methods for measurement of outside diameter.
(b)	Part 2 : 1986	Measurement of wall thickness.
(c)	Part 3 : 1986	Test for opacity.
(d)	Part 4 : 1986	Determining the detrimental effect on the composition of water.
(e)	Part 5 : 1986	Reversion test
(f)	Part 6 : 1986	Stress relief test
(g)	Part 7 : 1986	Test for resistance of sulphuric acid
(h)	Part 8 : 1986	Internal hydrostatic pressure test
(i)	Part 9 : 1986	Impact strength test
(j)	Part 10 : 1986	Method for determination of organizing as tin aqueous solution.
(k)	Part 11 : 1986	Extractability of cadmium and mercury occuring as impurities.
46.	12709 : 1989	Specification for glass fibre reinforced plastic (GRP) pipes for water supply and sewerage.
MISCELLANEOUS PIPES		
47.	1545 : 1982	Solid drawn copper alloy tubes for condensers and heat exchanger (second revision).
48.	407 : 1981	Brass tubes for general purposes (third revision).
49.	404	Lead pipes

Sl No.	IS No.	Title
(a)	Part 1 : 1977	For other than chemical purposes (second revision).
(b)	Part 2 : 1979	For chemical purposes (second revision).
50.	11906 : 1986	Recommendations for cement - mortar lining for cast iron, mild steel and ductile-iron pipes and fittings for transportation of water.

III. WATER FITTINGS

TAPS

1.	781 : 1984	Cast copper alloy screw drawn bib taps and stop valves for water services (third revision).
2.	1700 : 1973	Drinking fountains (first revision).
3.	1711 : 1984	Self-closing taps for water supply purposes (second revision).
4.	1795 : 1982	Pillar taps for water supply purposes (second revision).
5.	4346 : 1982	Washers for use with fittings for water services (first revision).
6.	8934 : 1978	Cast copper alloy fancy pillar taps for water services.
7.	9763 : 1981	Plastic bib taps and stop valves (rising spindle) for cold water services.

WATER METERS

8.	779 : 1978	Water meters (domestic type) (fifth revision).
9.	2104 :1981	Water meter boxes (domestic type) (first revision).
10.	2373 :1981	Water meter (bulk type) (third revision).
11.	2401 : 1973	Code of practice for selection, installation and maintenance of domestic water meters (first revision).
12.	6784 : 1984	Method for performance testing of water meters (domestic type)(first revision).

Sl No.	IS No.	Title
VALVES		
13.	780 :1984	Sluice valves for water works purposes (50 to 300 mm size) (sixth revision).
14.	2906 : 1984	Sluice valves for water works purposes (350 to 1200 mm size) (third revision).
15.	2685 : 1971	Code of practice for selection, installation and maintenance of sluice valves (first revision).
16.	3042 : 1965	Single faced sluice gates (200 to 1200 mm size)
17.	3950 : 1979	Surface boxes for sluice valves (first revision).
18.	778 : 1984	Copper alloy gate, globe and check valves for water works purposes (fourth revision).
19.	1701 : 1960	Mixing valves for ablutionary and domestic purposes.
20.	1703 : 1977	Ball valves (horizontal plunger type) including floats for water supply purposes (second revision).
21.	4838 : 1986	Foot valves for water works purposes (second revision).
22.	5312	Swing check type reflux (non return) valves for water works purposes
	5312(Part 1) 1984	Single door pattern (first revision).
	5312 (Part 2) 1986	Multi door pattern
23.	9338 : 1984	Cast iron screw down stop valves and stop and check valves for water works purposes(first revision.)
24.	9739 : 1981	Pressure reducing valves for domestic water supply systems.
25.	12234 : 1988	Equilibrium plastic float valve for cold water services.
MISCELLANEOUS FITTINGS		
26.	2692 : 1978	Ferrules for water services (first revision).

Sl No.	IS No.	Title
27.	3004 : 1979	Plug cocks for water supply purposes (first revision).
28.	9762 : 1981	Polyethylene floats for ball valves.
29.	10446 : 1983	Glossary of terms relating to water supply and sanitation.

IV TUBEWELLS PUMPS AND PRIME MOVERS

GLOSSARY

1.	IS 9439 : 1980	Glossary of terms used in waterwell drilling technology
2.	Codes of practice	IS 2800 : 1979 Code of practice for construction and testing of tubewells (a) Part I construction (first revision) (b) Part II Testing (first revision)
3.	IS 11189 : 1985	Methods for tube-well development
4.	IS 11632 : 1986	Code of practice for rehabilitation of tubewell
TUBEWELL COMPONENTS		
5.	IS 4097 : 1967	Gravel for use as pack in tubewells
6.	IS 4270 : 1983	Steel tubes used for water wells(first revision)
7	IS 8110 : 1983	well screens and slotted pipes (first revision)
DRILLING EQUIPMENT, ACCESSORIES AND METHODS		
8.	IS 7156 : 1974	General requirements for reverse circulation drilling rigs
9.	IS 7206 : 1974	General requirements for straight rotary drilling rigs
10.	IS 7209 : 1974	General requirements for blast hold drilling rigs
11.	IS 8986 : 1978	Dimensions for drill steels in bar form for percussive drilling

12.	IS 9026 : 1978	Rope threaded percussive long hole drilling equipment
13.	IS 11180 : 1985	Keeleys for direct rotary drilling
14.	IS 11312 :	External upset drill pipe assemblies for use in water well drilling
a)	Part 1- 1986	Screwed on joints drill pipe size
15.	IS 11672 : 1986	Tungsten carbide buttons and insets for use in down the hole (DTH) bits.
16.	IS 11710 : 1986	Code of practice for selection and design of diamond core drills.
17.	IS 11830 :	General requirements for down-the-hole hammer rigs for water wells.
a)	Part 1- 1986	Hydraulic rigs
18.	IS 12097 : 1987	Classification and selection of drilling rigs for water well drilling.
19.	IS 12194 : 1987	Dimensions for rock roller bits and blade drag bits for rock drilling equipment.
PUMPS AND RELATED STANDARDS		
20.	IS 8035 : 1976	Shallow well hand pumps
21.	IS 9301 : 1984	Deepwell hand pumps (second revision)
22.	IS 11004 : 1985	Code of practice for installation and maintenance of deep well hand pumps.
a).	Part 1	Installation
b).	Part 2	Maintenance
OTHER PUMPS		
23.	IS 1520 : 1980	Horizontal centrifugal pumps for clear, cold, fresh water (second revision)
24.	IS 1710 : 1972	Vertical turbine pumps for clear , cold, fresh water (first revision)
25.	IS 6595 : 1980	Horizontal centrifugal pumps for clear, cold, fresh water for agricultural purposes(first revision)

26.	IS 8034 : 1976	Submersible pump sets for clear, cold, fresh water.
27.	IS 8418 : 1977	Horizontal centrifugal self priming pumps.
28.	IS 8472 : 1977	Regenerative self priming pumps for clear, cold, fresh water.
29.	IS 9079 : 1979	monoset pumps for clear ,cold, fresh water for agricultural purposes
30.	IS 9137 : 1978	Code for acceptance test for centrifugal mixed flow and axial pumps – Class C
31.	IS 9542 : 1980	Horizontal centrifugal monoset pumps for cold, fresh water.
32.	IS 9694	Code of practice for selection, installation, operation and maintenance for horizontal centrifugal pumps for agricultural applications.
(a)	Part 1-1980	Selection
(b)	Part 2-1980	Installation
(c)	Part 3-1980	Operation
(d)	Part 4-1980	Maintenance
33.	IS 10572 : 1983	Methods of sampling pumps .
34.	IS 10804 : 1986	Recommended pumping systems for agricultural purposes (first revision)
35.	IS 10805 : 1986	Foot valves, reflux valves or non-return valves and bore valves to be used in suction lines of agricultural pumping systems (first revision)
36.	IS 10981 : 1983	Code for acceptance test for centrifugal mixed flow and axial pumps - Class B
37.	IS 11346 : 1985	Testing set up for agricultural pumps
38.	IS 12225 : 1987	Technical requirements for jet, centrifugal pump combination
39.	IS 5120 : 1977	Technical requirements for rotodynamic special purpose pumps
PRIME MOVERS		
40.	IS 325 : 1978	Three-phase induction motors

41.	IS 900 : 1965	Code of practice for installation and maintenance of induction motors
42.	IS 996 : 1979	Single-phase small A.C. and universal electric motors (second revision)
43.	IS 4029- 1967	Guide for testing three phase induction motors
44.	IS 7538- 1975	Three phase squirrel cage induction motors for centrifugal pumps for agricultural application
45.	IS 8789 : 1978	Values of performance characteristics for three phase induction motors
46.	IS 9283 : 1979	Motors for submersible pumpsets
47.	IS 10001 :1981	Performance requirement for constant speed compression ignition (diesel) engines for general purposes (up to 20 Kw)
48.	IS 11170 :1985	Performance requirement for constant speed compression ignition (diesel) engines for agricultural purposes (up to 20 Kw)
49.	IS 11501 : 1986	Engine monoset pumps for clear, cold, fresh water for agricultural purposes
50.	IS 10808 : 1984	Code of practice for installation, operation and maintenance of hydraulic rams
51.	IS 10809 : 1984	Hydraulic rams
52.	IS 11390 : 1985	Test code for hydraulic rams

V WATER QUALITY

1.	IS 258 : 1967	Potash alum (first revision)
2.	IS 259 : 1969	Aluminum alum (first revision)
3.	IS 260 : 1969	Aluminum sulphate
4.	IS 299: 1980	Aluminum ferric (third revision)
5.	IS 646 : 1986	Liquid chlorine (second revision)
6.	IS 1065 : 1971	Bleaching powder, stable
7.	IS 1622 : 1981	Methods of sampling and microbiological examination of water (first revision)
8.	IS 3025 :	Methods of sampling and test (physical and chemical) for water and waste water.

Part 1 - 1986	Sampling (first revision)
Part 3 -1987	Precision and accuracy
Part 4 - 1983	Colour (first revision)
Part 5 - 1983	Odour (first revision)
Part 6 - 1983	Odour threshold (first revision)
Part 7 - 1984	Taste threshold (first revision)
Part 8 - 1984	Taste rating (first revision)
Part 9 - 1984	Temperature (first revision)
Part 10 - 1984	Turbidity (first revision)
Part 11 - 1983	pH value(first revision)
Part 12 - 1983	Density (first revision)
Part 13	1983 Saturation index (with respect to calcium carbonate) (first revision)
Part 14-1984	Specific conductance (wheat-stone bridge conductance cell) (first revision)
Part 15 - 1984	Total residue (total dissolved and suspended)
Part 16 - 1984	Filterable residue (total dissolved solids) (first revision)
Part 17 - 1984	Nonfilterable residue (total dissolved solids) (first revision)
Part 18 - 1984	Volatile and fixed residue (total filterable and non filterable) (first revision)
Part 19 - 1984	Settleable matter (first revision)
Part 20-1984	Dispersion characteristics (flow patterns) (first revision)
Part 21 – 1983	Total hardness (first revision)
Part 22 – 1986	Acidity (first revision).

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	Part 23 – 1986	Alkalinity (first revision)
	Part 24 – 1986	Sulphates (first revision)
	Part 25 – 1986	Chlorine demand (first revision)
	Part 26 – 1986	Chlorine, residual (first revision)
	Part 27 – 1986	Cyanide (first revision)
	Part 28 – 1986	Sulphate (first revision)
	Part 29 – 1986	Sulphide (first revision)
	Part 30 – 1988	Bromide
	Part 31 –1988	Phosphorus
	Part 32 –1988	Chloride
	Part 33 –1988	Iodide
	Part 34 –1988	Nitrogen
	Part 35 –1988	Silica
	Part 36 –1988	Ozone residual
	Part 37 –1988	Arsenic
9.	IS 9825-1981	Chlorine tablets
10.	IS 10500-1983	Drinking
11.	IS 10553	Requirements for chlorination equipment
	Part 1-1983	General guidelines for chlorination plants including handling, storage and safety of chlorine cylinders and drums
	Part 2-1983	Vacuum feed type chlorinators
	Part 4-1983	Gravity feed type gaseous chlorinators
	Part 5-1987	Bleaching powder solution feeder displacement type chlorinator

VI MEASUREMENT OF FLUID FLOW

1.	IS 1191-1971	Glossary of terms and symbols used in connection with the measurement of liquid flow with a free surface(first revision)
2	IS 1192-1981	Velocity area methods for measurement of flow of water in open channels.
3.	IS 1194-1960	Forms for recording measurement of flow of water in open channels

4.	IS 2912-1964	Recommendations for liquid flow measurement in open channels by slope area method(approximate method)(Amendment No. 1)
5.	IS 2913 1964	Recommendation for determination of flow in tidal channels.
6.	IS 2914-1964	Recommendation for estimation of discharge by establishing stage-discharge relation in open channels. (Amendment No. 1)
7.	IS 2915-1964	Instructions for collection of data for the determination, of the flow by velocity area methods
8.	IS 2951-1965	Recommendation for estimation of flow of liquids in closed conduits.
	Part 1-1965	Head loss in straight pipes due to friction resistance
	Part 2-1965	Head loss in valves and fittings
9.	IS 2952-1964	Recommendation for methods of measurement of liquid flow by means of orifice plates and nozzles
	Part1 : 1964	Incompressible fluids
	Part2 : 1975	Compressible fluids
10.	IS 3910:1966	Specification for current meters (cup type) for water flow measurement (Amendment No. 1)
11.	IS 3911:1966	Specification for surface floats
12.	IS 3912:1966	Specification for sounding rods
13.	IS 3918:1966	Code of practice for use of current meter (cup type) for water flow measurement
14.	IS 4073:1967	Specification for fish weights
15.	IS 4080:1967	Specification for vertical staff gauges
16.	IS 4477:	Methods of measurement of fluid flow by means of venturi meters:
	(Part 1):1967	Part 1 Liquids
17.	IS 4477:	Methods of measurement of fluid flow by means of venturi meters:
	(Part 2):1975	Part 2 Compressible fluids
18.	IS 4858 : 1968	Specification for velocity rods

19.	IS 6059 : 1971	Recommendation for liquid flow measurement in open channels by weirs and flumes – Weirs of finite crest width for free discharge.
20.	IS 6062 : 1971	Method of measurement of flow of water in open channels using standing wave flume – fall
21.	IS 6063 : 1971	Method of measurement of flow of water in open channels using standing wave flume
22.	IS 6064 : 1971	Specification for sounding and suspension equipment
23.	IS 6330 : 1971	Recommendation for liquid flow measurement in open channels by weirs and flumes – end depth method for estimation of flow in rectangular channels with a free overall(approximate method)
24.	IS 6339 : 1971	Methods of analysis of concentration, particle size distribution and specific gravity of sediment in streams and canals
25.	IS 9108 : 1979	Liquid flow measurement in open channels using thin plate weirs
26.	IS 9115 : 1979	Method for estimation of incompressible fluid flow in closed conduits by bend meters
27.	IS 9116 : 1979	Specification for water stage recorder (float type)
28.	IS 9117 : 1979	Recommendation for liquid flow measurement in open channels by weirs and flumes – end depth method for estimation of flow in non Rectangular channels with a free over fall(approximate method)
29.	IS 9118 : 1979	Method for measurement of pressure by means of manometer
30.	IS 9119 : 1979	Method for flow estimation by jet characteristics (approximate method)
31.	IS 9163 (Part 1):	1979 Dilution Methods for measurement of steady flow constant rate injection method
32.	IS 9922 : 1981	Guide for selection of method for measuring flow in open channels.

APPENDIX 2.1

ESTIMATION OF FUTURE POPULATION

PROBLEM

The population of a town as per the Census records are given below for the years 1921 to 1981. Assuming that the scheme of water supply will commence to function from 1986, it is required to estimate the population 30 years hence, i.e. in 2016 and also the intermediate population 15 years after 1986, i.e. 2001.

Year	Population	Increment
1921	40,185	
1931	44,522	4,337
1941	60,395	15,873
1951	75,614	15,219
1961	98,886	23,272
1971	124,230	25,344
1981	158,800	34,570
	Total	118,615
	Average	19,769

SOLUTION

1. Arithmetical Progression Method

Increase in population from 1921 to 1981

i.e. in 6 decades = 1,58,800
- 40,185

1,18,615

or increase per decade = 1/6 x 118,615
= 19,769

Population in 2001 = Population in 1981 + Increase for 2 decades

	=	158,800 + 2 x 19,769
	=	158,800 + 39,538
	=	198,338
Population in 2016	=	Population in 1981 + Increase for 3.5 decades
	=	158,800 + 3.5 x 19,769
	=	227,992

2. Geometrical Progression Method

Rate of growth (r) per decade between 1931 and 1921	= 4,337/ 40,185	=	0.108
1941 and 1931	= 15,873/44,522	=	0.356
1951 and 1941	= 15,219/60,395	=	0.252
1961 and 1951	= 23,272/75,614	=	0.308
1971 and 1961	= 25,344/98,886	=	0.256
1981 and 1971	= 34,570/1,24,230	=	0.278

Geometric mean, $r_g = \sqrt[6]{0.108 \times 0.356 \times 0.252 \times 0.308 \times 0.256 \times 0.278}$

Assuming that the future growth follows the geometric mean for the period 1921 to 1981 r_g = 0.2442

Population in 2001 = Population in 1981 × $(1 + r_g)^2$

$$= 158{,}800 \times (1.2442)^2$$

$$= 2{,}45{,}800$$

$$\text{Population in 2016} = \text{Population in 1981} \times (1 + r_g)^{3.5}$$

$$= 1{,}58{,}800 \times (1.2442)^{3.5} = 3{,}05{,}700$$

3. Method of Varying Increment or Incremental Increase Method

In this method a progressively decreasing or increasing rather than a constant rate is adopted. This is a modification over the Arithmetical Progression method.

Year	Population	Increase X		Increamental Increase Y
1921	40,185			
1931	44,522	4337		
1941	60,395	15,873	+	11,536
1951	75,614	15,219	-	654
1961	98,886	23,272	+	8,053
1971	124,230	25,344	+	2,072
1981	158,800	34,570	+	9,226
	Total	118,615		30,233

$$\text{Average} = 1/6 \times 118{,}615 = 1/5 \times 30{,}233$$

$$= 19{,}769 = 6{,}047$$

$$P = P + nX + \frac{n(n+1)Y}{2}$$

$$P2001 = P1981 + 2 \times 19769 + \frac{2 \times 3 \times 6047}{2}$$

$$= 1,58,800 + 39,538 + 18,141$$
$$= 2,16,479$$

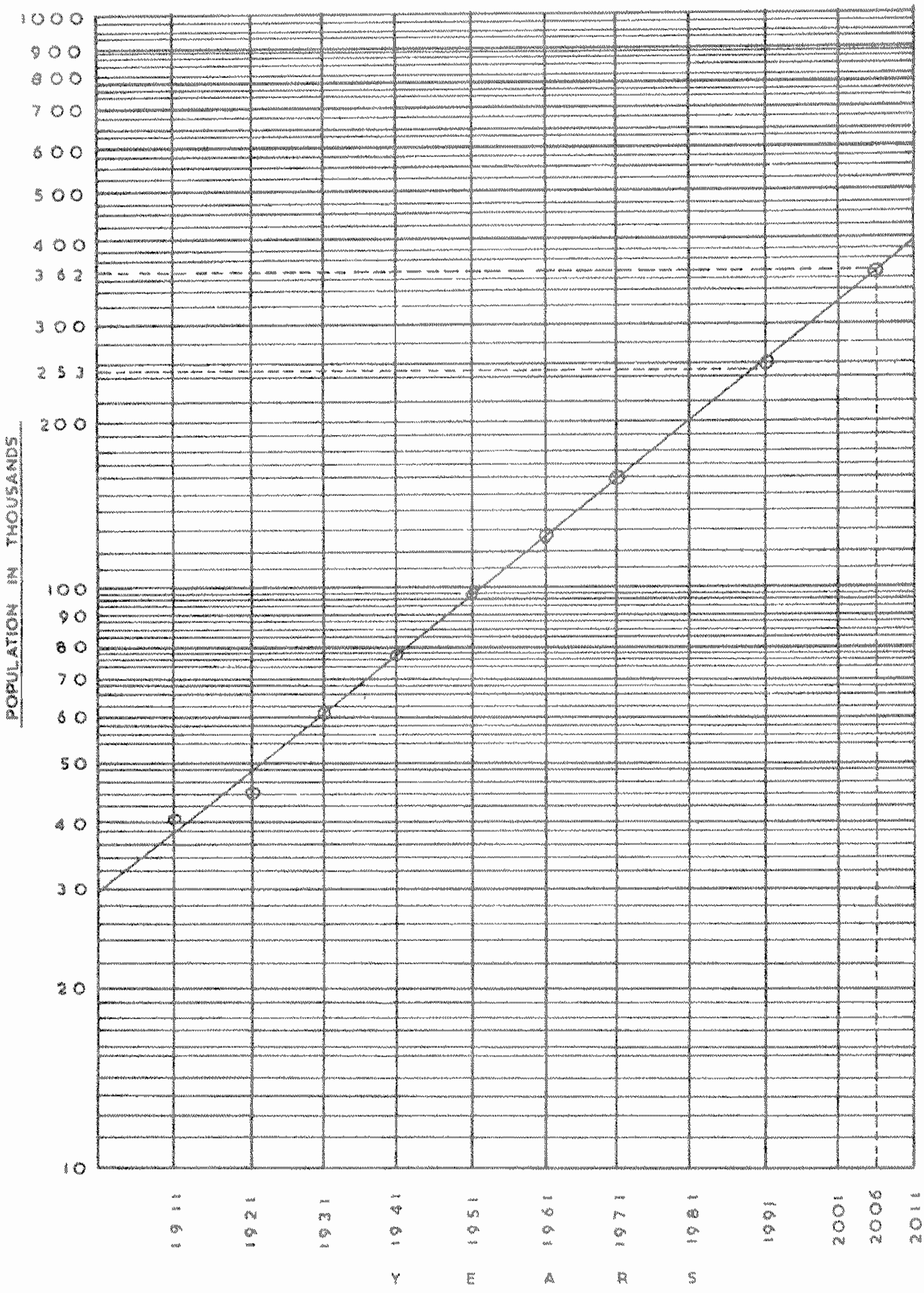
$$P_{2016} = P_{1981} + 3.5 \times 19769 + (3.5 \times 4.5 \times 6047)/2$$
$$= 1,58,800 + 69,192 + 24,188$$
$$= 2,52180$$

4. Graphical Projection Method

From the graph presented on the following page, the figures for 2001 and 2016 years obtained are as follows:

2001 - 253000

2016 - 362000.



SEMI LOG GRAPH FOR ESTIMATION OF FUTURE POPULATION

APPENDIX 3.1

CPM NETWORK DIAGRAM FOR A TYPICAL WATER SUPPLY AUGMENTATION SCHEME

A. PARTICULARS OF THE SCHEME

The present water supply scheme supplies 41.5 mLd for a 1971 census population of 258971.To meet the growing needs of both drinking and industrial requirements, a scheme to provide 78.4 mLd in the immediate stage of 1986 and 106.8 mLd in the ultimate stage of 2001 has been envisaged to meet the perspective daily requirement of water at the rate of 180 lpcd, apart from the industrial demand and the requirement of some villages *en route*. The perspective population in the immediate stage of 1986 and ultimate stage of 2001 are assumed as 3,50,000 and 5,00,000 respectively.

B. ASSUMPTIONS

(i) Land Acquisition

(a) Acquisition proceedings are in progress for the private land required for the headworks.The 68 weeks period prior to execution indicated in the diagram is expected to be adequate before actual construction could be started.

(b) Though the military authorities have claimed the ownership of the government land required for treatment works, clear water sump and pump-house and the elevated service reservoir at the treatment works, the revenue and Municipal Corporation authorities are of the opinion that this is government land and are confident to make it available to the department earlier than the lag period of 84 weeks provided in the diagram before the commencement of the treatment works.

(ii) Tenders

Combined tenders will be invited for providing and installing the raw water and clear water pumping machinery including provision of C.I. pipes , specials and valves for suction and delivery connections and gantry girders, etc. to obviate the delay in the procurement of machinery through the Central Stores Purchasing Organization. No rigid delivery period is generally specified in supply order placed for C.I. pipes, specials and valves against the rate contract though it is mentioned that delivery be effected as early as possible and the delivery time is invariably more than a year. In view of these indeterminates, a delivery period of 15 months for the new supplies (assuming that the necessary follow up will be carried out at different levels) and the utilisation of the available lead joint pipes in department to the possible extent to avoid the delay in the execution is considered reasonable.

C. NETWORK DIAGRAM

The administrative approval to the scheme was accorded on 8-9-1972, but the actual execution was delayed due to various reasons. The network diagram was prepared on 15 2 1974, i.e. about 75 weeks after the administrative approval. Thus, the time duration for the activities which are already over have been taken as per actuals while the activities which are yet to take place are projected taking into consideration the most probable period required. It will be seen that while working out the actual activities completed, the time durations for certain items are much higher than the normal which happened in this case due to the delays mentioned earlier. For examples, the activity (14) viz. working plans and estimates for raw and clear water pumping machinery should not have taken 73 weeks. A realistic figure would be around 20 to 30 weeks depending upon several factors like availability of staff etc. Therefore while drawing up the CPM chart at the beginning stages itself, it may be necessary to assume more rational figures of time duration based on the experience of the department and not providing for any delays. The charts can , however, be updated periodically. The following 11 major components of the scheme are further sub-divided into 102 activities to complete the project. The number of the activities on the network diagram is also shown in brackets for ready reference.

Major Components	Activity			Time Duration (weeks)
	Item		No.	
I. Head Works (including Raw Water Rising Main)	(i)	Working plans & estimates	(2)	43
	(ii)	Sanction	(3)	7
	(iii)	Draft tender papers	(4)	2
	(iv)	Receiving tenders	(5)	6
	(v)	Evaluate tenders &award of contract	(6)	10
				68
	(vi)	Execution work		
	(a)	Intake well connecting pipe, twin jack well	(10)	78
	(b)	Pump house	(73)	26
	(c)	Approach bridge with approach road and fencing to head works	(11)	78
	(d)	Part excavation of raw water rising main	(7)	20
	(e)	Part laying of raw water rising main	(9)	36

Major Components		Activity		Time Duration (weeks)
		Item	No.	
	(f)	Excavation of balance raw water rising main	(8)	20
	(g)	Laying balance length of rising main	(72)	36
	(h)	Fixing specials and valves constructing chambers on the rising main	(73)	12
II Pumping machinery for raw water and clear water group I	(i)	Working plans and estimates	(14)	73
	(ii)	Sanction	(15)	10
	(iii)	Draft tender papers	(16)	4
	(iv)	Receive tenders	(17)	8
	(v)	Evaluate tender & award of contract	(70)	12
	(vi)	Deliveries of pumping machinery,	(74)	48
		C.I pipes, specials, valves,	(88)	48
		gantry girders, etc	(87)	48
	(vii)	Erection works	(101)	12
			(101)	
			(83)	
	(viii)	Trial run	(102)	4
III Clear Water Pumping Machinery Group II	(i)	Working plans and estimates	(43)	73
	(ii)	Sanction	(44)	10
	(iii)	Draft tender papers	(45)	4

Major Components	Activity			Time Duration (weeks)
	Item		No.	
	(iv)	Receive tenders	(46)	6
	(v)	Evaluate tenders & award of contract	(64)	4
	(vi)	Delivery of pumping machinery, C.I pipes, specials, valves, gantry girders etc.	(69)	32
			(68)	32
	(vii)	Erection work	(100)	8
			(100)	8
	(viii)	Trial run	(102)	4
IV Treatment Works	(i)	Draft tender papers	(18)	62
	(ii)	Receive tenders	(19)	6
	(iii)	Evaluate tenders & award of contract	(76)	16
				84
	(iv)	Execution work		
	(a)	Supply of mechanical and electrical equipment for clariflocculator.	(79)	44
	(b)	Supply of mechanical and electrical equipment for filters.	(81)	60
	(c)	Erection of all equipment upto clariflocculator	(80)	8
	(d)	Erection of all equipment upto filters	(82)	14
	(e)	Civil works for clariflocculator	(77)	44

Major Components		Activity: Item		No.	Time Duration (weeks)		
		(f)	Civil works for filters	(78)	55		
		(g)	Trial run of filters	(102)	4		
V.	Clear Water Reservoir And Pump House	(i)	Working plans & estimates	(20)	78		
		(ii)	Approval	(21)	3		
		(iii)	Draft tender papers	(22)	3		
		(iv)	Receive tenders	(23)	6		
		(v)	Evaluate tenders and award of contract	(85)	8		
		(vi)	Execution	(86)	35		
				Group I At treatment works at point A		Group II At points B & C	
VI	R.C.C. Service Reservoirs	(i)	Working plans B&C	(24)	80	(36)	80
		(ii)	Approval	(25)	6	(37)	6
		(iii)	Draft tender papers	(26)	8	(38)	8
		(iv)	Receive tenders	(27)	6	(39)	6
		(v)	Evaluate tenders and award of contract	(89)	8	(70)	8
		(vi)	Execution	(90)	65	(98)	65
				(91)	65	(99)	65
VII	Clear Water Rising Mains and Gravity Mains	(a)	Rising Main to S.R. at treatment works				
		(b)	Rising Main to S.R. Point A				

Major Components			Activity Item	No.	Time Duration (weeks)
		(c)	Booster main to Point B		
		(d)	Booster main to Point C		
		(e)	Gravity main from S.R. at Point A to Point B.		
		(i)	Working plans and estimates	(31)	73
		(ii)	Approval	(32)	10
		(iii)	Draft tender papers	(33)	4
		(iv)	Receive tenders	(34)	6
		(v)	Evaluate tenders and of contract award	(92)	4
		(vi)	Execution	(93,94, 95,96,97)	24
VIII	Booster Pumping stations		At Point B and C		
		(i)	Working plans and estimates and approval	(40)	90
		(ii)	Draft tender papers	(41)	4
		(iii)	Receive tenders	(42)	4
		(iv)	Evaluate tenders and contract award	(65)	2
		(v)	Execution	(66,67)	24

Major Components		Activity Item		Activity No.	Time Duration (weeks)
IX	Nalla Diversion Nalla to be diverted to Downstream of Head Works to control pollution	(i)	Working Plans and estimates	(48)	97
		(ii)	Approval	(49)	4
		(iii)	Draft tender papers	(50)	2
		(iv)	Receive tenders	(51)	4
		(v)	Evaluate tenders and contract award	(52)	2
		(vi)	Execution	(53)	20
X.	Staff Quarters		At Head Works, Treatment Works, etc.		
		(i)	Working plans and estimates	(54)	93
		(ii)	Approval	(55)	6
		(iii)	Draft tender papers	(56)	4
		(iv)	Receive tenders	(57)	6
		(v)	Evaluate tenders	(58)	8
		(vi)	Execution	(59)	52
XI	Miscellaneous Works		(a) i) Land acquisition for head works and booster pumping stations including barbed wire fencing.	(60)	140
			ii) Providing internal roads, external W.S and drainage arrangements to quarters, etc.	(61)	32
		(iii)	Providing electrical installation to staff quarters, pump house, boosting stations and at headworks &	(62)	32

Major Components	Activity			Time Duration (weeks)
		Item	No.	
		treatment works premises.		
	(b)	Telephone connections	(63)	155
	(c)	C.I pipes, valves, specials		
		(New order to be placed)		
		(i) Indent	(12)	77
		(ii) Supply order by S.E	(13)	4
		(iii) Delivery	(71)	65
	(d)	C.I pipes against old order placed by S.E on 31-5-1973		
		(i) Supply order	(29)	37
		(ii) Delivery	(30)	65
	(e)	Final transfer of Govt. land for treatment works, clear water reservoir and pump house and S.R at Treatment works.	(75)	85
	(f)	Obtaining permission of B & C Deptt. for crossing NH for clear water rising main to S.R at point A.	(28)	130
	(g)	Obtaining permission of Railways for crossing railway track for	(35)	130

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Major Components	Activity			Time Duration (weeks)
		Item	No.	
		clear water gravity main from S.R to Lane at Point A.		
	(h)	Erecting transmission lines & transformers at headworks, treatment works and booster pumping station.	(47)	130

From the Network Diagram, it may also be seen that the Prime Critical Path is through the Headworks and covering the activities 1-2-3-4-5-6-10-73-83-101-102 as shown and the time of completion is 188 weeks. Since 4 weeks of testing for all pumping plant and machinery and 12 weeks for erection of raw water pump set are included in this 188 week periods the time duration for the different major components could be summarized as:

I	Headworks including Raw Water Rising Main	172 Weeks
II	Raw Water and Clear Water Pumping Machinery	167 weeks
III	Treatment works	
(a)	Clariflocculators	136 weeks
(b)	Filters	158 weeks
IV	Clear Water Sump and Pump House	128 weeks
V.	R.C.C. Services Reservoirs	173 Weeks
V	Clear Water Rising and Gravity Mains	128 weeks
VII	Booster Pump Stations	124 weeks
VIII	Nallah Diversion	129 weeks
IX.	Staff Quarters	169 weeks
X.	Miscellaneous Works	

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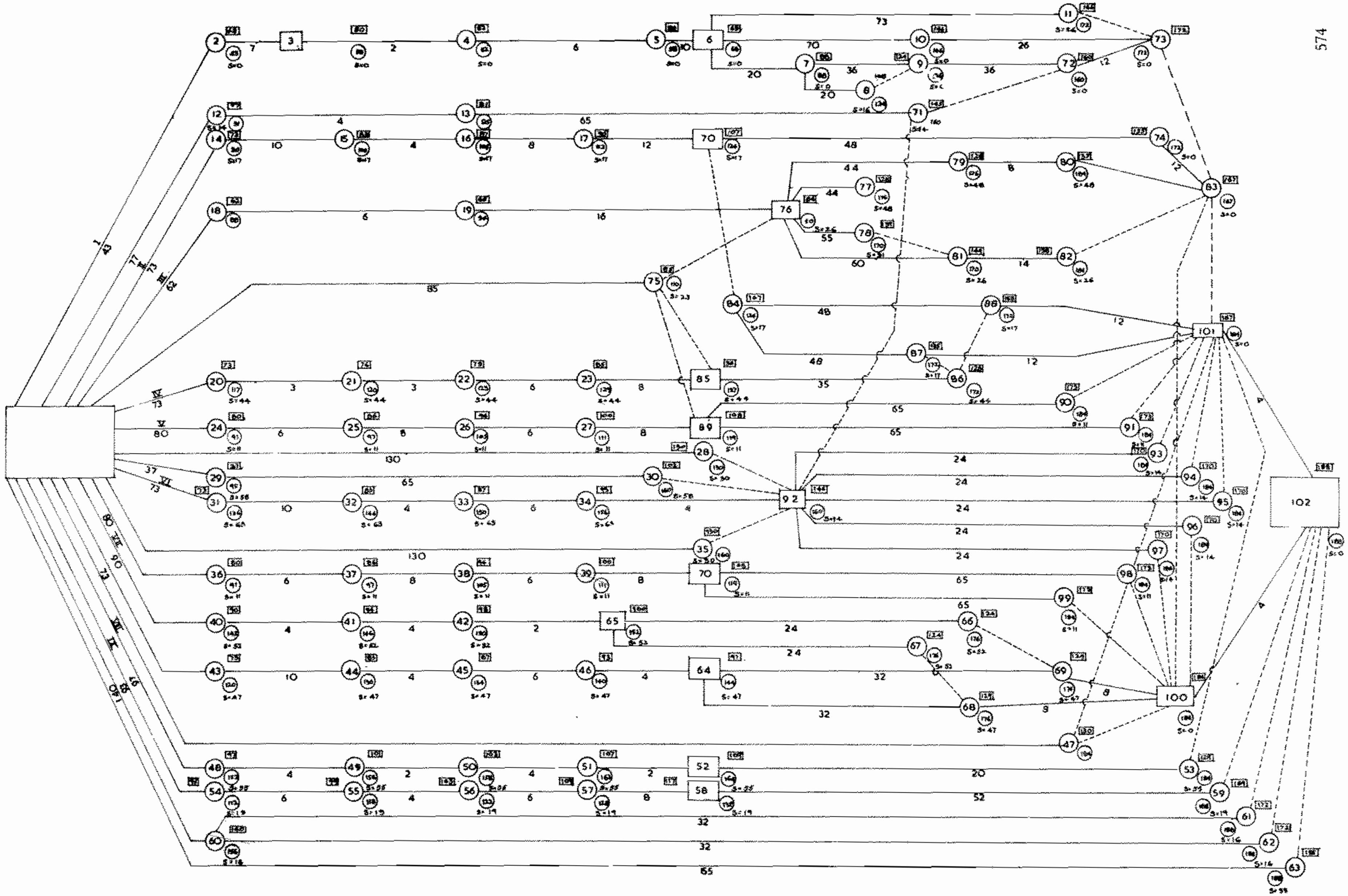
(a)	Land acquistion for head works and provisions of barbed wire fencing, internal roads, etc.	172 weeks
(b)	Transfer of Government land for treatment works, etc.	85 weeks
(c)	Telephone connections	155 weeks
(d)	Supply of C.I. pipes and specials	
(i)	New order to be Placed	146 weeks
(ii)	Orders already placed	102 weeks
(e)	Obtaining permission of PWD for National Highway crossing of the rising main	130 weeks
(f)	Obtaining permission of Railways for crossing railway tracks	130 weeks

By proper advance planning and continuous persuasive efforts, it should be possible that the salient works are completed in 188 weeks which is the critical period for the major items of headworks, raw water mains and treatment works so that water could be made available to the consumers even if the scheme is not complete in all respects.



C.P.M. NET-WORK DIAGRAM FOR A TYPICAL WATER SUPPLY AUGMENTATION SCHEME

APPENDIX 5.1

MASS DIAGRAM FOR IMPOUNDING STORAGE

PROBLEM

Draw the mass diagram and compute the storage needed for an impounding reservoir for a constant draft of 23 ml/sqkm/month of 30.4 days with the following recorded mean monthly run of values.

Order of month	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Observed monthly mean run off, million litres per square Kilometers	94	122	45	5	5	2	0	2	16	7	72	92	21	55	33

SOLUTION

Methodology

The mass diagram is obtained by plotting the time interval (order of the month) as abscissa and the cumulative run off and cumulative draft up to the corresponding time interval as calculated in table below as ordinates.

TABLE SHOWING CALCULATION OF REQUIRED STORAGE

(Volume of water in million liters per square kilometre)

Order of month	Recorded run-off Q	Estimated draft D	Comul-ative run-off ΣQ	Deficiency D-Q	Cumula-tive deficiency $\Sigma(D-Q)$	Reservoir state
(1)	(2)	(3)	(4)=Σ(2)	(5)=(3)-(2)	(6)=Σ(5)	(7)
1	94	23	94	-71	0(192)	
2	122	23	216	-99	0(121)	
3	45	23	261	-22	0(22)	Reservoir full at the beginning dry period
4	5	23	266	18	18*	
5	5	23	271	18	36	
6	2	23	273	21	57	*Reservoir empties
7	0	23	273	23	80	

Order of month	Recorded run-off Q	Estimated draft D	Comul-ative run-off ΣQ	Deficiency D-Q	Cumula-tive deficiency Σ(D-Q)	Reservoir state
8	2	23	275	21	101	
9	16	23	291	7	108	
10	7	23	298	16	124	Maximum deficiency at end of dry period
11	72	23	370	-49	75	
12	92	23	462	-69	6	
13	21	23	483	2	8	
14	55	23	538	-32	0(24)	Reservoir refilled
15	33	23	571	-10	0(34)	

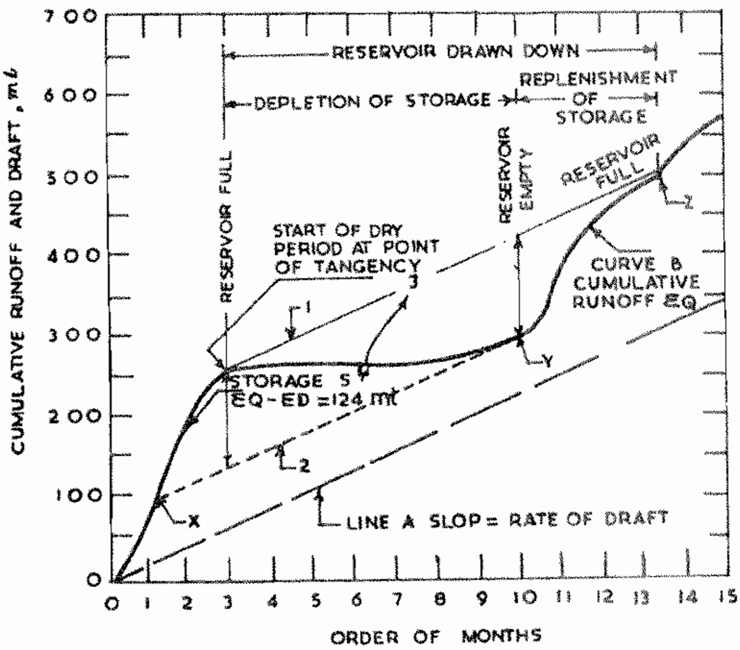
Col 3 Constant rate of draft = 23 mL/ sq km for an average month of 30.4 days.

Col. 5. Negative value indicates surplus.

Col. 6. Negative values are not included in Σ (D-Q) until the beginning of dry period i.e. until water is lost from storage and there is room to store incoming flows. The surplus preceding the dry period, however, must equal or exceed the preceding maximum deficiency; otherwise the reservoir will not be full at the beginning of dry period. The cumulative surplus, calculated backwards from the beginning of dry period, is shown in brackets in column 6 and is seen to exceed 124 mL/sq km of catchment area. The cumulative run-off curve 'B' has been drawn as shown in the figure.

The cumulative draft line for the area under consideration is also plotted in the same scale (line A) assuming constant draft of 23 mL/sq km of catchment area for a month of 30.4 days. The slope of line 'A' indicates the rate of draft.

The maximum deficit of run off from the draft is obtained by drawing a straight line parallel to the cumulative draft curve at the crest and through the cumulative run-off curve tangentially. The vertical ordinate length intercepted between two such parallel lines tangential to the crest and trough gives the maximum deficit for the period between the points of intersection of the parallel line with the mass curve. The maximum cumulative deficiency as observed from the mass curve (which could also be determined analytically as shown in the table) is 124 mL/sq km of catchment area. For the constant rate of draft of 23 mL/sq km of catchment area for a month of 30.4 days and for this cycle of runoff values , the impounded storage needed is for (124/23)x 30.4 i.e 165 days (almost half a year).



1. DRAW CUMULATIVE DRAFT ΣD PARALLEL TO RATE OF DRAFT A AND TANGEN TO CURVE B
2. DRAW PARALLEL TO LINE A AND TANGENT TO CURVE B
3. MEASURE MAXIMUM DEFICIENCY CUMULATIVE ΣD − ΣQ = 124 ml

X. MUST INTERSECT RUNOFF CURVE, IF RESERVOIR IS TO BE FULL AT START OF DRY PERIOD.

Y. END OF DRY PERIOD AT POINT OF TANGENCY.

Z. MUST INTERSECT RUNOFF IF RESERVOIR IS TO REFILL.

MASS—DIAGRAM FOR IMPOUNDING STORAGE

APPENDIX 5.2

GROUND WATER RESOURCES AND IRRIGATION POTENTIAL

(Provisional)

States/ Uts	Total Replenshi-ble Ground Water Resource	Provision For Drinking Industrial & Other Uses	Utilisable Ground Water Resources For Irrigation	Net Draft	Balance Available For Irrigation	Net Irrigation Require-ments	Ultimate Irrigation Potential	Potential Utilised	Balance Irr. Pot. To Be Developed
	(m ha m/Yr)	(m ha m/Yr)	(m ha m/Yr)	(m ha m/Yr)	(m ha m/Yr)	(Range) (m)	(m ha)	(m ha)	(m ha)
1	2	3	4	5	6	7	8	9	10
STATES									
Andhra Pradesh	4.34	0.65	3.69	0.74	2.95	(0.558-.0909)	5.19	1.04	4.15
Arunachal Pradesh	0.14	0.02	0.12	0.00	0.12		0.002		0.02
Assam	2.35	0.35	2.00	0.05	1.95	1.280	1.56	0.04	1.52
Bihar	3.38	0.51	2.87	0.68	2.19	0.400	7.18	1.70	5.48
Goa	0.061	0.015	0.046	0.0040	0.0420	0.600	0.076	0.006	0.070

States/ Uts	Total Replenshi-ble Ground Water Resource (m ha m/Yr)	Provision For Drinking Industrial & Other Uses (m ha m/Yr)	Utilisable Ground Water Resources For Irrigation (m ha m/Yr)	Net Draft (m ha m/Yr)	Balance Available For Irrigation (m ha m/Yr)	Net Irrigation Require-ments (Range) (m)	Ultimate Irrigation Potential (m ha)	Potential Utilised (m ha)	Balance Irr. Pot. To Be Developed (m ha)
1	2	3	4	5	6	7	8	9	10
Gujrat									
Confined	0.22	0.03	0.19	0.11	0.08	(0.0364 - 0.500)	0.44	0.25	0.19
Un-confined	2.04	0.31	1.73	0.53	1.20	(.0315 -0.500)	4.37	1.37	3.00
Haryana	0.85	0.13	0.72	0.51	0.21	0.385	1.88	1.32	0.56
Himachal Pradesh	0.036	0.007	0.029	0.006	0.023	0.385	0.074	0.016	0.088
Jammu & Kashmir	0.44	0.07	0.37	0.005	0.365	(0.385 -0.600)	0.783	0.012	0.771
Karnataka	1.62	0.24	1.38	0.50	0.88	(0.350 - 0.360)	3.12	0.70	2.420
Kerala	0.81	0.12	0.69	0.07	0.62	0.690	0.99	0.09	0.90
Madhya Pradesh	5.97	0.89	5.08	0.60	4.48	0.400	12.70	1.50	11.20
Mahara-shtra	3.38	0.67	3.20	0.70	2.50	(0.400 -0.750)	5.84	1.32	4.52
Manipur	0.012	0.002	0.010	0.00	0.01	0.650	0.016		0.016

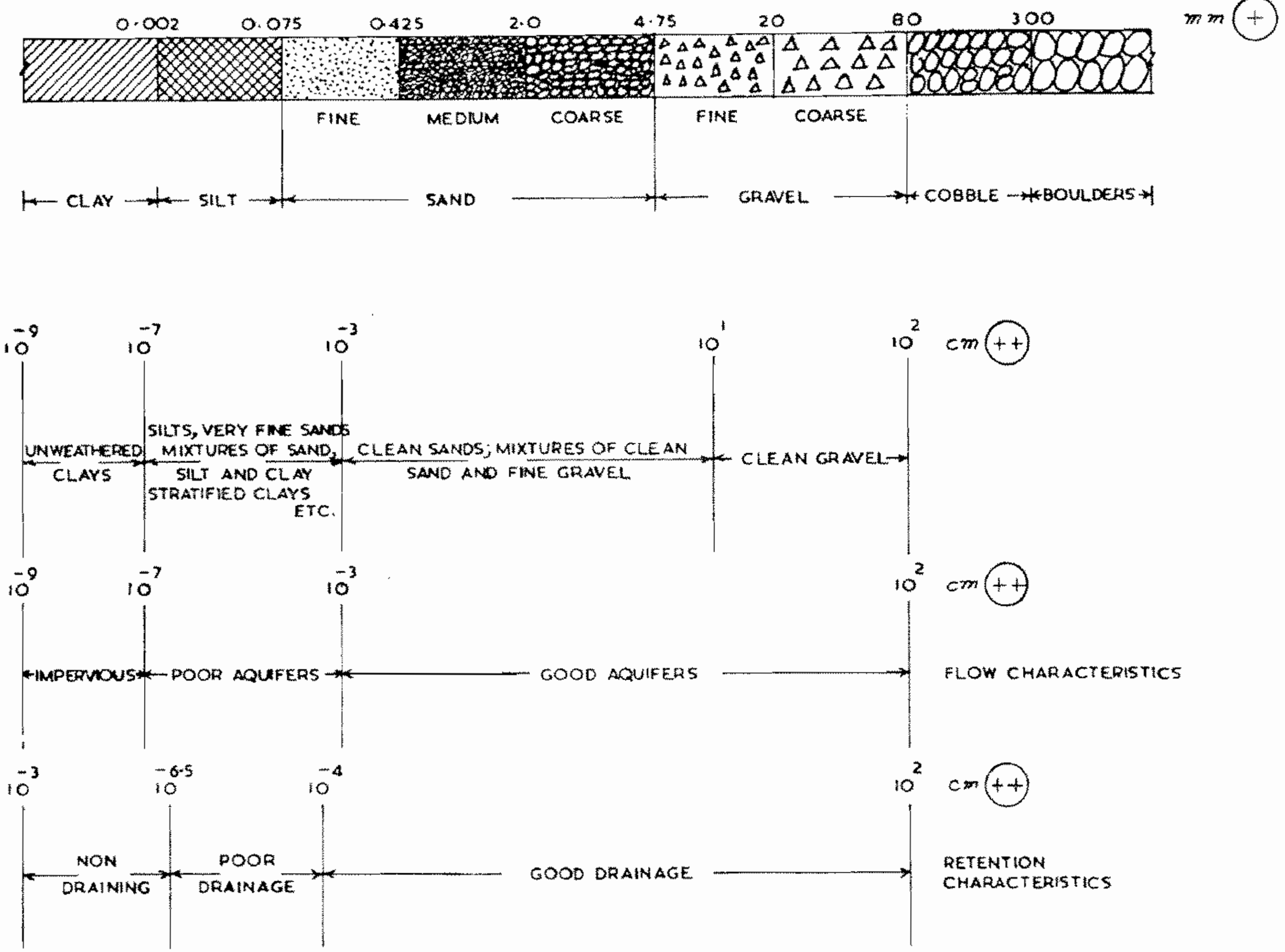
States/ Uts	Total Replenshi-ble Ground Water Resource (m ha m/Yr)	Provision For Drinking Industrial & Other Uses (m ha m/Yr)	Utilisable Ground Water Resources For Irrigation (m ha m/Yr)	Net Draft (m ha m/Yr)	Balance Available For Irrigation (m ha m/Yr)	Net Irrigation Require-ments (Range) (m)	Ultimate Irrigation Potential (m ha)	Potential Utilised (m ha)	Balance Irr. Pot. To Be Developed (m ha)
1	2	3	4	5	6	7	8	9	10
Meghal-aya	0.043	0.007	0.036	0.000024	0.035976	0.650	0.056	0.00004	0.05596
Mizoram					Not Assessed				
Nagaland	0.006	0.001	0.004	0.00	0.004				
Orissa	2.33	0.35	1.98	0.10	1.88	(0.34 - 0.44)	5.40	0.25	5.15
Punjab	1.80	0.27	1.53	1.52	0.01	0.400	3.82	3.80	0.02
Rajasthan	1.62	0.29	1.33	0.50	0.83	(0.39- 0.42)	3.44	1.26	2.18
Sikkim					Not Assessed				
Tamilnadu	3.02	0.46	2.56	1.20	1.36	(0.360 -0.937)	3.35	1.45	1.90
Tripura	0.06	0.01	0.05	0.005	0.045	0.630	0.06	0.008	0.072
Uttar pradesh	8.05	1.21	6.84	2.50	4.34	.360	18.00	11.50	6.50
West Bengal	2.07	0.31	1.76	0.29	1.47	(0.6-1.67)	1.88	0.23	1.65
Total states	45.147	6.922	38.225	10.620	27.605		80.265	27.862	52.40296

States/ Uts	Total Replenshi-ble Ground Water Resource	Provision For Drinking Industrial & Other Uses	Utilisable Ground Water Resources For Irrigation	Net Draft	Balance Available For Irrigation	Net Irrigation Require-ments	Ultimate Irrigation Potential	Potential Utilised	Balance Irr. Pot. To Be Developed
	(m ha m/Yr)	(m ha m/Yr)	(m ha m/Yr)	(m ha m/Yr)	(m ha m/Yr)	(Range) (m)	(m ha)	(m ha)	(m ha)
1	2	3	4	5	6	7	8	9	10
Say	45.15	6.92	38.23	10.62	27.61		80.27	27.86	52.41
UNION TERRI-TORIES									
Andam-an & Nicobar									
Chandi-garh	0.0035		0.0035	0.0059	0.0024				
Dadra & Nagar Haveli	0.0075	0.0023	0.0062	0.005	0.0047	0.6500	0.0080	0.0007	0.0073
Delhi	0.0604	0.0076	0.0428	0.0287	0.0141	0.3850	0.1112	0.0745	0.0367
Daman & Diu					Not Assessed				
Laksha-dweep					Not Assessed				
Pondic-herry	0.0175	0.0026	0.0149	0.0204	-0.0055	0.0000	0.0000	0.0000	0.0000

States/ Uts	Total Replenshi-ble Ground Water Resource	Provision For Drinking Industrial & Other Uses	Utilisable Ground Water Resources For Irrigation	Net Draft	Balance Available For Irrigation	Net Irrigation Require-ments	Ultimate Irrigation Potential	Potential Utilised	Balance Irr. Pot. To Be Developed
	(m ha m/Yr)	(m ha m/Yr)	(m ha m/Yr)	(m ha m/Yr)	(m ha m/Yr)	(Range) (m)	(m ha)	(m ha)	(m ha)
1	2	3	4	5	6	7	8	9	10
Total UTs	0.0789	0.0125	0.0664	0.0555	0.0109	1.0350	0.1192	0.0752	0.0440
Say	0.08	0.01	0.07	0.06	0.01		0.12	0.08	0.04
Total All India	45.2259	6.9345	38.2914	10.6755	27.6159		80.3842	27.9372	52.4469
Say	45.23	6.93	38.30	10.68	27.62		80.38	27.94	52.44

*The estimates of ground Water Resources and as per the norms and guidelines laid down by the Ground Water Estimation Committee (1984) assessed by the working group based by the state Irrigation Secretary incharge of Ground Water Department and comprised of the head of the Ground Water Organisation in the State Director of State Agricultural Department and representative from Agriculture Universities and the regional Director of Control Ground Water Board as the convenor.

APPENDIX 5.3



(+) EFFECTIVE SIZE IN *MM*

(++) CO-EFFICIENT OF PERMEABILITY *CM*/SEC. AT UNIT HYDRAULIC GRADIENT CLASSIFICATION OF SOIL

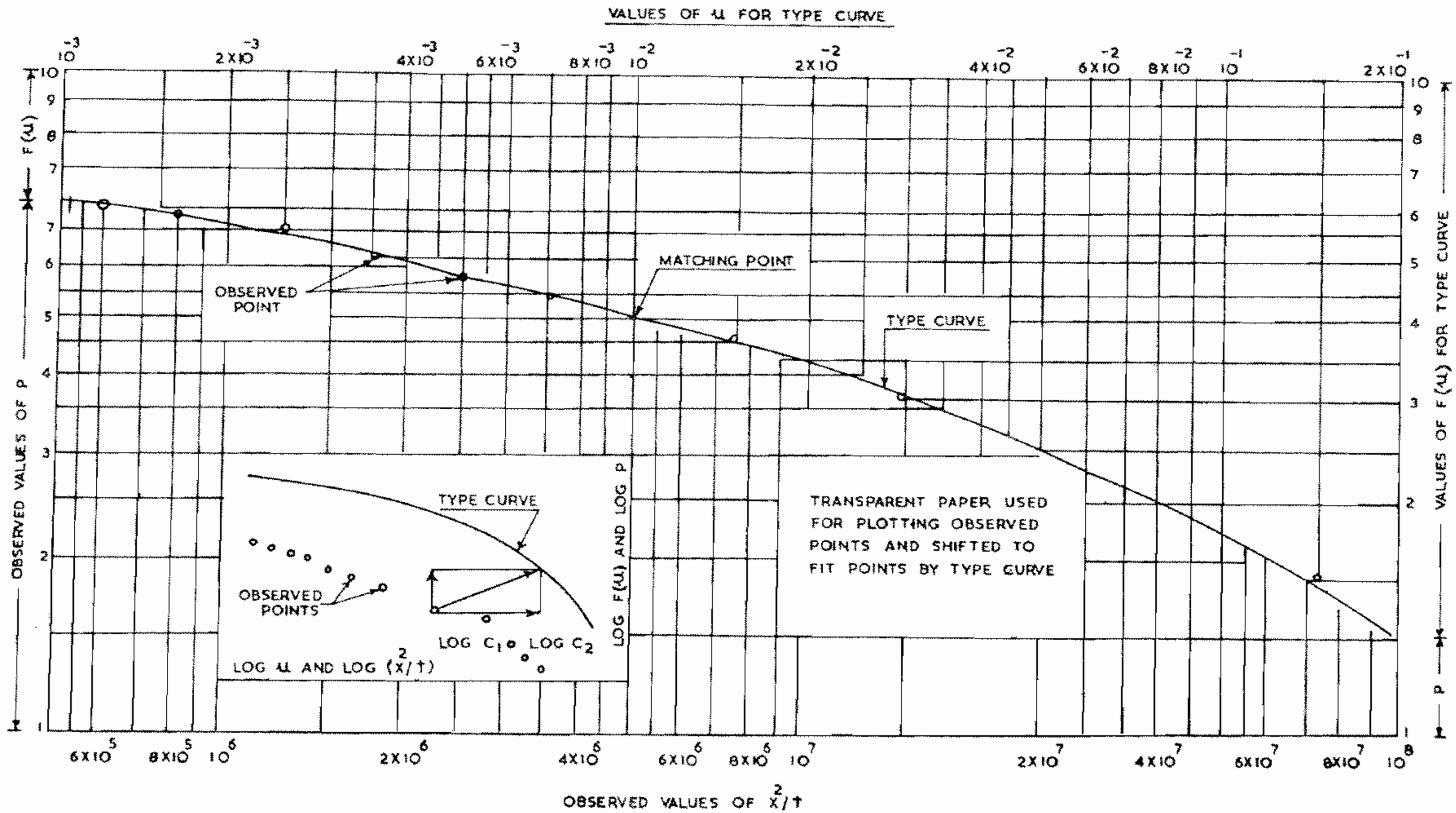
APPENDIX 5.4

VALUES OF THE WELL FUNCTION F(U) FOR VARIOUS VALUES OF U

N	N x 10^{-15}	Nx10^{-14}	Nx10^{-13}	Nx10^{-12}	Nx10^{-11}	Nx10^{-10}	Nx10^{-9}	Nx10^{-8}
1	2	3	4	5	6	7	8	9
1.0	33.96	31.66	29.36	27.05	24.75	22.45	20.15	17.84
1.5	33.56	31.25	28.95	26.65	24.35	22.04	19.74	17.44
2.0	33.27	30.97	28.66	26.36	24.06	21.76	19.45	17.15
2.5	33.05	30.74	28.44	26.14	23.83	21.53	19.23	16.93
3.0	32.86	30.56	28.26	25.96	23.65	21.53	19.05	16.75
3.5	32.71	30.41	28.10	25.80	23.50	21.20	18.69	16.59
4.0	32.57	30.27	27.97	25.67	23.36	21.06	18.76	16.46
4.5	32.46	30.15	27.85	25.55	23.25	20.94	18.64	16.34
5.0	32.35	30.05	27.75	25.44	23.14	20.84	18.54	16.23
5.5	32.26	29.95	27.65	25.35	23.05	20.74	18.44	16.14
6.0	32.17	29.87	27.56	25.26	22.96	20.66	18.35	16.05
6.5	32.09	29.79	27.48	25.18	22.88	20.58	18.27	15.97
7.0	32.02	29.71	27.41	25.11	22.81	20.50	18.20	15.90
7.5	31.95	29.64	27.34	25.04	22.74	20.43	18.13	15.83
8.0	31.88	29.58	27.28	24.97	22.67	20.37	18.07	15.76
8.5	31.82	29.52	27.22	24.91	22.61	20.31	18.01	15.70
9.0	31.76	29.46	27.16	24.86	22.55	20.25	17.95	15.65
9.5	31.71	29.41	27.11	24.86	22.50	20.20	17.89	15.59

N	$N \times 10^{-7}$	$N\times10^{-6}$	$N\times10^{-5}$	$N\times10^{-4}$	$N\times10^{-3}$	$N\times10^{-2}$	$N\times10^{-1}$	N
10	11	12	13	14	15	16	17	18
1.0	15.54	13.24	10.94	8.6333	6.332	4.038	1.823	2.194×10^{-1}
1.5	15.14	12.83	10.53	8.228	5.927	3.637	1.465	1.000×10^{-1}
2.0	14.85	12.55	10.24	7.094	5.639	3.355	1.223	4.890×10^{-2}
2.5	14.62	12.32	10.02	7.717	5.417	3.137	1.044	2.491×10^{-2}
3.0	14.44	12.14	9.837	7.585	5.235	2.959	0.9057	1.305×10^{-2}
3.5	14.29	11.99	9.683	7.381	5.081	2.810	0.7942	6.970×10^{-3}
4.0	14.15	11.85	9.550	7.247	4.948	2.681	0.7024	3.779×10^{-3}
4.5	14.04	11.73	9.432	7.130	4.831	2.568	0.6253	2.073×10^{-3}
5.0	13.93	11.63	9.326	7.024	4.726	2.468	0.5598	1.148×10^{-3}
5.5	13.84	11.53	9.231	6.929	4.631	2.378	0.5034	6.409×10^{-4}
6.0	13.75	11.45	9.144	6.842	4.545	2.295	0.4544	3.601×10^{-4}
6.5	13.67	11.37	9.064	6.762	4.465	2.220	0.4115	2.034×10^{-4}
7.0	13.60	11.29	8.990	6.688	4.392	2.151	0.3738	1.155×10^{-4}
7.5	13.53	11.22	8.921	6.619	4.323	2.087	0.3403	6.583×10^{-5}
8.0	13.46	11.16	8.856	6.555	4.259	2.027	0.3106	3.767×10^{-5}
8.5	13.40	11.10	8.796	6.494	4.199	1.971	0.2840	2.162×10^{-5}
9.0	13.34	11.04	8.739	6.437	4.142	1.919	0.2602	1.245×10^{-5}
9.5	13.29	10.99	8.685	6.383	4.089	1.870	0.2387	7.185×10^{-5}

APPENDIX 5.5



Type Curve

APPENDIX 5.6
YIELD TESTS FOR WELLS

GENERAL

Pumping tests are made on wells to determine their capacity and other hydraulic characteristics and to obtain information so that permanent pumping equipment can be intelligently selected. Preliminary tests of well drilled as test holes are sometimes made to compare the yielding ability of different water bearing formation or different locations in same formation. This information is then used as a basis for selecting the best site for a supply well and the aquifier in which it should be completed.

MEASUREMENTS

The measurement that should be made in testing wells include the volume of water pumped per minute or per hour, the depth to the static water level before pumping is started, the depth to the pumping level at one or more constant rates of pumpage, the recovery of water level after pumping is stopped and the length of time the well is pumped at each rate during test procedure.

PUMPING PROCEDURE

The pump and power unit used for testing a well should be capable of continuos operation at a constant rate of pumpage for several hours. It is important that the equipment be in good condition for an accurate test, since it is not desirable to have a shut down during the test. If possible, the test pump should be large enough to test the well beyond the capacity at which it will eventually be pumped, but this may not be always practicable under field operations.

In the pumping test, the pump is fixed close to the well and water is pumped out. The quantity of water pumped is measured using a circular orifice meter or a V notch. The water discharging from the V notch chamber should be let away in a channel, so that water pumped out does not find its way back into the well through the soil. As water from the well is pumped out, there will be stage when water level remains fairly constant, without any further increase in drawdown. The pumping rate in this position is the yield from the well for that head of depression or drawdown.

Aquifer performance test results for a typical case are given below:

AQUIFER PERFORMANCE TEST RESULTS

Site I	189.6 - 204.2 m
Site II	213.6 - 246.2 m

Test carried out on 11.2.75 using an air compressor of 7.1 kg/cm^2 capacity; static water level 4.41 m below measuring point which is 0.82 meter above ground level.

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Time since pump started (t) min	Time since pump stopped (t') min	t/t'	Drawdown S(m)	Residual drawdown Rd(m)	Yield (m^3/min)
1	2	3	4	5	6
1			6.55		
9			12.34		
11			11.81		
12			10.86		
14			12.62		
16			10.54		
18			10 63		
20			10.68		
22			10.78		
24			10.84		
26			10.87		
28			10.77		
30			10.65		
32			10.56		
34			10.45		
36			10.43		
38			10.60		
40			10.66		
42			10.43	240 litres/minute	
44			10.27		
46			10.36		
48			10.61		
50			10.56		

2962 UA&E/97—40B

Time since pump started (t) min	Time since pump stopped (t') min	t/t'	Drawdown S(m)	Residual drawdown Rd(m)	Yield (m^3/min)
1	2	3	4	5	6
52			10.24		
54			10.22		
56			10.37		
58			10.49		
60			10.26		
62			10.04		
67			10.18		
72			10.49		
77			10.47		
82			10.35		
87			10.30		
92			10.21		
97			10.12		
102			10.83		
117			10.40		
132			10.22		
147			10.04		
162			10.38		
192			10.36		
200			9.99		
202	2	101		5.44	
203	3	67.6		4.41	
204	4	51.0		4.26	
205	5	41.0		4.16	

Time since pump started (t) min	Time since pump stopped (t') min	t/t'	Drawdown S(m)	Residual drawdown Rd(m)	Yield (m^3/min)
1	2	3	4	5	6
206	6	34.3		3.67	
207	7	29.57		3.75	
208	8	26.0		3.65	
209	9	23.2		3.57	
210	10	21.0		3.50	
211	11	19 19		3.43	
212	12	17.66		3.36	
213	13	16.33		3.38	
214	14	15.28		3.30	
215	15	14.33		3.27	
216	16	13.50		3 24	
217	17	12.76		3.23	
218	18	12.11		3.22	
219	19	11.52		---	
220	20	11.0		3.215	
221	21	10.52		3.21	
223	23	9.69		3.20	
224	24	9 33		3.19	
225	25	9.0		3.18	
226	26	8.69		3.17	
227	27	8.47		3.16	
228	28	8.14		3 15	
229	29	7.89		3 14	
230	30	7.66		3.13	

Time since pump started (t) min	Time since pump stopped (t') min	t/t'	Drawdown S(m)	Residual drawdown Rd(m)	Yield (m^3/min)
1	2	3	4	5	6
231	31	7.45		3.13	
232	32	7.25		3.11	
234	34	6 88		3.10	
236	36	6.55		3 09	
238	38	6.26		3 07	
240	40	6.00		3.03	
242	42	5.76		3.01	
244	44	5.54		2.97	
246	46	5 34		2.94	
248	48	5.16		2.90	
250	50	5.00		2.87	
252	52	4.84		2 83	
254	54	4.70		2.80	
256	56	4.57		2 77	
258	58	4.44		2.74	
260	60	4.33		2.70	
262	62	4.22		2.62	
267	67	3.98		2.61	
272	72	3.77		2 54	
277	77	3.59		2.47	
282	82	3.43		2.41	
287	87	3.29		2.36	
292	92	3 17		2.26	
297	97	3.06		2.19	

Time since pump started (t) min	Time since pump stopped (t') min	t/t'	Drawdown S(m)	Residual drawdown Rd(m)	Yield (m^3/min)
1	2	3	4	5	6
302	102	2.96		2.17	
307	107	2.86		2.10	
312	112	2.78		2.01	
322	122	2.63		1.88	
332	132	2.51		1.76	
342	142	2.40		1.70	
352	152	2.31		1.48	
362	162	2.24		1.35	
372	172	2.16		1.17	
382	182	2.09		1.05	
392	192	2.04		0.90	
402	202	1.99		0.78	
422	222	1.90		0.58	
442	242	1.82		0.45	
462	262	1.76		0.36	
482	282	1.70		0.29	
492	292	1.68		0.25	
512	312	1.64		0.21	
542	342	1.58		0.16	
572	372	1.53		0.12	
602	402	1.49		0.10	
632	432	1.46		0.08	
662	462	1.43		0.07	
692	492	1.40		0.06	

Time since pump started (t) min	Time since pump stopped (t') min	t/t'	Drawdown S(m)	Residual drawdown Rd(m)	Yield (m^3/min)
1	2	3	4	5	6
722	522	1.38		0.05	
752	552	1.36		0.05	
782	582	1.34		0.04	
812	612	1.32		0.04	
1402	1202	1.16		0.004	
1462	1262	1.15		---	

Using the recovery formula;

$T = 264Q/\Delta s$

Where,

Q is the discharge from the aquifer m^3 per minute under given condition of test and Δs is the residual drawdown between two values of t/t' which are one log cycle apart.

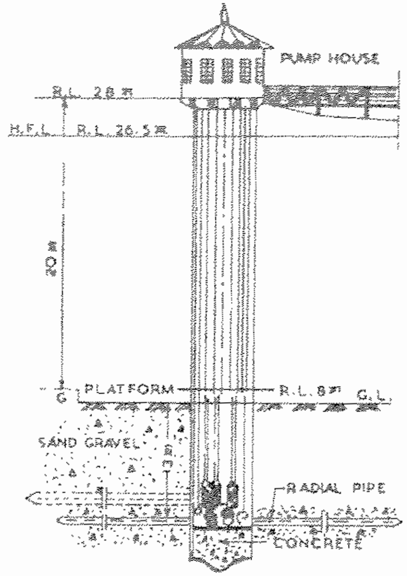
By plotting t/t' along the logarithmic scale and residual drawdown Rd along the arithmetic scale in a semi-log co-ordinate paper, the value of Δs is obtained from the graph which is 0.98.

$T = 264 \times 0.240/0.98$ cubic meters/meter/day

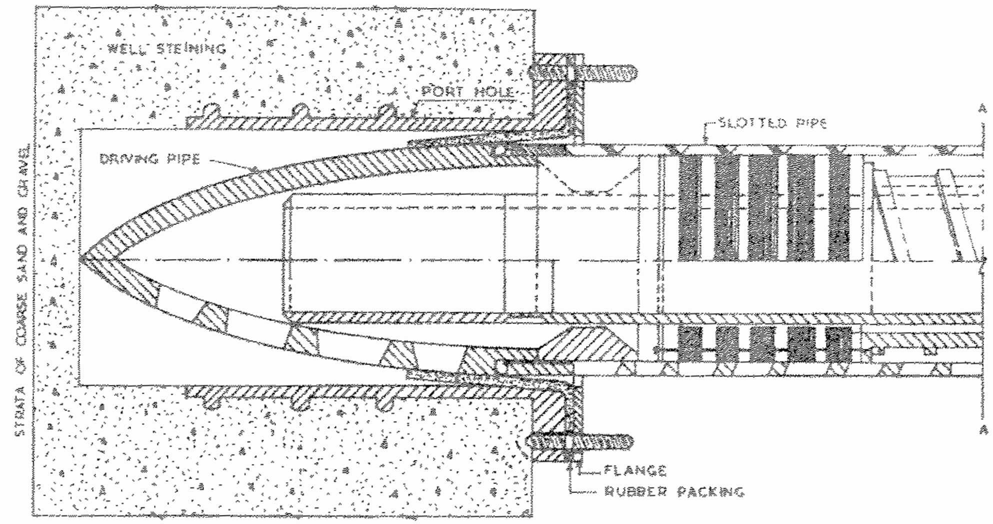
or $T = 0.0449$ cubic meters/meter/minute.

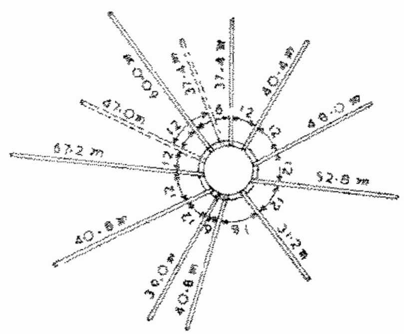
It may be noted that the t/t' plotted against residual drawdown Rd in a semi-log co-ordinate paper, in this particular case, give a very steep straight line plot so that the value of Rd works out 0.98. The high value of residual drawdown is indicative of poor recharge into the well. This was obviously due to the fact that sand nearly 4 % by volume was pumped out as the air compressor continued to lift out water. It is apparent that the strainers were filled with sand and so was the well assembly. Therefore, the test results have to be viewed with a great degree of reservation as far as the true character of the aquifer is concerned.

APPENDIX 5.7

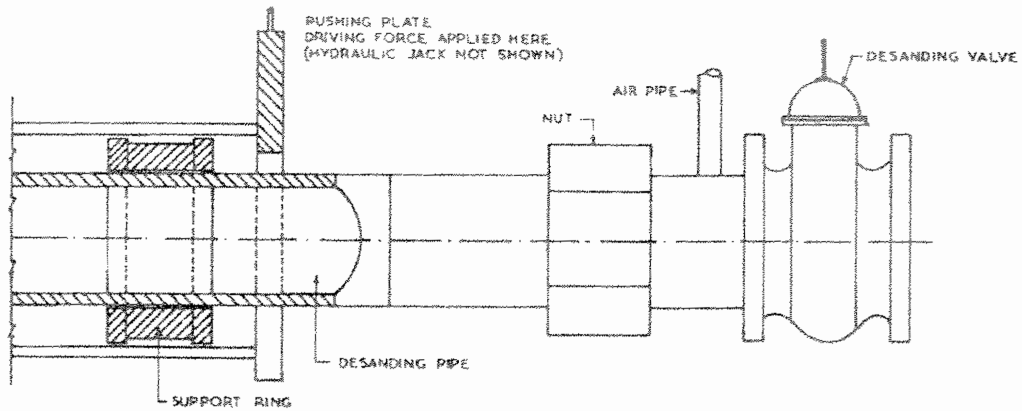


SECTIONAL ELEVATION





PLAN OF RADIALS



ARRANGEMENT FOR DRIVING HORIZONTAL SLOTTED PIPE

RADIAL COLLECTOR WELL

APPENDIX 5.8

DISINFECTION OF NEW OR RENOVATED WELLS,TUBEWELLS AND PIPELINES

DISINFECTION OF WELLS

New wells as well as those after repairs have to be disinfected by heavy doses of chlorine. The doses applied are generally of the order of 40 to 50 mg/l of available chlorine and bleaching powder is usually employed.

DUG WELLS

1. After the casing or lining is completed, the procedure outlined below may be carried out before the cover platform is placed over the well:
   - (i) Remove all equipment and materials including tools, platforms etc. , which do not form a permanent part of the completed structure
   - (ii) Wash the interior walls of the casing or lining with a strong solution of the bleaching powder (50 mg/l chlorine) using a steel broom or brush to ensure thorough cleaning
   - (iii) Pump the water from the well until it is perfectly clear and remove the pumping equipment that was temporarily set up for this purpose .

2. Place the cover over the well and pour the required amount of bleaching powder solution in to the well through the manhole or pipe opening just prior to inserting the pump cylinder and drop-pipe assembly. The bleaching powder added should give a dose of 50 mg/l. of chlorine in the volume of water in the well. Care should be taken to distribute the chlorine solution over as much of the surface of the water as possible to obtain proper mixing of the chemical with well water, which may be facilitated by running the solution into the well through a hose or pipeline as the line is being alternatively lowered and raised.
3. Wash the exterior surface of the pump cylinder and drop pipe with bleaching powder solution giving 50 mg/l of chlorine when the assembly is being lowered into the well.
4. Allow the chlorine solution to remain in the well for not less than 24 hours.
5. After 24 hours or more have elapsed , the well should be flushed by pumping the water to waste, till the residual chlorine is brought to 1mg/l.

TUBEWELLS

1. When the well is tested for yield ,the test pump should be operated until the well water is as clean and free from turbidity as possible.
2. After the testing equipment has been removed, pour the required amount of bleaching solution into the well slowly just prior to installing the permanent pumping equipment. The

dose of chlorine should be maintained at 50mg/l. Mixing of the chemical with well water may be facilitated by running the solution into the well through a hose or pipeline as the line is being alternatively raised and lowered.

3. Wash the exterior surface of the pump cylinder and drop pipe with bleaching powder solution before positioning
4. Allow the chlorine solution to remain in the well for not less than 24 hours
5. After 24 hours or more have elapsed, the well should be flushed by pumping the water to waste till a residual of 1 mg/l of chlorine is obtained. In the case of deep wells having a high water level, it may be necessary to resort to special methods of introducing the disinfecting agent in to the well so as to ensure proper mixing of chlorine throughout the well .

Similar procedure is adopted when troubles due to iron bacteria are noticed in the tube wells particularly when they come out as stringy masses along with the water.

DISINFECTION OF PIPELINES

When a section of water main is laid or repaired it is impossible to avoid contaminating the inner surface with dirt, mud or water in the trench while the pipes are being fixed into place. Contamination may also occur by accident, negligence or malice; adequate surveillance during working hours and the plugging of open ends after the day's work will reduce these risks. It should be assumed however that the pipe is contaminated despite all the precautions taken to prevent the entry of foreign matter. Secondly the main must be disinfected before it is put into service.

To obtain good results from disinfection and to avoid the hazards of subsequent obstructions and damage to valves, all foreign objects and material should be removed before hand by swabbing and flushing to clean the pipeline. Packing and jointing material should be cleaned and disinfected immediately before use by immersion in a 50 mg/l of chlorine solution for at least 30 minutes.

The presence of hydrants, air valves, gate valves and other openings in and around the section to be disinfected facilitate the injection and extraction of water for flushing and disinfection. Recently developed plastic foam swabs are also useful in the disinfection of mains. As they are displaced by water pressure, these swabs wipe clean the inner surface of the pipe. They can isolate the section to be disinfected from the rest of the main and prevent the loss of the disinfected solution.

Chlorine compounds are the most commonly used disinfectants for water mains. Strength of the disinfecting solution should be much higher than that normally used for water chlorination.Under normal conditions a strength of 10 mg/l is recommended for a contact period of 12-24 hours. Application for 24 hours is necessary when the chlorine has to penetrate through organic matter coating the inner surface. In emergencies, when it is not possible to leave the section of the main out of service for a long time, the period of contact can be shortened by proportionately increasing the strength of the solution. Thus for a contact period of 1 hour the strength of solution varies between 120 and 240 mg/l. When strong solutions are used particular attention should be paid to thorough removal from the main after completion of disinfection as illness and discomfort may result from using highly chlorinated water and the corrosive action of the chlorine may damage pipes, valves, hydrants and house hold plumbing and fixtures.

PROCEDURE FOR APPLICATION

Chlorine gas may be injected directly under the section of the main by a dry-feed chlorine or supplied with a special gas diffuser or silver tube and attached to a hydrant or other opening by means of specially plugged valve. After the section has been thoroughly flushed, the entire valve is partly shut to bring water pressure below 1.70 Kg/cm^2.

At the hydrant or opening where the water is discharged, the flow rate is measured to determine the rate at which chlorine gas needs to be delivered. To obtain a concentration of 10 mg/l in the section to be disinfected, the chlorine gas input rate should be 0.9 Kg/24 hours for every litres per second of flow. The valve of the chlorine cylinder is opened and adjusted so that the dial shows the required rate of chlorine flow.

To ensure that the chlorine concentration remains at 10 mg/l throughout the period of contact, the strength of the injected solution should be at least twice as high. A table below shows the amount of disinfectants required for pipes of various diameters in order to provide a chlorine concentration of about 20 mg/l

QUANTITY OF DISINFECTANTS REQUIRED TO PROVIDE CONCENTRATION OF 20 mg/l IN A 100 m PIPE LENGTH

Dia of pipe mm	Quantity in litres in which disinfectant has to be dissolved 10 × litre	Bleaching Powder (25% available chlorine) gm	Calcium Hypochlorite (70% available chlorine) gm	Sodium Hypochlorite (5%available chlorine) litres
75	46	37	13	0.16
100	81	65	23	0.33
150	183	146	53	0.73
200	325	260	92	1.30
250	507	405	145	2.03
300	730	584	210	2.92
400	1298	1040	368	5.20

The volume in litres of the disinfecting solution required for 100 m of pipe can be expressed by $V = 0.08\ d^2$ where d is the diameter of the pipe in mm.

As soon as the odour of chlorine is detected in water discharged from the main, water samples are taken to determine the chlorine content. When chlorine content reaches a value of 20 mg/l at the other end of the section being disinfected, the discharge hydrant is closed and the flow of the water and chlorine gas are stopped. The water is allowed to stand in the main for 12-24 hours and the chlorine content should be ensured to be not less than 10 mg/l at the end of the period. The mains should be thoroughly flushed with treated water until the water is cleared. Samples for bacteriological tests should be taken everyday during the 3 days following disinfection to ascertain that the water is satisfactory in quality.

A similar procedure is used for feeding a mixture of chlorine gas and water by means of a solution feed chlorinator; special rubber hose should be fitted to the plug valve and the silver tube diffuser. A booster pump may be required to provide pressure at least 3 times higher than that in the main, in order to ensure satisfactory injection of the solution.

When calcium hypo-chlorite or chlorinated lime is used for disinfection of a section of a main, the easiest method of application is to inject a strong chlorine solution by means of a portable chlorinator. If the intake valve is kept partly open, a small flow of water can enter the pipe to assist in the dispersion of the chemical. The discharge hydrant or valve is shut off when the odour of chlorine is detected in the water flowing out and the section of the main is allowed to fill. The intake valve is regulated so that the required amount of disinfecting solution is injected before the pipe is completely full.

When there is no chlorinator or pump to inject the disinfection solution, the intake valve is shut off after the flushing operation and the section is allowed to drain dry. Then the discharge hydrant or valve is shut off thus leaving the section to be disinfected, isolated from the rest of the main. The disinfecting solution is slowly poured through a funnel or a hose into an intermediate hydrant, valve or opening made for this purpose until the section is completely filled. Precaution should be taken to allow air trapped in the pipe to escape; where there is no air valve or other orifice by which the air can be released, one or more service connections could be detached or a hole could be drilled in the top of the pipe.

If the section to be disinfected is short, weighed quantities of calcium hypochlorite or chlorinated lime in powder form may be placed at regular intervals inside the pipes while they are fixed into place. When water is introduced later, the powder will mix with it and produce strong solution of chlorine. The disadvantage is that the powder will be flushed to the far end of the section even when water is admitted slowly and no uniform distribution of disinfectant is possible.

While disinfecting solution remains in two pipes, the valves and hydrants in the section of the main should be operated to ensure that all surfaces come into contact with the disinfectant. The valves at either end of the treated section should remain shut during the whole period of contact to prevent the loss of disinfecting solution.

at works
= Test pr at field.

I D P.

APPENDIX 6.4

HYDROSTATIC TEST PRESSURES FOR PIPES

S No.	Pipe IS: No.	Usual dia in mm	Class	Test Pressure at works		Maximum working pressure at Field	
				Kg/cm² = 10 m of water	Period in second.	Kg/cm²	Period
1	2	3	4	5	6	7	8
1	Spun Iron pipe IS:1536-1989 & 3114-1985	80,100,125, 150-50-500, 600,700,750, 800,900,1000, 1050	LA A B	35 35 35	15 20 25	12 18 24	
2	Cast iron pipe IS:1537-1976	80,100,125, 150-50-500, 600,700,750, 800-100-1200, 1500	A-dia(mm) Upto 600 600-1000 1000-1500 B-dia(mm) Upto 600 600-1000 1000-1500	20 15 10 25 25 20 15		Not less than two-thirds of the works test pressure maintained for the field test pressures are less, the period of test should be atleast 24 hours, the test pressure being gradually raised at the rate of 1 kg/cm²/min	
3	A.C Pressure Pipes IS: 1592-1980	50,65,80,100, 125, 150-50-500, 600	5 10 15 20 25	5 10 15 20 25	30 30 30 30 30	Maximum working pressure will be half the test pressure in each case	
4	R.C Pipes IS: 458-1988	80,100,150, 250-50-500-100-1200	P_1	2		For use on gravity mains only-working pressure not to exceed two-third of test pressure.	
	R.C. Pipes (Cont.)	80,100,150, 250-50--500-600, 700, 800, 900, 1000,	P_2	4		For use in pumping mains working pressure not to exceed half the test pressure	
		80,100,150,250, 300,350,400,500	P_3	6			

		600,700,800,		
5.	Steel cylinder R.C Pipes IS:1916-1963	200-50-500, 600,700,900 1100,1200-200-1800	1 2 3 4 5 Spl.	5 10 15 20 25
6	Prestressed concrete Pipes IS: 784-1978	80,100,125,150-50-500-100-1200-200-1800		1.5 times design pressure
7	Electrically Welded steel pipes IS : 3589-1987	200-2000	1 2 3 Spl.	15 20 25
8	M.S Tubes : 1239 (Part I) 1982	6-100 6-150 6-150	Light Medium Heavy	50 50 50

APPENDIX 6.5

DESIGN FOR ECONOMIC SIZE OF PUMPING MAIN

PROBLEM:- Design an economic size of pumping main, given the following data:

1)	Water requirements	Year	Discharge
	Initial	1989	5 MLD
	Intermediate	2004	7.5 MLD
	Ultimate	2019	10 MLD
2)	Length of pumping main	7000m	
3)	Static head for pump	50m	
4)	Design period	30 years	
5)	Combined efficiency of pumping set	60%	
6)	Cost of pumping unit	Rs. 2000 per kw	
7)	Interest rate	10 %	
8)	Life of electric motor and pump	15 years	
9)	Energy charges	Rs. 1 per unit	
10)	Design value of 'C' for C.I. pipes	100	

Solution		1st 15 years	2nd 15 years
1	Discharge at installation	5 MLD	7.5 MLD
2	Discharge at the end 15 years	7.5 MLD	10.0 MLD
3	Average discharge	5+7.5/2	7.5+10.0/2
		=6.25 MLD	=8.75MLD
4	Hours of pumping for discharge at the end of 15 years	23	23
5	Average hours of pumping for average discharge	(23/7.5)× 6.25 = 19.17	(23/10)×8.75 =20.12

6. K.W required at 60% combined efficiency of pumping set

$$\frac{7.5\times10^6\times H_1\times100\times24}{60\times60\times24\times102\times60\times23}=KW_1 \qquad \frac{10\times10^6\times H_2\times100\times24}{60\times60\times24\times102\times60\times23}=KW_2$$

$1.48H_1 = KW_1$ $\qquad$ $1.972H_2 = KW_2$

KW required = $(Q \times H)/102 \times 1/\eta \times 24/X$

Where,

Q = Discharge at the end of 15 years in 1ps

H = Total head in m for dischgarge at the end of 15 years

η = Combined efficiency of pumping set

X = Hours of pumping for discharge at the end of 15 years

7. Annual cost in Rs. of electrical energy @ Rs. 1 per unit (KWX average hours of pumping × average days per year × 1.00)

$= KW_1 \times 19.17 \times 365.24 \times 1.00$ $\quad KW_2 \times 20.12 \times 365.24 \times 1.00$

$= 7001.65\ KW_1$ $\quad = 7348.63 KW_2$

8. Pump Cost Captilised

$P_n = C = P_o (1 + r)^n$

$P_o = C/(1+r)^n$

Where,

P_o = Initial (1989) Capitalised investment

C = Amount needed after 15 years, that is, in 2004 to purchase the second stage Pumping set.

r = Rate of compound interest

= 10% per year

n = No. of years = 15

$P_o = C/(1+0.1)^{15} = C/4.177$

9. Energy Charges Capitalised

$C_c = C_R \ \{(1 - (1+r)^{-n})/n\}$

For values n =15 and r =10%

$C_c = 7.606\ C_R$

(C_c) 1st stage = 7.606 (C_R) 1st stage and

(C_c) 2nd stage = 7.606 (C_R) 2nd stage

Present (1989) energy charges (C_P) for second stage capitalised value

i.e. for (C_c) 2nd and stage in 2004

$C_p = (C_c)$2nd stage/4.177

10. Table I, II,III show the calculations to arrive the most economical pumping main size for the given data

TABLE I

TABLE SHOWING VELOCITY AND LOSS OF HEAD FOR DIFFERENT PIPE SIZE

Sl. No.	Pipe size in mm	Frictional head loss per 1000m		Velocity in m/s		Total head in 'm' for 7000 m pipe lengths including 50 m of static head					
						1st stage flow			2nd stage flow		
		1st stage flow of 7.5 MLD	2nd stage flow of 10 MLD	1st stage flow of 7.5 MLD	2nd stage flow of 10 MLD	Frictio-nal loss m	Other losses	Total	Friction-al loss m	Other losses	Total
1	2	3	4	5	6	7	8	9	10	11	12
1	300	8.00	14.50	1.25	1.68	56.00	5.60	111.60 say 115.00	101.50	10.15	161.05 say 165.00
2	350	3.80	6.70	0.88	1.00	26.6	2.66	78.26 say 80.00	46.90	4.69	101.59 say 105.00
3	400	2.00	3.40	0.72	0.87	14.00	1.40	75.40 say 75.00	23.80	2.38	76.18 say 80.00
4	450	1.10	1.95	0.56	0.75	7.70	0.77	58.47 say 60.00	13.65	1.37	65.02 say 65.00
5	500	0.66	1.15	0.45	0.66	4.62	0.46	55.08 say 55.00	8.05	0.80	58.85 say 60.00

TABLE II

TABLE SHOWING KILOWATTS REQUIRED AND COST OF PUMP SETS FOR DIFFERENT PIPE SIZE

		1st stage flow of 7.5 MLD			2nd stage flow of 10 MLD		
Sl.No.	Pipe size in mm	H_1 total head loss in mm	KW required (rounded to nearest ten including 50% standby	Cost of pump @ Rs. 2000 per KW (Rs. in thousands)	H_2 total head in mm	KW required (rounded to nearest ten including 50% standby	Cost of pump @ Rs. 2000 per KW (Rs. in thousands)
1	2	3	4	5	6	7	
1	300	115	260	520	165	330	660
2	350	80	180	360	105	210	420
3	400	75	170	340	80	160	320
4	450	60	140	280	65	130	260
5	500	55	130	260	60	120	240

Note : Assuming other losses = 10% of frictional loss

TABLE III

TABLE SHOWING COMPARATIVE STATEMENT OF OVERALL COST STRUCTURE OF PUMPING MAIN `L` FOR DIFFERENT PIPE SIZE

Sl. No.	Pipe size in mm	Total head in m		Class of CI pipes required	Rate* per m length	Cost of 7000 m pipe line	1st stage flow of 7.5 MLD				2nd stage flow of 10 MLD				Grand total of capitalized cost for 30 years (16) = (11) + (15)
		1st stage	2nd stage												
1	2	3	4	5	6	7	8*	9*	10*	11*	12*	13*	14*	15*	16*
1	300	115	165	A	640.77	4486	520	1192	9067	14073	660	2391	18186	4512	18585
2	350	80	105	LA	804.92	5635	360	829	6306	12301	420	1522	11577	2872	15173
3	400	75	80	LA	918.72	6431	340	777	5910	12681	320	1160	8823	2189	14870
4	450	60	65	LA	1106.51	7746	280	622	4731	12757	260	942	7165	1778	14535
5	500	55	60	LA	1302.66	9119	260	570	4336	13715	240	870	6618	1642	15357

8* Cost of pumpsets

9* Annual cost of energy charges = 7001.65 KW_1 = CR_1

10* Energy charges capitalised = C_{C1} = 7.606 CR_1

11* Total capitalised cost = (7) + (8) + (10)

12* Cost of pump sets

13* Annual cost of energy charges = 7348.63 KW_2 = CR_2

14* Energy charges capitalised = C_{C2} = 7.606 CR_2

15* Initial capital investment for pumpsets and annual electrical charges = (Col. 12 + Col. 14) / 4.177

* Cost of pipe includes cost of specials (10% of actual pipe cost), of excavation and cost of laying and jointing

REMARKS: From this table it is seen that most economical sizes of main is 450 mm `LA` Class C.I. pipe

APPENDIX 6.6

DESIGN OF THRUST BLOCKS

To design a thrust block for 900mm diameter main conveying water at 11kgs/cm^2 pressure(P).

The deviation angle α is 45° and density of concrete is 2300 kgs/m^3. Soil density is assumed to be 1800 kgs/m^3 and angle of internal friction $\phi = 30°$.

Assume minimum cover of earth is 600 mm. Cohesion is 0 for sandy soils.

HORIZONTAL THRUST : $F = 2\ pA \text{ Sin } \alpha/2$

Cross sectional area $A = (\pi/4)\ (90)^2 = 6364$ sq. cms.

$\text{Sin } \alpha/2 = 0.382$

$F = 2\ (11)\ (6364)\ (0.382)\ (10^{-3}) = 53.48$ Tonnes

(i) Lateral resistance to counteract the horizontal thrust:

Try a thrust block of size = 3.2M × 3.2 M × 3.2 M

Weight of thrust block = 3.2M × 3.2 M × 3.2 M × 2.3	= 75.36 tonnes
Weight of water in the pipe = $0.785 \times (0.9)^2 \times 1 \times 3.2$	= 2.03 tonnes
Weight of earth = $0.9 \times (3.2)\ (0.6)\ (1.8)$	= 3.11 tonnes
Total Weight	80.50 tonnes

Total force available considering frictional resistance of soil = 80.5 (0.3) = **24.15 tonnes**

(ii) Lateral resistance of soil against the block:

$$fp = \gamma_s \frac{(H)^2}{2} .L. \frac{1 + Sin\ \theta}{1 - Sin\ \theta} + 2CHL \sqrt{\frac{1 + Sin\ \theta}{1 - Sin\ \theta}}$$

By assuming cohesion as 0, the above equation

$$\text{Yields} = 1.8\ \frac{(3.2)^2}{2}\ (3.2)\ \frac{(1.5)}{0.5} = 88.47\ tonnes$$

(ii) Lateral resistance of soil when the thrust block is free to yield away from the soil mass i.e., the portion of projected pipes :-

$$fa = \gamma_s h \frac{1 - Sin \ \theta}{1 + Sin \ \theta} - 2C \sqrt{\frac{1 - Sin \ \theta}{1 + Sin \ \theta}}$$

$$= (1.8)(0.9) \frac{(0.5)}{1.5} = 0.54 \ tonnes$$

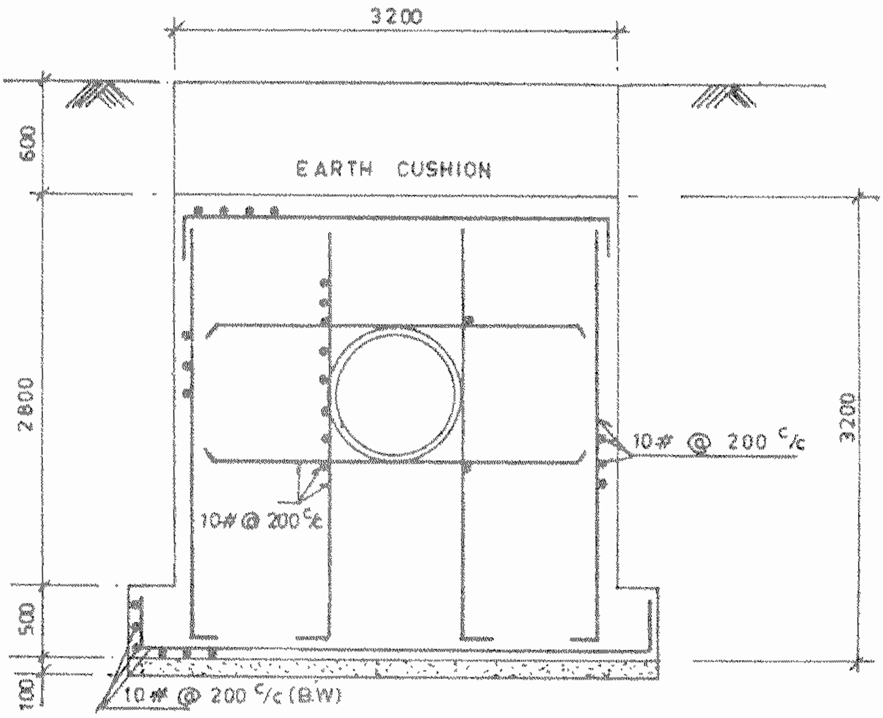
Total lateral resistance = 24.15 + 88.47 + 0.54 = **113.16 tonnes/m^2**

Total horizontal thrust = **53.48 tonnes**

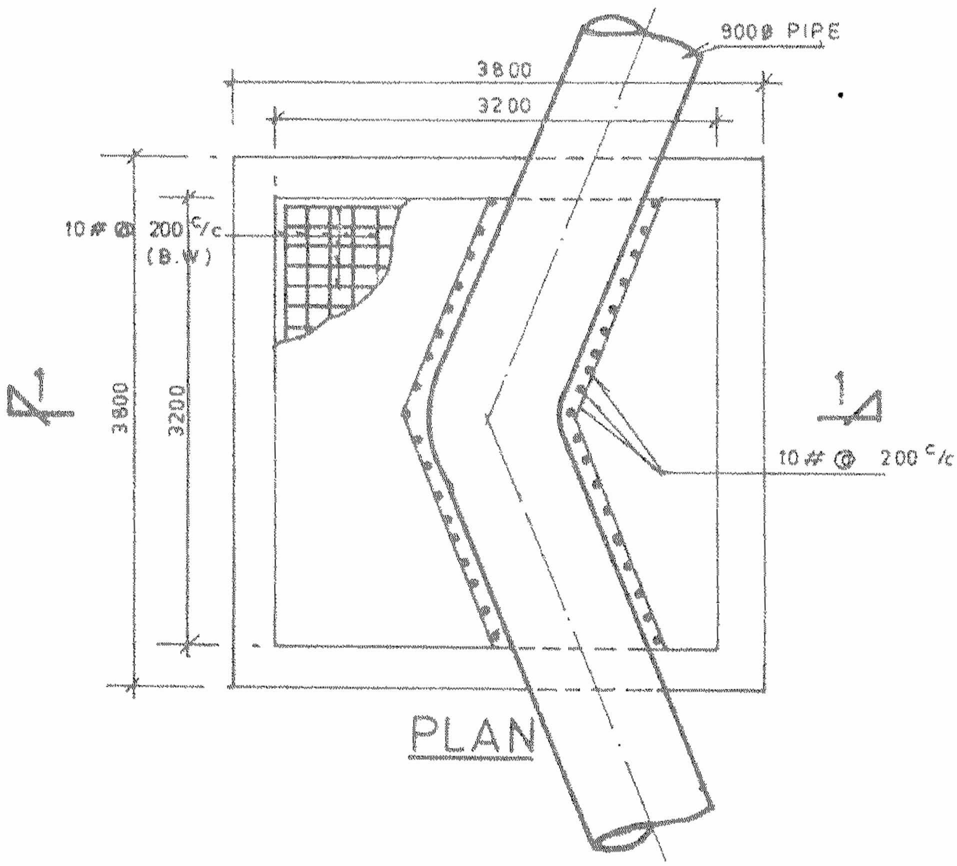
Factor of safety = 113.16/53.48 = 2.19 which is O.K.

REINFORCEMENT:

The minimum surface reinforcement in all thrust blocks shall be 5 kgs/sq.m(as per IRG 21-1972 Article 306.4). The spacing of these bars is not to exceed 500mm. Hence provide 10 bars at 200 c/c which is more than 5 kgs/sqm.



SECTION 1-1



PLAN

THRUST BLOCK FOR 45° HORIZONTAL BEND

APPENDIX 6.7

DESIGN OF AIR VESSEL

DATA

Discharge through pipe line	= 1.944 cumecs
Material of pipe line	= steel
Diameter of pipe line	= 1550 mm
Thickness of pipe (ct)	= 10 mm
Static Head	= 120 m
Length of pumping main	= 18000 m
Total Head including friction Head & other losses of 15 meter.	= 135 m
Design head of pumps.	= 150 m
Atmospheric Head	= 10 m
H_o = Absolute Head = Total head + Atmospheric Head	= 135 + 10 = 145 m

V_0 = Initial velocity $= \dfrac{1.994}{(\pi/4)(1.55)^2} = 1.03\ m/\sec$

C = Water Hammer Wave velocity.

$$= \frac{1425}{\sqrt{1+\dfrac{kd}{Ec_t}}} \qquad \text{(Refer 6.17.2 of Manual)}$$

$$= \frac{1425}{\sqrt{1+\left[\left(2.07\times10^{8}\right)\quad\left(2.1\times10^{10}\times0.01\right)\right]}} = 896.27\ m/s$$

Water Hammer head $= \dfrac{cV_0}{g} = \dfrac{896.27\times1.03}{9.81}$

$= 94.10$ meters

Pipeline Parameter $\rho = \dfrac{cV_0}{2gH_o}$

$= \dfrac{896.27\times1.03}{2\times9.81\times145}$

$= 0.324$

$2\rho = 0.65$

AIR VESSEL PARAMETER = $2C_oC/Q_oL$

Referring Chart $f_{o,}$ k_c as 0.50 for limiting Upsurge to 1.20 H_o and Downsurge to 0.5 H_o Air vessel parameter for $2\rho = 0.65$ is calculated as follows:

From chart for $2\rho = 1.00x(2C_oC/Q_oL)=10.50$
For $2\rho = 0.5$, $2C_oC/Q_oL = 6.50$

By interpolation for $2\rho = 0.65$ and for $k_c = 0.5$

Air vessel parameter $2C_oC/Q_oL = 7.70$

$$\text{Volume of air } C_o = \frac{7.7x1.944x18000}{2x896.27}$$

$$= 150.31 \text{ Cubic meters}$$

$$\text{Volume of Air Vessel} = C_o[H_o/H_{min}]^{1/1.20}$$
$$= 150.31[(145)/0.5x(145)^{1/1.20}]$$
$$= 150.31x(2)^{1/1.2}$$
$$= 267.20 \text{ Cum}$$

Increase the capacity by 20% to cater for upsurge of 1.20 H_o

$$= 267.20 \text{ x } 1.2$$
$$= 320 \text{ Cums}$$

WATER COLUMN SEPERATION LENGTH

The water column separation is calculated on the basis of the following formula.

$$V_1^2 - V_2^2 = (2g/L)\{(t_1-t_2)\ V_i\ \{H+F\ (V_1^2/V_o)\}\}$$

H= Static Head, (Absolute Head)

F= Loss of head due to friction

V_1, V_2=Velocities at instances $t_1$and t_2.

$(t_1 - t_2)$ = Period between time intervals in seconds .

V_o = Initial Velocity.

L=Length of pipeline

Initial velocity will come to rest over a time period after the stoppage of pumps. Assuming a time interval of 0.20 seconds and by using above formula the subsequent velocities are calculated till the final velocity (V_n) is almost Zero. The water column separation length l is given by Laws =

$$lwcs\ \Sigma\ [V_1+V_2+\ldots\ldots V_n]\ (t_2-t_1)$$

For the given diameter of pipe and for the calculated water column separation Length the volume of water required to be stored in Air vessel is calculated.

For Worked Example

$$(1.03)^2 - V_2^2 = 2 \times \frac{9.81}{18000}(0.20)(1.03)\left[145 + 15\frac{(1.03)^2}{1.03}\right]$$

$$(1.01)^2 - V_3^2 = 2 \times \frac{9.81}{18000}(0.20)(1.01)\left[145 + 15\frac{(1.03)^2}{1.01}\right]$$

Repeat n times till $V_n = 0.01$ m/sec.

Then $V_1+V_2+V_3+\ldots\ldots\ldots\ldots V_n\ (0.20)$=say =6.1 meters.

For a pipe of 1.55 per dia volume of water required to fill this separation length

$$= \frac{\pi}{4}(1.55)^2(6.10) \quad = \quad 11.51 \text{ Cum}$$

Fixing The Size Of Vessel And Levels Of Water And Air In Airvessel Chamber

(i) Air And Water Volume

Air Vessel volume required = 320 Cum.

If two vessels are provided volume of each vessel = 160 Cum.

Provide 90 Cum of Air and 70 Cum of water in each vessel.

(ii) Determination Of Size Of Air Vessel

Absolute Head at working head of pumps = 150 + 10.35 = 160.35 meters.

Maximum upsurge permitted 160.35 x1.2 = 192.42 meters

Pressure = 19.25 kg/ cm^2

Using 25 mm thick M S Plate i. e 22 mm +3mm for corrosion allowance

$$d = \frac{2f_t \times e \times t}{p}$$

f_t = Permissible tensile strength in steel plates = 1260 kgs/cm^2

e = Weld efficiency say 0.9

t = Thickness in cms of plate = 2.2 cm

p = Pressure in kgs/cm^2

$$= \frac{2 \times 1260 \times 1.90 \times 2.20}{19.25}$$

$= 259.20$ cms

$=$ Say 260 cms

Provide 2.60-m dia of vessel with a length L and two hemi-spherical ends.

Volume of (two hemispheres) spherical portion $= \frac{4}{3} \times \pi(1.3)^3 = 9.2$ Cum.

Total Volume of cylinder = 160 cum – 9.20 = 150.80 cum

Length of vessel of 2.6 m dia with volume 150.80 cum is = 28.40 meters

Provide 2 vessels each of 2.6-m dia and 28.40 m long with hemi-spherical ends.

(iii) Fixing Of Levels Of Water And Air In The Vessel

The levels are fixed by trial by assuming a depth and calculating volume in cylindrical and spherical portions.

(a) Normal Working Level

Volume of Air = 90 cum

Volume of Water = 70 cum

The normal working level is fixed by trial by assuming 1.15 meter of water depth from bottom. Volume of water = 70.95 Cum which is more than required 70 Cum. Hence normal working level will be at 1.15 m from bottom of vessel.

(b) Upper Emergency Level

Air dissolves in water in the vessel. Assuming that 10% Air dissolves in water the level of water rises by 10% of volume of Air i.e.

Volume of water = 70 Cum +10% of 90 Cum = 79 Cum.

The depth of water from bottom will be 1.35 m which gives volume of water as 79 Cum. Hence upper emergency level will be 1.35 m from bottom of vessel.

(c) Lower Emergency Level

When pumps trip as per water column separation about 11.51 Cum of water is required to fill the pipeline. As calculated volume of water at a depth of 1.00 m from bottom of vessel = 56.43 Cum . Volume of water at normal working level is 71 Cum.

Quantity of water available is the difference between normal working level and lower emergency level.

APPENDIX 7.1

DESIGN OF SPRAY TYPE AERATOR

(Removal of Iron & Manganese)

I. PROBLEM STATEMENT

Design a spray aerator given the following data:

1. Design flow = 250 m^3/hr.

   Pipe used = 70 mm dia 'B' Class C.I. Pipe with a C value of 100

   S = 3.60 meters /1000 meters

   V = 1.35 m/s

2. Iron present in raw water = 1.8 mg/l.
3. Saturation concentration of O_2 at 28 ^{0}C = 7.92 mg/l.
4. Aeration constant (to the common base) at 28 ^{0}C = 70 cm/hour.

II. DESIGN CRITERIA

1. Nozzle dia. usually 10 to 40 mm – spaced in the pipe at intervals of 0.5 to 1.0 m
2. Nozzles are usually tilted 3^0 to 5^0 to the vertical to avoid interference due to falling water.
3. Nozzle discharges should be uniform as far as possible. Variation in no case should be greater than 5% i.e. the discharge ratio between the first and the last nozzle, should not be less than 0.95(a variation 2 to 5% may be allowed).
4. Velocity of water in the aerator pipe should be between 1 and 1.5 m/s .
5. Pressure required at the nozzle varies from 2 to 9 meter of water (usually 7m).
6. Discharge ratings per nozzle vary from 300 to 600 lpm.
7. Aerator area should be 1.25x 10^{-3} to 3.75 x10^{-3} m^2 per m^3/day of design flow.

III. SOLUTION

1. Design flow = 6000 m^3/day.
2. Assuming 25-mm dia. nozzle with an inclination of 3^0 to the vertical, dia of one drop is 25 mm.
3. Iron present in raw water = 1.8 mg/l.

   Permissible limit of iron in treated water = 0.1 mg/l.

   Iron to be removed = (1.8-0.1) mg/l = 1.7 mg/l.

4. $Fe^{++} + 3O_2 = 2Fe_2O_3$

   1.7 mg/l of Fe requires 1.7 x96/224 = 0.7286 mg/l of O_2.

5. By applying 'Gas absorption equation in 7.2.2 in the form

$$Log_{10}[(C_s - C_0)/(C_s - C_t)] = \frac{K.A.t}{V}$$

where

C_s = 7.92 mg/l at 28^{0}C, Co = 0.0 mg/l.

C_t = 0.73 mg/l, K = 70 cm/hr

$$\frac{A}{V} = \frac{6}{d} = \frac{6}{2.5}\left(\frac{1}{cm}\right)$$

$$\therefore \log_{10}\frac{7.92}{7.19} = \frac{70}{60x60} \times \frac{6}{2.5} \times t = \frac{7}{150}t$$

$$\therefore t = \frac{150}{7} \times \log_{10}\frac{7.92}{7.19}$$

$$t = \frac{150}{7} \times 0.042$$

t=0.9 seconds

say t = 1 second & small case

t_r = time of rise = t/2 = 0.5 seconds

V = nozzle velocity and α = inclination to horizontal.

V Sin α = g.t_r

$$\therefore V = \frac{gt_r}{Sin\alpha}$$

= (980x0.5)/Sin (90-3)^0

= (980x0.5)/Sin 87^0

= 4.91 cm/s

6. Number of nozzles

Assuming:

N = No. of nozzles required

q = Discharge through each nozzle = C_d x V x a

where,

Cd = Coefficient of discharge = 0.9 (assuming)

V = nozzle velocity = 4.91 mps

a = nozzle area = (3.14/4)d^2

d= dia of nozzle = 25 mm

$\therefore$ Discharge through "N" number nozzles = N x C_d x V x a

= N x 0.9 x 4.91x (3.14/4) x $[25x10^{-3}]^2$ m^3/sec.

But design flow i.e. discharge through N nozzles = 6000 m^3/day

N x 0.9 x 4.91x (3.14/4) x $[25x10^{-3}]^2$x 60 x 60 x 24 = 6000 m^3/day

$\therefore$ N = 32

$\therefore$ Nozzles required = 32 Nos of 25 mm dia each.

7. **Spacing of Aerator Pipes**

Radius of spray = V Cosα x 2t_r = 4.91 Cos 87^0x2x0.5 = 0.257 m

Assuming wind velocity = 8 km/hr.

Wind Drag = Cd x Vw x t (assuming C_d = 0.6)

= 0.6 x [(8 x 10^3)/(60 x 60)] x 1 = 1.33 m

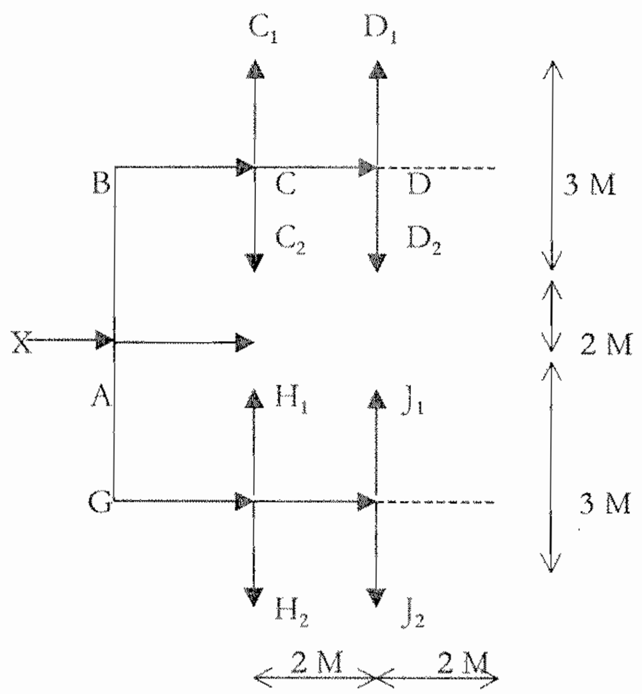
$\therefore$ Minimum spacing required = Radius of Spray + Wind drag

= 0.257 + 1.33

= 1.587 m say = 2m apart

8. **Arrangement of nozzles**

Nozzles are fixed on 4 rows of pipes as shown below;



No. of nozzles in each pipe = 32/4 = 8

Providing a spacing of 0.3 m c/c of nozzles and spacing in two adjacent rows and in staggered position.

Provide 4 pipes each of length 3m at a spacing of 2m.

Allowing 2m space on all the sides, the size of the aerator tray will be 8m x 6m.

Checking

Aerator pipes enclose an area of 2 x (3x2) = 12 m^2

$\therefore$ Area provided per m^3/day of design flow = 12/(250x24)= $2.0 \times 10^{-3} m^2/m^3$/day of design flow .

(O.K. since it is between 1.25×10^{-3} to $3.75 \times 10^{-3} m^2$/day of design flow).

9. Uniformity in distribution

The uniformity in distribution of water is maintained by arrangement of aerator pipes as in figure above.

Discharge through each pipe = (250x24)/4 = 1500 m^3/d

Assuming h = head loss at each nozzle

$V = C_v \sqrt{(2gh)} = 4.91$ m/s

$h = (4.91)^2 / [(0.9)^2 x2x9.81]$ (Assuming $C_v = 0.9$)

Assuming variation of head = 2%

$$m_1 = \frac{\text{discharge through last nozzle in the pipe}}{\text{discharge through first nozzle in the pipe}} = 0.98$$

$H = h (1-m_1^2) = 1.52(1-0.98^2) = 0.06$ m

Head loss in the pipe for gradually diminishing flow = H = 0.06 m

$\therefore$ Corresponding head loss for uniform flow = H_u = 3H =3x0.06

= 0.18 m (per aerator pipe length)

$\therefore$ Head loss /1000 m = (0.18 /1.5) x1000 = 120

10. Design of pipes and head losses:

The arrangement of pipe is shown in Figure. The aerator pipes are so chosen that the velocity remains within 1 to 1.5 mps and corresponding head losses for pipes (C.I.) are calculated and are shown in the following table:

Pipe Section.	Design Flow (m^3/d)	Length (m)	Dia. (m)	Velocity (m/sec)	Head Loss(m)	Total Head Loss(m)
1	2	3	4	5	6	7
AB	3000	2.5	200	1.1	0.01	0.025
BC	3000	2.0	200	1.1	0.01	0.020
C_1C_2	1500	3.0	125	1.42	0.03	0.090
CD	1500	2.0	125	1.42	0.03	0.060
D_1D_2	1500	3.0	125	1.42	0.03	0.090
					Total	0.285

Total Head loss = 0.285 + 10% for valves and specials

= 0.314 m

say = 0.32 m

Head at 'A' = Terminal head + Total Head loss

= 1.52 + 0.32

= 1.84 m

APPENDIX 7.2

DESIGN OF MECHANICAL RAPID MIX UNIT

1. PROBLEM STATEMENT

Design a mechanical rapid mix unit using following data:

1. Design flow to be treated = 250 m^3/hr
2. Detention time = 30 secs (20-60 s)
3. Ratio of tank height to diameter = 1.5:1 (1-3:1)
4. Ratio of impeller diameter to tank diameter = 0.4:1(0.2 – 0.4:1)
5. Rotational speed of impeller = 120 rpm (>100 rpm)
6. Velocity gradient = $600S^{-1}(>300S^{-1})$
7. Assume temperature of 20^0 C.

2. SOLUTION

(i) Determine dimensions of tank

Volume = Flow x detention time

= 250 x (30/3600)= 2.083 m^3

Diameter of the tank, D, is calculated from $(\pi/4)D^2(1.5D) = 2.083$

Therefore, diameter of tank = 1.20 m

and height of tank = 1.80 m

Total height of the tank = 2 m which will provide a free board of 0.2 m

(ii) Compute power requirements

Power spent, P = μG^2. (Volume of tank)

= $1.0087 \times 10^{-3} \times (600)^2 \times 2.083 = 756$ watts

Power per unit volume = 756/2.083 = 362.94

= Say 363 watts/m^3

Power per unit flow of water = 756/250 = 3.02 watts/m^3/hr of flow

Determine dimensions of flat blades and impeller

Diameter of impeller = 0.4 x tank diameter

= 0.4 x 1.2 = 0.48 m

Velocity of the tip of impeller = $(2.\pi.r.n/60)$ m/s

= $((2\pi \times 0.48)/2) \times (120/60)$) m/s

$= 3.02$ m/s (O.K.)

Determine the area of blades A_p of impeller by the equation.

Power spent $= (1/2).C_D.\rho.A_p.V_r^3$

Assuming Newtons coefficient of drag, $C_D = 1.8$ for flat blades and relative velocity of paddle, V_r, as three fourths of the tangential velocity of the tip of the blade,

$756 = (1/2) \times (1.8 \times 1000 \times A_p) \times [0.75 \times 3.03]^3$

$A_p = 0.072\ m^2$

Provide 6 blades of size 0.1x0.12 m

(iv) Provide 4 Nos of baffles of length 1.9 m and projecting 0.10 m from the wall of the tank to reduce vortex formation.

(v) Provide inlet and outlet pipes of 200 mm diameter.

APPENDIX 7.3
DESIGN OF CLARIFLOCCULATOR

I. PROBLEM STATEMENT

Design clariflocculator using following data and design criteria:

1. Desired average outflow from clariflocculator = $250\ m^3 / hr$
2. Water lost in desludging = 2%
3. Design average flow = (250 x 100) / (100 - 2)
   = 255.1 m3 / hr
4. Detention period = 20 minutes
5. Average value of velocity gradient, G = $40S^{-1}$

II. COMPONENTS TO BE DESIGNED

A circular clariflocculator is to be designed having vertical paddles. The water enters through a central influent pipe and is fed into the flocculation zone through parts. The effluent from flocculation zone passes below the partition wall dividing the flocculator portion and the clarifier portion. The clarified effluent is collected by a peripheral effluent launder. The components of clariflocculator to be designed include the influent pipe, the flocculator, the clarifier and the effluent launder.

III. DESIGN OF INFLUENT PIPE

Assuming a velocity of 1 m/s

$$\text{Influent pipe diameter} = \sqrt{\frac{255.1 \times 4}{3600 \times 1 \times \pi}} = 0.24$$

Provide an influent pipe of 300 mm diameter

IV. DESIGN OF FLOCCULATOR

Dimensionless Parameter, G.t = 40 x (20 x 60) = 4.8×10^4

This is acceptable as G.t = 2 to 6×10^4 for alum coagulants

Volume of flocculator = (255.1 x 20) / 60 m^3 = 85 m^3

Provide a water depth of 2.5 m

Plan area of flocculator = 8.5 / 2.5 m^2 = 34 m^2

Let D be the diameter of flocculator and D_p the diameter of the inlet pipe. Then

$$\frac{\pi}{4}\left(D^2 - D_P^2\right) = 34, \qquad \frac{\pi}{4}\left(D^2 - 0.3^2\right) = 34$$

D = 6.57 m

Provide a tank diameter of 6.6 m.

V. Dimensions Of Paddles

Total power input to flocculator, $P = G^2 \mu$ (vol.)

$(40)^2 \times [0.89 \times 10^{-3}] \times [\pi \times (6.6)^2 \times 2.5/4] = 122$ watts

Power input $= (1/2) .C_D. \rho. A_p (V - v)^3$

Where

$C_D =$ Newtons coefficient of drag, 1.8

$\rho =$ Density of water at 25^0C, 997kg/m^3

$V =$ Velocity of the tip of blades

$= 0.4$ m/s (recommended range 0.3-0.4 m/s)

$v =$ Velocity of water at tip of blade

$= 0.25 \times 0.4$ (25% of V)

$= 0.1$ m/s.

$122 = 1.8 \times 997 \times Ap (0.4 - 0.1)^3 / 2$

$A_p = 5.04$ m^2

Ratio of area of paddles to cross-sectional area of flocculator

$= A_p / \pi (D - D_p) \times h$

$= (5.04) / (\pi .(6.6 - 0.3) \times 2.5) = 0.102$ or 10.2%

This is acceptable as it is within the limits of 10 to 25%

Provide 8 Nos. of paddles of height 2.0 m and width of 0.32 m

Two shafts will support eight paddles, each shaft supporting 4 paddles. The shaft will be at a distance of $(6.6 - 0.3)/4 = 1.58$ m from the centre line of clariflocculator. The paddles will rotate at a rpm of 4.

Distance of paddle edge, r, from the centre line of vertical shaft is given by the equation.

$V = (2.\pi.r.n) / 60$

$0.4 = (2. \pi.r \times 4) / 60$

$\therefore r = 1$ m

Let the velocity of water below the partition wall between the flocculator and clarifier be 0.3 m / minute. Therefore area of opening required for a velocity 0.3 m/min below the partition wall will be

Area $= 250 / (0.3 \times 60) = 13.9$ m^2

Depth below partition wall

= 13.9/ (π x 6.6) = 0.67m

Provide 25% additional depth for the storage of sludge in case the mechanical scraper is out of order.

Depth provided for sludge storage = 0.25 x 2.5 = 0.625 m say 0.63 m

Provide 8% slope for the bottom.

Total depth of tank at the partition will assuming a free board of 0.3 m

= 0.3 + 2.5 + 0.67 + 0.63 = 4.10 m.

VI. Design Of Clarifier

Assume a surface overflow rate of 40 $m^3/m^2/day$

Surface area of clarifier = 255.1 x 24 /40 = 153.06m2

Diameter of the clariflocculator, D_{cf} is given by

$$\frac{\pi}{4}\left[D_{cf}^2 - (6.6)^2\right] = 153.06 \qquad \therefore D_{cf} = 15.44m$$

Length of weir = $\pi.D_{cf}$ = π x 15.44 = 48.53m

Weir loading = (255.1 x 24)/(48.83) m^3/day.m = 126.2m^3/day m (< 300 m^3/day m) O.K

APPENDIX 7.4

DESIGN OF RECTANGULAR PLAIN SEDIMENTATION TANK

I. PROBLEM STATEMENT

Design rectangular sedimentation tank with following data.

1. Desired Average Outflow from sedimentation tank	=	250 m^3/hr.
2. Water lost in desludging	=	2%
3. Design Average flow	=	(250x100)/(100-2)
	=	255.1m^3/hr
4. Minimum size of the particle to be removed	=	0.02 mm
5. Expected removal efficiency of min. size particle	=	75 %
6. Nature of particles	=	discrete and non flocculating
7. Specific gravity of particles	=	2.65
8. Assumed performance of the settling tank	=	good (n = 1/4)
9. Kinematic viscosity of water at 20 ° C	=	1.01×10^{6} m^2/s

II. DESIGN PROCEDURE

For the given diameter and specific gravity of minimum size particles to be removed in settling tank, vertical settling velocity of the particle is calculated initially using Stoke's law. The computed settling velocity is used to determine Reynolds number to check whether Stoke's law is applicable. If Reynolds number exceeds 1, Hazen's formula is used to determine the settling velocity of particle. The settling velocity thus calculated is employed for computation of surface over flow rate for expected removal efficiency of minimum size particles and assumed performance of the settling basin. Alternatively the surface over flow rate for average design flow may be assumed on the basis of data presented in Table in section 7.5.6. The plan area is determined next, followed by tank dimensions. The depth of tank may be determined using detention period. Sizing of components of inlets and outlets is done using relevant design criteria & assumptions.

III. DESIGN STEPS

1. Compute vertical settling velocity of minimum size particles.

$$v_s = g(S_s-1).d^2 / (18 \times \nu)$$

$$= 9.81(2.65-1)(0.02 \times 10^{-3})^2 / (18 \times 1.01 \times 10^{-6})$$

$$= 3.56 \times 10^{-4} \text{ m/s}$$

Reynolds number = $(v_s.d) / \nu$

$= 3.56 \times 10^{-4} \times (0.02 \times 10^{-3}) / (1.01 \times 10^{-6})$

$= 704 \times 10^{-3} < 1$

Hence Stoke's law is applicable and computed settling velocity is correct.

2. Determine Surface Overflow Rate

For Ideal settling basin and complete removal of minimum size particles, equate settling velocity to theoretical surface over flow rate for 100% removal.

$V_s = V_o$

$V_o = 3.56 \times 10^{-4}$ m/s

$= 3.56 \times 10^{-4} \times 3600 \times 24 = 30.76$ m/d

However due to short circuiting, there is reduction in efficiency and decrease in surface overflow rate. To obtain design surface overflow rate, which would give expected removal efficiency of minimum size particles in real basin, use following relationship.

$y / y_o = 1 - [1 + n. (V_o / (Q/A))]^{-1/n}$

For $y / y_o = 0.75$, $n = 1/4$ (good performance of tank)

$V_o / (Q/A) = 1/ n\{[1- y / y_o]^{-n} - 1 \}$

$= 4 \times [(1 - 0.75)^{-1/4} - 1] = 1.66$

Hence Design Surface overflow rate at average design flow, Q/A

$Q/A = (V_o / 1.66) = 30.76/1.66 = 18.53$ m/d

Typical values for design surface overflow rate range between 15 and 30 $m^3/m^2/d$. for plain sedimentation tanks.

3. Calculate Dimensions Of Tank

Surface area of tank, A $= (Q / (Q/A))$

$= 255.1[m^3/hr] \times 24 / 18.53$

$= 330.4\ m^2$

Assume length to width ratio as 4

Length x width = surface area

$Width,\ B = \sqrt{(330.4/4)} = 9.09$

Length of tank, L = 36.36 m

Assume detention period, t, as 4 hrs.

Water depth of settling zone at average flow $= Q \times t / A$

$= 255.1 \times 4 / (36.36 \times 9.09) = 3.09$ m

4. Check Against Resuspension Of Deposited Particles

Flow velocity that can initiate resuspension of deposited particles in the sludge zone, V, is given by

$$V = \sqrt{[(8k/f)g(S_s - 1)d]}$$

For unigranular particles k = 0.04 and

Weisbach - Darcy friction factor, f = 0.03

$$V = \sqrt{[(8 \times 0.04/0.03) \times 9.81 \times (2.65-1) \times (0.02 \times 10^{-3})]}$$
$$= 5.88 \times 10^{-2} \text{ m/s}$$

To avoid resuspension, this critical displacement velocity should not be exceeded and horizontal velocity of flow in basin should be less than critical displacement velocity. Horizontal velocity of flow in settling basin at average flow, V_h

$$V_h = Q/(B \times D)$$
$$= 255.1\ [m^3/hr]/(3600 \times 9.09 \times 3.09)\ m/s = 2.52 \times 10^{-3}\ m/s < 5.88 \times 10^{-2} \text{ hence O.K.}$$

5. Influent Structure

The influent structure is designed to minimize turbulence, to distribute the water and suspended solids uniformly across the width and throughout the depth of settling basin and to avoid deposition of suspended solids in influent structure. It may consist of an influent channel, submerged orifices and baffles in front of orifices.

Provide 0.6 m wide and 0.6-m deep influent channel that runs across the width of the tank. Provide 4 submerged orifices 0.20 m x 0.20 m in the inside wall of influent channel to distribute the flow uniformly into the basin. A baffle 1 m deep is provided at a distance of 1 m away from orifices to reduce turbulence.

Velocity of flow in channel = 255.1 / (3600 x 0.6 x 0.4)

at average design flow

(Assuming a depth of flow of 0.4 m) = 0.3 m/s.

Head loss through orifices $= \left[\dfrac{255.1}{3600 \times 4 \times 0.6 \times (0.2)^2 \times \sqrt{2 \times 9.81}}\right]^2 = 0.03m$

6. Effluent Structure

The components of effluent structure are effluent weir, effluent launder, outlet box and an outlet pipe.

(a) Compute weir length & number of V-notches

outflow from sedimentation tank = 250 m^3/hr.

Assuming a weir loading of 200 m^3/d per m length of weir,

Weir length $= (250 \times 24) / 200 \quad = 30$ m

No. of 90° V-notches assuming centre to centre spacing of 200 mm.

$= 30 \times 1000 / 200 \quad = 150$

(b) Provide 30-m length of effluent launder with V-notches fixed only on one side of the launder. For a 0.30-m wide effluent launder, the critical depth at the end of effluent launder can be computed from

$$y_2 = [(q'L)^2 / (b^2 \times g)]^{1/3}$$

$$= [(250/(2 \times 3600))^2 / ((0.3)^2 \times 9.81)]^{1/3} = 0.11 \text{ m}$$

Depth of water at upper end of the trough, y_1, is

$$= [y_2^2 + 2(q'LN)^2 / (gb^2y_2)]^{1/2}$$

$$= [0.11^2 + [2 \times [\{250/(2 \times 3600) \times 1\}^2/\{9.81 \times (0.3)^2 \times 0.11\}]]^{1/2}$$

$$= 0.19 \text{ m}$$

Accounting for head loss due to frictional resistance in the launder channel and the free board, a depth of launder of 0.4 m may be provided.

APPENDIX 7.5

DESIGN FOR RADIAL CIRCULAR SETTLING TANK

I. PROBLEM STATEMENT

Design a secondary circular sedimentation tank to remove alum floc with following data.

1. Average output from settling tank	=	250 m^3/hr.
2. Amount of water lost in desludging	=	2 %
3. Average design flow	=	255.1m^3/hr.
4. Minimum size of alum floc to be removed	=	0.8 mm.
5. Specific gravity of alum floc	=	1.002
6. Expected removal efficiency of alum floc	=	80%
7. Assumed performance of settling tank	=	Very good (n=1/8)
8. Kinematic viscosity of water at 20°C	=	$1.01 \times 10^{-6} m^2/s$

II. DESIGN SOLUTION STEPS

Calculate the settling velocity of particles

$$Vs = g (S_s-1) d^2/(18 \times \nu)$$

$$= 9.81(1.002-1)(0.8 \times 10^{-3})^2 / (18 \times 1.01 \times 10^{-6}) \text{ m/s} = 6.91 \times 10^{-4} \text{ m/s}$$

Reynolds number = $N_R = (V_s d)/\nu = (6.91 \times 10^{-4} \times 0.8 \times 10^{-3})/ 1.01 \times 10^{-6} = 0.55 < 1$

Hence Stoke's law is applicable.

2. COMPUTE SURFACE OVERFLOW RATE, SOR

For ideal basin and complete removal of wanted particles

$V_s = V_o$

$V_o = 6.91 \times 10^{-4}$ m/s = 59.7 m/d.

However due to short circuiting etc., basin efficiency is reduced and to achieve desired removal efficiency, the surface overflow rate has to be decreased.

$Y / Yo = 1 - [1 + nVo/(Q/A)]^{1/n}$

For given values of y/yo = 0.8, n = 1/8

$Vo/(Q/A) = 1.78$

$Q/A = 59.9/1.7 = 33.49$ m/d O.K.

This is acceptable as it is within the typical design range of 30-40 $m^3/m^2/d$.

3. DETERMINE THE DIMENSION OF TANK

Surface area of tank, $A = Q/(Q/A)$

$= 255.1 \times 24/33.49 = 182.8 \text{ m}^2$

Hence diameter of tank = 15.26 m

Assume detention period, t, of 2.5 hours as given in Table

Depth of tank = $Q \times t/A = 255.1 \times 2.5 / 182.8 = 3.49$ m say 3.5 m

4. **CHECK FOR WEIR LOADING**

Weir length = periphery of the tank = $\pi D = \pi \times 15.25 = 47.94$ m

Weir loading = $255.1 \times 24 / 47.94 = 127.7 \text{ m}^3/\text{d.m} < 300 \text{ m}^3/\text{d/m}$

Hence O.K.

APPENDIX 7.6

DESIGN FOR TUBE SETTLERS

1. PROBLEM STATEMENT

Design tube settler module of square cross section with following data

1. Average output required from tube settler = 250 m^3/hr
2. Loss of water in desludging = 2% of output required
3. Average design flow = (250 x 100) / (100 - 2) = 255.1 m^3/hr
4. Cross section of square tubes = 50 mm x 50mm
5. Length of tubes = 1 m
6. Angle of inclination of tubes = 60^0

2. DESIGN STEPS

1. Compute relative length of settler

L_R = 1000 / 50 = 20

Effective relative length of tube, L

$$L = L_R - 0.058\ N_R$$

$$= L_R - 0.058 \times V_o d / \nu$$

$$= 20 - (0.058 \times Vo \times 0.05) / (1.01 \times 10^{-6} \times 86400)$$

$$= 20 - 0.033\ V_o$$

where V_o is flow through velocity for tube settler in m/d

3. DETERMINE FLOW VELOCITY THROUGH TUBES

$S_c = V_{sc} / V_o \times (\sin\theta + L\cos\theta)$

$11/8 = (120/ V_o) \times (\sin 60 + (20 - 0.03\ V_o)\cos 60\ )$

V_o = 388.65 m/d

4. COMPUTE TOTAL TUBE ENTRANCE AREA AND NO. OF TUBES

Tube entrance area = Q/ V_o = 255.1 x 24 / 388.65 = 15.75 m^2

No. of tubes required = 15.75 / (0.05 x 0.05) = 6300

Provide 6400 square tubes of 0.05 m x 0.05 with 80 tubes along the length of the square module and 80 tubes along the width of the module.

Length of the tube module = No. of tubes x (inside dimension of square tubes + 2 x thickness of tubes)

= 80 x (0.050 + 2 x 0.005) m

$= 4.24$ m

Height of tube module for 1m length of square tubes inclined at an angle of 60^0

$= 1 \sin 60° = 0.866$ m say 0.87 m

Therefore overall dimension of tube module $=$ 4.24 m x 4.24 m x 0.87 m

Size of individual square tubes $=$ 0.05 m x 0.05m

Thickness of individual square tubes $=$ 1.5 mm

APPENDIX 7.7

DESIGN FOR RAPID GRAVITY FILTER

1. Problem Statement

Design rapid gravity filter for producing a net filtered water flow of 250 m^3/hr. The relevant data is

(i) Quantity of backwash water used	=	3 % of filter output
(ii) Time lost during back washing	=	30 minutes
(iii) Design rate of filtration	=	5 $m^3/m^2/hr$
(iv) Length to width ratio	=	1.25 – 1.33: 1
(v) Under drainage system	=	Central manifold with laterals
(vi) Size of perforations	=	9 mm

2. Solution

(a) Filter Dimensions

Required flow of filtered water	=	250 m^3/hr
Design flow for filter after accounting for backwash water and time	=	250 x (1+0.03) x 24 /23.5 m^3/hr
lost in backwashing	=	263 m^3/hr
Plan area of filter required	=	263/5 = 52.6 m^2

Provide two filter units, two being minimum no. to be provided

Length x width	=	26.3
Assume length to width ratio as 1.3: 1		
Width of the filter	=	$(26.3/1.3)^{0.5}$ = 4.50 m
Length of the filter	=	5.85 m

Provide two filter units, each with a dimension of 5.85 x 4.50 m

(b) Estimation Of Sand Depth

Assume a depth of sand as 60 cm and effective size of sand as 0.5 m.

The depth can be checked against break through of floc through sand bed by calculating minimum depth required by Hudson formula

$$\text{In F.P.S unit } Qd^3h/1 = B \times 29323$$

Where Q is the rate of filtration in gpm/sft, d is the sand size in cm, h is the terminal loss of head in ft., l is the depth of bed in inches and b is a breakthrough index whose value ranges between 4×10^{-4} to 6×10^{-3} depending on response to coagulation and degree of pre-treatment of filter influent, In metric units $Qd^3h/l = B \times 29323$

Where Q is in $m^3/m^2/h$, d in mm and h & l are in m.

Assume $B = 4 \times 10^{-4}$ for poor response to filtration and average degree of pre-treatment, terminal head loss of 2.5 m, rate of filtration = 5 x 2 = 10 $m^3/m^2/hr$. (Assuming 100 % overloading of filter under emergencies), and assuming d= 0.6 mm as mean diameter,

$10 \times (0.6)^3 \times 2.5 / 1 = 4 \times 10^{-4} \times 29323$

Minimum depth of sand required to avoid breakthrough = 46 cm. Hence the assume depth of 60 cm to be adequate to avoid break through of floc.

(c) Estimation Of Gravel And Size Gradation

Assume a size gradation of 2mm at top to 50mm at the bottom .The requisite depth 1 in inches of a component gravel layer of size d in inches can be computed from empirical formula

$l = k(\log d + 1.40)$

Where k varies from 10 to 14. The equivalent formula in metric units where l is in cm and d is in mm is

$l = 2.54\, k(\log d)$

For k=12, the depth of various layers of gravel are

Size, mm	2	5	10	20	40
Depth, cm	9.2	21.3	30.5	40	49
Increment, cm	9.2	12.1	9.2	9.5	9

Provide a gravel depth of 50 cm

(d) Design Of Under Drainage System

Plan area of each filter	= 5.85 x 4.50 = 26.33 m^2
Total area of perforations	= 3×10^{-3} x Area of filter
	= 0.0789 m^2
	= 790 cm^2
Total number of perforation of 9 mm dia	= $790/((\pi/4)(0.90)^2)$ = 1241.8
	Say 1242
Total cross sectional area of laterals	= 3 x Area of perforations
	= 3 x 790 = 2370 cm^2
Area of central manifold	= 2 x Area of laterals
	= 2 x 2370 cm^2

$$= 4740 \text{ cm}^2$$

Diameter of central manifold $= \sqrt{\dfrac{4740 \times 4}{\pi}}$

$$= 77.7 \text{ cm}$$

Provide a commercially available diameter of 800 mm

Assuming a spacing of 15 cm for laterals,

The number of laterals $= (2 \times 5.85 \times 100)/15 = 78$

Cross sectional area of each lateral $= 2370 / 78 \text{ cm}^2 = 30.39 \text{ cm}^2$

Diameter of lateral $= \sqrt{\dfrac{(30.39 \times 4)}{\pi}} = 6.22\ cm$

Provide laterals of diameter of 80 mm

Number of perforation per lateral = 1242 /78 say 16

Length of lateral = 1/2 (width of filter - dia of manifold)

$= 1/2 (4.5 - 0.8) = 1.85$ m

Spacing of perforations $= 1.85 \times 100 / 16 = 11.56$ cm

Provide 16 perforations of 9 mm dia at centre to centre spacing of 115 mm.

(E) Compute Dimension Of Wash Water Trough

Assume a wash water rate of $36 \text{ m}^3/ \text{ m}^2/\text{hr}$

Washwater discharge for 1 filter $= 36 \times 26.33 \text{ m}^3/\text{hr}$

$= 947.88 \text{ m}^3/\text{hr}$

$= 0.2633 \text{ m}^3/\text{sec}$

Assuming a spacing of 1.6 m for wash water trough which will run parallel to the longer dimension of the filter unit.

No. of troughs $= 4.50 / 1.6 = 3$

Discharge per unit trough $= 0.2633 / 3 = 0.0878 \text{ m}^3/\text{sec}$

For a width of 0.4m, the water depth at upper end is given by

$Q= 1.376\ bh^{3/2}$

$0.0878 = 1.376 \times 0.4 \times h^{3/2}$, $h = 0.294$ say 0.3 m

Assume a free board of 0.1 m, provide a depth of 0.4 m

Provide three trough of 0.4 m wide x 0.4 m deep in each filter

(f) Computation Of Total Depth Of Filter Box

Depth of filter box = sum of depths for (i) underdrains

(ii) gravel (iii) sand (iv) water depth (v) free board

= 0.8 + 0.45 + 0.6 + 1.2 + 0.3 = 3.35 m

(g) Determine Initial Head Loss

The sieve analysis of filter sand is as follows:

Sand size, mm	0.3	0.4	0.5	0.6	0.7	0.8	1.0	1.45
(% of sand smaller than stated size)	0.0	2.0	10.0	27.0	50.0	70.0	90.0	100.0

Porosity of sand bed = 0.4

Sphericity of sand = 1.0

Head loss for a clean filter can be determined using Kozeny's equation for stratified beds

$$\frac{h}{l} = \frac{kv}{g}.\upsilon.\frac{(1-f)^2}{f^3}\left(\frac{6}{\psi}\right)^2 \sum_{i=1}^{n} \frac{p_i}{d_i^2}$$

Where h is the head loss, l the depth of sand bed, k Carman Kozeny constant having a value of 5, v velocity of filtration, ν kinematic viscosity, f porosity of clean bed, ψ grain sphericity, p_i fraction of sand and d_i geometric mean diameter of sand.

Computation of $\sum_{i=1}^{n} \frac{p_i}{d_i^2}$

Size of sand mm	% of sand larger than stated size	Sand fraction within adjacent sieve size p_i x 100	di cm x 100	p_i/d_i^2
0.3	00.00	2	3.5	16
0.4	2.0	8	4.5	40

0.5	10.0	17	5.5	56
0.6	27.0	23	6.5	52
0.7	50.0	20	7.5	36
0.8	70.0	20	8.9	25
1.0	90.0	10	11.8	8
1.4	100.0			
		100		233

$$\frac{h}{l} = \frac{5 \times 500}{981 \times 3600} \times 1.01 \times 10^{-2} \times \frac{(1-0.40)^2}{(0.4)^3}\left[\frac{6}{1.0}\right]^2 \times 233 = 0 / 0.337$$

Head loss = 0.337 x 0.6 = 0.20 m

Head loss for clean filter bed for given sand is 0.20 m.

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APPENDIX 7.8

PREPARATION OF FILTER SAND FROM STOCK SAND

1. Problem Statement

Prepare a filter sand of effective size 0.5 mm and uniformity coefficient 1.5 from the stocksand, the sieve analysis for stock sand being given as follows:

Sand size, mm	0.21	0.30	0.42	0.84	1.12	1.68	2.38
Cumulative weight, %	3.5	11	22	42	64	83	90

2. Solution

The given size distribution of stock sand is plotted on log - normal probability paper and from the plot determine the percentages of sand having size less than the effective size of 0.5 mm (p_1) and having size less than the 60 percentile size (p_2)

$p_1 = 24\ \%$

$p_2 = 43\ \%$

(i) Hence percentage of usable sand $= 2 \times (P_2 - P_1)$

$= 2 \times (43-24) = 38\ \%$

(ii) Percentage of stock sand below

Which stock sand is too fine, $P_4 = 1.2\ P_1 - 0.2\ P_2$

$p_4 = 1.2 \times 24 - 0.2 \times 43$

$p_4 = 20.2\ \%$

Determine the size of this sand, d_4 from graph

For $P_4 = 20.2\ \%$, $d_4 = 0.41$ mm

(iii) Percentage of stock sand above $= 1.8\ p_2 - 0.8\ p_1$

which stock sand is too coarse

$P_5 = 1.8 \times 43 - 0.8 \times 24$

$P_5 = 58.2\ \%$

Determine the size of this sand, d_5 from the graph,

For $P_5 = 58.2\ \%$, $d_5 = 1.0$ mm

It follows that all-stock sand finer than 0.41 mm size and coarser than 1.0 size should be removed to obtain the filter sand of effective size 0.5 mm and uniformity coefficient of 1.5.

APPENDIX 7.9

INFORMATION TO BE INCLUDED IN THE TENDER SPECIFICATIONS FOR WATER TREATMENT PLANT

General

The principal requirement must be a spacious and convenient layout. The structures should represent a pleasing appearance with aesthetic features forming a balance between function and form. The interiors of the structures shall be eye appealing and in keeping with the objectives of the plant viz., production of pure and wholesome water.

While the mode of design and construction could be a matter of individual choice, it should be ensured that all materials, design, construction and fabrication details for different units including doors and windows conform to the IS Specifications and codes of practice wherever available and in their absence, to the established standards.

Adequate provision shall be made in the Civil Engineering works for laboratory, office buildings, administration area, sanitary facilities and water supply etc. The area requirement of these ancillary requirements shall be stipulated. Roadways with adequate lighting shall be provided. Adequate ladders or steps and handrails where required shall be provided for easy access to each unit of the treatment plant and wherever necessary, walkways should be provided. Interconnecting facilities shall be provided to enable the operator to move freely for maintenance and operation of the plant.

All water retaining structures shall be designed in conformity with IS 3370 while the other structures shall be designed according to IS 456.

The tender specifications should include *inter alia,* process requirements and specifications for equipment.

A. Process Requirements

1. The following data shall be furnished to the tenderers:

(a) Raw water analysis comprising of monthly average figures preferably for a full year period covering various seasonal variations in respect of, at least the following. If the full year data is not available, the worst seasonal values may be given :

(i) pH

(ii) Turbidity

(iii) Total Alkalinity

(iv) Total hardness

(v) Chlorides

(vi) Coliform organism (MPN)

(b) Any other additional data, if the water is known to contain constituents or contaminants which are required to be removed:

(i) Phenols

(ii) Tastes and odours

(iii) Colour

(iv) Carbon dioxide

(v) Algal content

(vi) Iron

(vii) Manganese

(viii) Hardness (Carbonate and noncarbonate alongwith magnesium content of water)

(ix) Fluoride content, and

(x) Chlorine demand.

and any other pollutants arising from industrial effluents and agricultural runoff.

(c) Hydraulic data such as the relevant raw water inlet and filtrate outlet levels.

2. The following requirements shall be furnished:

(a) The flow requirements of the plant in terms of the nett output expected of the plant for a given period of time, say 23.5 hours a day (allowing for washing of the filters, etc. and also overload capacity.)

(b) The quality of the treated water in terms of pH, turbidity, coliform organisms (MPN) and E. *coli:* and where needed Iron, Manganese, Hardness (carbonate and noncarbonate along with Magnesium content of water), fluoride content and colour.

(c) Design parameters for various treatment units such as chemical dosing, rapid mixing, slow mixing, sedimentation, filtration and chlorination as well as special processes like aeration, microstraining, iron and manganese removal, fluoride removal, taste and odour control as per specific local requirements and in accordance with the details furnished in the Manual.

(d) A suggested layout of a Water Treatment Plant including following details, to the extent possible:

(i) Unit sizes and location of plant structures;

(ii) Schematic flow diagram showing flow through various units;

(iii) Piping arrangement including bypasses showing the material and size of pipes as well as direction of flow;

(iv) Hydraulic profile of the units showing the flow of water.

(v) Contour map of the area including provision for future expansion.

(vi) Approach roads and water supply facilities for construction purposes .

(vii) Other information about site such as proneness to flooding and earthquakes, groundwater table fluctuations, type and nature of soils met upto maximum anticipated depths, soil characteristics like bearing capacity and corrosivity, intensity and duration of rainfall and total annual rainfall, locations of areas for disposal of excavated spoils and of borrow pits if required for filling purposes.

e) The contract should establish where guarantees apply and clearly define their requirements performance guarentees must be demonstrated by a test run of specified length or over an agreed period of operation .

B. Mechanical Equipment

1. The following data may be given while inviting tenders for pumping plant :

   (a) Number of units required to work in parallel.

   (b) Nature of liquid to be pumped :

      (i) Fresh or salt water

      (ii) Temperature of liquid

      (iii) Specific gravity

      (iv) Amount of suspended matter present

   (c) Required capacity as well as minimum and maximum amount of liquid the pump must deliver .

   (d) Suction conditions :

      (i) Suction lift or suction head.

      (ii) Constant or variable suction condition.

   (e) Discharge conditions.

      (i) Maximum/Minimum discharge pressures against which pump has to deliver liquid.

      (ii) Static head description : constant or variable.

      (iii)Friction head description and how estimated.

   (f) Type of service : continous or intermittent.

   (g) Pump installation : horizontal or vertical position.(If vertical type of pit, wet and dry).

   (h) Power available to drive the pump.

   (i) Space, weight or transportation limitations.

   (j) Location of installation .

   (k) Special requirements with respect to pump design : construction or performance.

2. The following requirements may be indicated.

(a) The pump equipment as well as the component shall conform to the relevant I.S . standards and in their absence, to any other accepted international or national standard.

(b) Any special duty conditions such as temperature, humidity, corrosive atmosphere should be specified.

(c) Submerged structure parts except hot rolled sections shall not be less than 6 mm thick under normal atmosphere and 8 mm in aggressive atmospheres.

(d) Prime movers and allied components such as electrical motor, starter switches reduction gear, drive mechanism, bearings, plummerblocks,etc. shall be of approved make.

(e) All rotating machinery particularly gears shall be designed with adequate safety margins and service factors.

(f) An itemwise price list of spare parts shall be furnished by the tenderer.At least two years requirement of fast moving spares should be supplied along with the equipment.

(g) The supplier of special equipments like softeners, recording gauges, rate controllers, chlorinators, proportioning chemical feeders, meters, etc. shall furnish the services of a competent representative for a specified number of days during a specified period to instruct the plant operating personnel in the maintenance and care of the equipment and to conduct tests and make recommendations for producing most efficient results.

(h) Equipment selection with respect to specification, spare units,spare parts and servicing can affect maintanence, operating and investment costs. It is the purchaser's responsibility to incorporate into the contract all requirements and limitation which affect cost. Equipment performance is usually guaranteed by the manufacturer.

The contractor shall furnish bonds converting items of work like mechanical equipments, piping etc. for specified period as a gurantee of satisfactory operation and correction and correction of any defect in the work, material or equipments furnished by them.

On special equipment extended guarantees, maintenance over a period of time and supervision of a complete installation may be provided by the manufacturer. On most large equipments, the manufacturer provides field service with respect to installation.

(i) All water submerged parts, rotating mechanical parts, and steel pipes under water shall be adequately protected after surface preparation. Oil, grease, dirt,soil and all surface contaminants from structural and fabricated steel parts are removed by cleaning with solvent vapour, alkali emulsion, or steam. Loose rust or paint, weld spatter, etc. are removed by hand chipping, scraping, sanding, wire brushing and grinding. The bare finished shafting, finished flanges and other mechanical surfaces are protected by grease line or rust protection measures. Structural mechanism support and super structure, walkway, hand rails, fabricated shafts, etc., shall be protected with at least one coat of primer and two coats of paint. IS: 800 – 1962 gives the code of practise for use of structural steel in general building construction.

APPENDIX 7.10

COMMON CHEMICALS USED IN WATER TREATMENT

Sl. No	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance And Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used For Handling Solution	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12
1.	Activated Carbon	-	C	Taste and odour control, dechlorinati on	Granular	Not less than 80% C	Black granules 1-3 mm	Water passed through granular beds	Dry	-	-
2.	Activated carbon	-	C	Do	Black powder	Do	200 to 400 mμ black powder insoluble	-	Dry or in slurry form careful mixing required to maintain proper slurry	Iron or steel tank	-
3.	Activated silica	Silica sol	SiO_2	Coagulant aid	Produced at site as needed from sodium silicate and activating agents	-	Clear, often opalescent	0.6	Wet batch made up by pH Adjustment and aged	Mild steel or stainless steel or rubber containers	Appliances likely to be clogged with improper pH adjustment of feed

Sl. No	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance And Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used For Handling Solution	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12
4.	Aluminium sulphate	Alum; filter alum; sulphate of alumina	$Al_2(SO_4)_3$ $4H_2O$	Coagulant	Blocks; lump: powder	Atleast 16% Al_2O_3	Light tan to gray ; crystalline acidic corrosive, slightly hygroscopic	8-10 %	Wet or dry	Acid proof brick tanks bitumen coated concrete or rubber lined tanks	-
5.	Aluminium sulphate (ferric)	Alum; filter alum ;sulphate of alumina;alumino ferric	$Al_2(SO_4)_3$ $4H_2O$ (approx.)	Do	Do	15% (approx.) Al_2O_3	Brown to dark brown; crystalline, acidic, corrosive,hygroscopic	8-10 %	Do	Do	High concentration of iron
6.	Alum liquid	Liquid alum	$Al_2(SO_4)_3$	Coagulant	Solution sp.g. 1.1	8% Al_2O_3	Brown solution, acidic, corrosive	Direct or in 1% solution	Wet, orifice box rotameter and proportionating pumps	Acid proof brick, tanks, bitumen coated concrete or rubber lined tanks	Costs less than dry alum if close enough to source of manufacture
7.	Ammonium sulphate	Sulphate of Ammonia	$(NH_4)_2SO_4$	Chloramine treatment in disinfection	Crystal	20-25% NH_3	White sugar sized crystals	0.1 to 0.5 %	Wet Proportionating pumps	stainless or plastic containers	
8.	Anhydrous ammonia	Ammonia	NH_3	Do	Liquefied gas	98-99 % NH_3	Colourless gas; pungent irritating offensive odour	-	Wet ammoniator	Iron, steel or glass	Dangerous chemical

Sl. No	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance And Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used For Handling Solution	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12
9.	Bentonite	Collodial clay	$H_2O(Al_2O_2, Fe_2O_2, MgO) 4SiO_3.nH_2O$	Coagulant aid, floc weighting agent	Powder pellets	-	Yellow brown	-	Wet suspension	Iron or steel	-
10.	Calcium hydroxide	Hydrated lime slaked	$Ca(OH)_2$	pH adjustment softening	Powder	80-90 % $Ca(OH)_2$	White powder; caustic	1.5%	Dry or wet. Can be fed in suspension	Iron, steel or concrete tanks	Not very soluble, lead tank cannot be used
11.	Calcium hypochl-orite	HTH, per chloron	$Ca(OCl)_2.4H_2O$	Disinfectan, taste or odour control	Granular powder	70% available chlorine	white	2 to 4%	wet	Stoneware, plastic, rubber tank	Dangerous chemical
12.	Calcium oxide	Quick lime, burnt, unslaked lime	CaO	pH adjustment and softening	Pebble crushed lumps to powder	40-90 % CaO	White or light gray, caustic	1.5 %	Dry or wet can be fed in suspension. Dry feeders generally discharge to slake before application	Iron, steel or concrete tanks	Lead tank cannot be used
13.	Chlorine	Chlorine gas, liquid chlorine	Cl_2	Disinfection taste and odour control general oxidant	Liquified gas under pressure	99-99.8 % Cl_2	Green yellowish gas, pungent ;corrosive heavier than air, health hazard	-	Wet chlorinator	Dry : iron copper steel solution :silver glass hard rubber, lead special alloys	Dangerous chemical very careful handling required, should use gas mask

Sl. No	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance And Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used For Handling Solution	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12
14.	Chlorinated ferrous sulphate	Chlorinated copperas	$Fe_2(SO_4)_3$ $FeCL_2$	Coagulant	Yellow solution	Produced at site by reaction of chlorine and ferrous sulphate	-	3-5 %	Wet	Rubber lined or stainless steel containers; plastic containers	-
15.	Chlorinated lime	Bleaching powder, chloride of lime	$CaO.$ $2CaOCl_2$ H_2O	Disinfection	Powder	25-33 % available chlorine	White,hygroscopic, unstable pungent powder	1-2 %	wet	Plastic, stoneware or rubber tanks	-
16.	Chlorine dioxide	-	ClO_2	Taste and odour control disinfection	Gas	26.3 % available chlorine	Generated at site	0.1 %	wet	Plastic, soft rubber	-
17.	Copper sulphate	Blue vitriol	$CuSO_4$ $5H_2O$	Algicide	Crystal lumps, powder	90-95 %	Clear blue crystals	1-2 %	Dry put in bags dragged behind boat, dusted on surface with special equipments.	Stainless steel, plastics	-

Sl. No	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance And Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used For Handling Solution	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12
18.	Ferric chloride	Chloride of iron	$FeCl_2$ $.6H_2O$	Coagulant	Lump sticks crystals	60 % $FeCl_3$	Yellow brown highly hygroscopic, very corrosive	3-5 %	Wet, proportion-ating pump	Rubber lined tank or stoneware containers plastic	-
19.	Ferric chloride (solution)	Do	$FeCl_2$	do	Solution	30-40 % $FeCl_3$	Brown solution, very corrosive	3-5 %	Wet proportion-ating pump	do	-
20.	Ferric sulphate	Iron sulphate, ferrifloc	$Fe_2(SO_4)_3$. H_2O	Coagulant	Granules crystals lumps	18.5-0.1 % Fe	Red brown or grey crystals mildly hygroscopic	3-5 %	wet	Dry; iron stainless steel and concrete wet : lead or stainless steel plastics	Solution is corrosive
21.	Ferrous sulphate	Copperas, green vitriol;sugar sulphate	$FeSO_4$ $7H_2O$	do	do	20 % Fe	Green to brownish yellow crystals hygroscopic	4-8 %	Wet	do	Cakes in storage above 20 °C lime addition necessary
22.	Hydrazine hydrate	-	$H_2H_4H_2O$	Deoxygena-tion	Solid powder	64 %	White powder	-	Wet, proportion-ating	Stainless steel pump	plastics
23.	Hydroch-loric acid	Muriatic acid	HCl	in cation exchange	Regenerant	Liquid	30 % HCl liquid	5 – 10%	Wt. Pump	Glass rubber lined	Dangerous chemical

Sl. No	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance And Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used For Handling Solution	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12
				operation						tanks	
24.	Potassium aluminium sulphate	Potash alum	K_2SO_4 $Al_2(SO_4)_3$ $24H_2O$	Coagulant	Lump, powder, blocks	8-11 % Al_2O_3	White crystalline, acidic corrosive hygroscopic	1 %	Wet or dry	-	Generally used in business filters
25.	Sodium aluminate	Soda alum	$Na_2Al_2O_3$	Coagulant	Crystalline flakes,solution	43-45 % Al_2O_2	White to grey crystal, liquid caustic, alkaline	5 %	Wet and dry	Cast iron mild steel concrete	Generally used along with filter alum
26.	Sodium carbonate	Soda ash	Na_2CO_3	pH adjustment softening	Dense crystals; powder	98-99 % Na_2CO_3	White powder, caustic	1-10 %	Dry or wet proportion-ating pumps	-	Dangerous chemical
27.	Sodium chloride	Common salt	NaCl	Softening regenerant	Crystals	90-95 % NaCl	Colourless crystals	8-10 %	Wet, fed through, injection nozzles	Bitumen or epoxy coated M.S. tanks	-
28.	Sodium chromate	Chromate	$NaCrO_4$	Corrosion prevention anodic inhibition	Crystals	80-90 % Na_2CrO_4	Yellow/ brown crystals	-	Wet, proportion-ating pump	Stainless steel, plastic	Dangerous chemical, harmful to eyes. Used in combination with sodium silicate

Sl. No	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance And Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used For Handling Solution	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12
29.	Sodium hexameta phosphate	Calgan, glassy phosphate	$(NaPO_3)_4$	Scale and corrosion prevention	Powder, flakes	60-63 % P_2O_5	Like broken glass	0.25	Wet proportionating pump	Stainless steel, plastic, hard rubber	Holds up ppm of Fe, Mn,Ca and Mg ;for M.S. protective coating
30.	Sodium hydroxide	Caustic soda	NaOH	pH adjustment, softening and filter cleaning	Flakes lumps, pellets, powder	96-99 %	Alkaline, corrosive, hygroscopic	1-10 %	Wet, proportionating pumps, orifices box; rotameter	Cast iron; mild steel; rubber lined	Dangerous chemical
31.	Sodium hydroxi-de(solut-ion)	Caustic soda, iye	NaOH	do	solution	10-40 % NaOH	Syrupy solution	do	do	do	do
32.	Sodium sulphate	-	Na_2SO_4	Deoxygena-tion	Powder lumps	90-99 % Na_2SO_3	White powder	1 %	Wet, proportion-ating pump	Satinless steel plastics	8mg/l sodium sulphate required to remove 1 mg/l O_2
33.	Sulphur dioxide	-	SO_2	Dechlorination or filter cleaning	Gas	100 %	Colourless pungent	-	Dry	Steel	-

Sl. No	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance And Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used For Handling Solution	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12
34.	Sulphuric acid	Vitriol	H_2SO_4	pH adjustment, lowering of alkalinity	Liquid	60-90 % (commercial H_2SO_4)	Syrupy : corrosive; hygroscopic	1-2 %	Wet, dilute solution, orifice box rotameter	Lead, porcelain or rubber	Always add acid to water ; dangerous chemical
35.	Sulphuric acid	do	do	Regenerant cation exchange operation	do	98 % H_2SO_4	do	do	Wet, pumping	Lead, glass or rubber lined tank	do

APPENDIX 9.1

COMPUTATION OF CHEMICAL DOSAGES IN WATER SOFTENING TREATMENT PROCESS USING LIME SODA PROCESS

1. Problem Statement

The water with following chemical constituents is to be softened using Lime Soda Process.Compute the quantities of chemicals required to treat 250 m^3/hr of water flow assuming practical units of removal for $CaCO_3$ to be 30 mg/l and for $Mg(OH)_2$ as 10 mg/l.

CO_2	= 9.9 mg/l	Alkalinity (CO_3)	= 175 mg/l
Ca^{++}	= 100 mg/l	SO_4^{2}	= 107 mg/l
Mg^{++}	= 8.9 mg/l	Cl^-	= 17.8 mg/l
Na^+	= 11.5 mg/l		

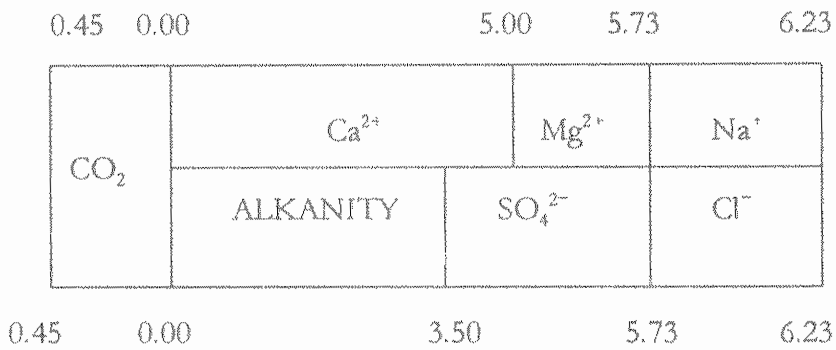
2. Solution

1. Compute me/l of all components present in water.

Sl.No.	Component	Concentration mg/l	Equivalent weight	Conc. me/l.
1	CO_2	9.9	22.0	0.45
2	Ca^{2+}	100.0	20.0	5.00
3	Mg^{2+}	8.9	12.2	0.73
4	Na^+	11.5	23.0	0.50
5	Alkalinity	175.0	50.0	3.50
6	SO_4^{2}	107.0	48.0	2.23
7	Cl	17.8	35.5	0.50

2. Prepare a me/l bar graph of raw water with hypothetical combination.

In preparing such a bar graph , the concentration of cations is usually arranged left to right starting with Ca2+ followed by Mg^{2+}, Na^+ etc. in that order . Similarly, below the cations, anions are arranged left to right commencing with alkalinity followed by SO_4^{2-}, Cl^- etc.The CO2 being a molecule is conventionally shown to the left of zero mark .



Graph

3. Compute the quantities of chemicals required.

(a) Chemical reactions:

CO_2	+	$Ca(OH)_2$	$\rightleftharpoons$	$CaCO_3$	+	H_2O	(1)
$Ca(HCO_3)_2$	+	$Ca(OH)_2$	$\rightleftharpoons$	$2CaCO_3$	+	$2H_2O$	(2)
$MgSO_4$	+	$Ca(OH)_2$	$\rightleftharpoons$	$Mg(OH)_2$	+	$CaSO_4$	(3)
$CaSO_4$	+	Na_2CO_3	$\rightleftharpoons$	$CaCO_3$	+	Na_2SO_4	(4)

It may be noted that it is necessary to add excess lime, usually a surplus of 35 mg/l as CaO or 1.25 me/l above stoichiometric requirement, to raise pH for precipitation of Magnesium by reaction at (3)

No.	Components	Conc. me/l	Lime required me/l	Soda ash Required me/l	Remarks
1	CO_2	0.45	0.45	-	Reaction(1)
2	$Ca(HCO_3)_2$	3.50	3.50	-	(2)
3	$CaSO_4$	1.50	-	1.50	(4)
4	$MgSO_4$	0.73	0.73	0.73	(3) & (4)
			4.68	2.23	

Lime required = Stiochiometric quantities + excess lime

= 4.68 + 1.25 = 5.93 me/l

= 166.04 mg /l as CaO

= 219.41 mg/l as $Ca(OH)_2$

Soda Ash required = 2.23 me/l.

= 118.19 mg/l as Na_2CO_3

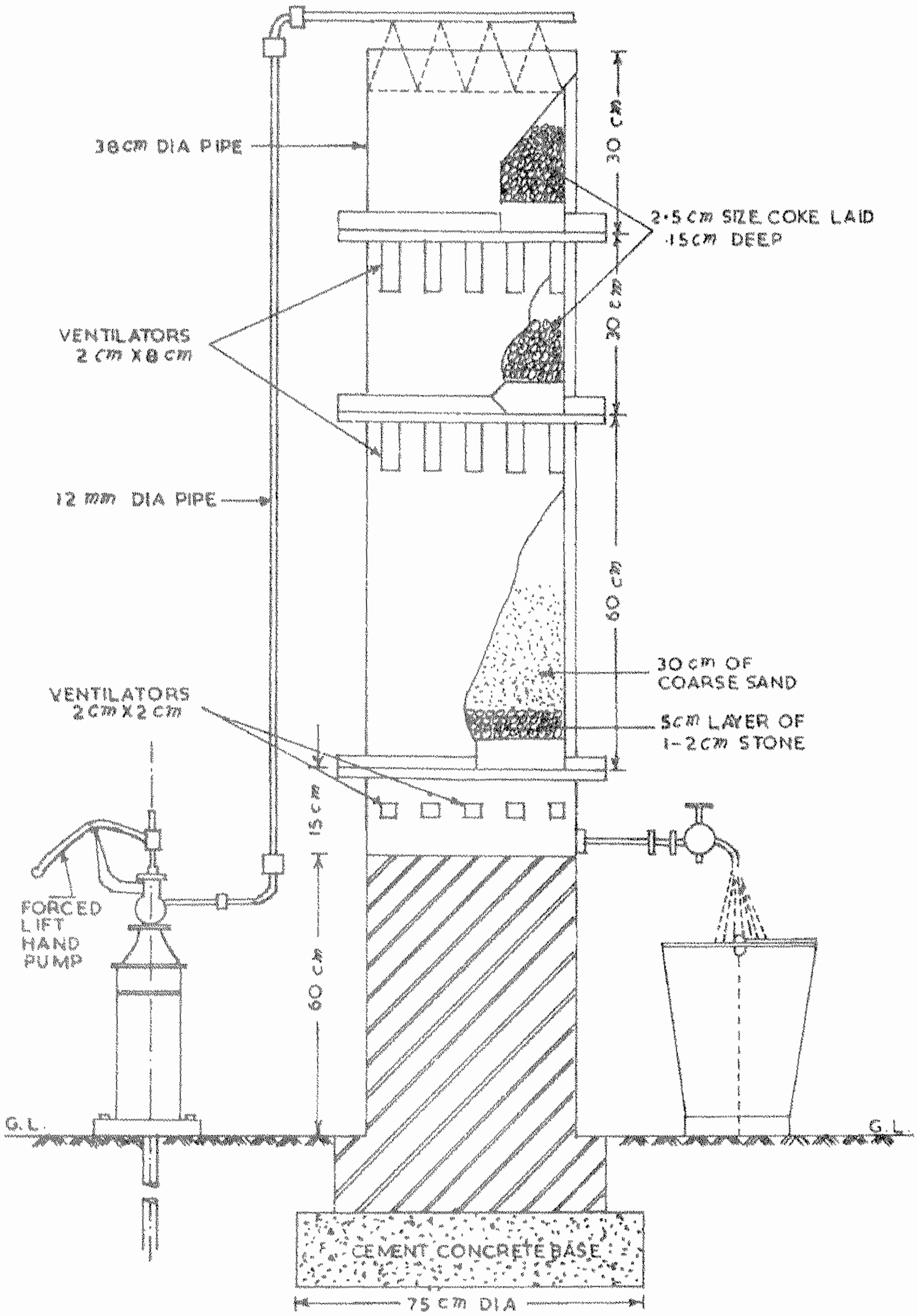
Annual consumption of lime as CaO

$$= (166.04 \times 250 \times 24 \times 10^3 \times 365) / (10^6 \times 10^3)$$
$$= 363.63 \text{ metric tons}$$

Annual consumption of Soda ash as Na_2CO_3

$$= (118.19 \times 250 \times 24 \times 10^3 \times 365) / (10^6 \times 10^3) \text{ metric tons}$$
$$= 258.84 \text{ metric tons}$$

APPENDIX 9.2



TYPE DESIGN OF IRON REMOVAL PLANT

(CAPABLE OF GIVING 200 LITRES PER HOUR)

2962 UA&E/97—44B

APPENDIX 9.3

DESIGN OF IRON REMOVAL UNITS

Typical designs of iron removal units for 5,10 and 20 m^3/h follow.

DESIGN CONSIDERATIONS

- Schemes have been designed for 5, 10 and 20 m^3/h flow and 10 % extra water quantity to provide for sedimentation bleed losses and filter back wash requirements.
- Power shut-downs are frequent and rarely more than two hours supply is available in the morning and evening. Accordingly, raw water pumping hours assumed to be 2 hours in the morning and two hours in the evening. During these four hours pumping period, total daily requirements of water are to be pumped to elevated storage tank to draw water by gravity flow to the treatment unit(s).
- To avoid extra cost for additional overhead tank for filtered water, it is assumed that the filtered water from the sump-well will be directly pumped for the distribution. The distribution of treated water would follow the same time schedule as contemplated for pumping raw water.
- Backwashing of the sand filter would be carried out by using raw water from the overhead tank.

DESIGN CRITERIA

Water consumption		40 lpcd
Tray aerator		
	Spacing of Trays	0.3 m.
	Aeration Rate	1.26 $m^3/m^2/h$
Sedimentation Basin		
	Detention Period	2.5 h
Sand Filter		
	Effective Size	0.6-0.8 mm
	Uniformity Coefficient	1.3 to 1.7
	Sand Depth	1.2 m
	Total head above sand	1.35 m
	Rate of Filtration	4.88 $m^3/m^2/h$
	Minimum Backwash Rate	35 $m^3/m^2/h$
	Total Head for Filter Wash	12 m
	Gravel Depth	0.39-0.62 m

Gravel Size

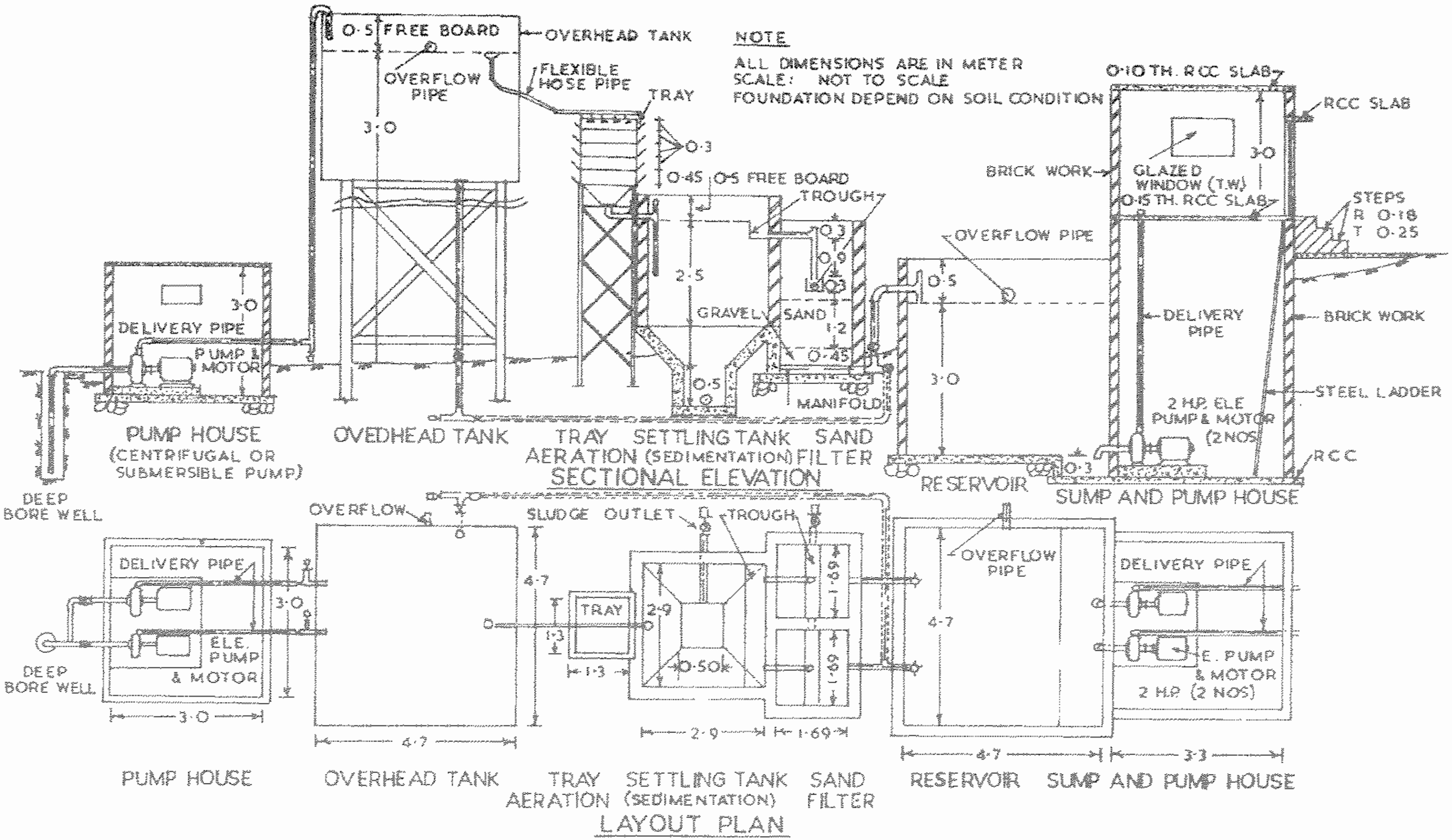
65-38 mm	13-20 cm
38-20 mm	8-13 cm
20-12 mm	8-13 cm
12-5 mm	5- 8 cm
5- 2 mm	5- 8 cm

specifications of units are detailed in the following Table and the arrangements are as shown in Fig. 9.8 to 9.10.

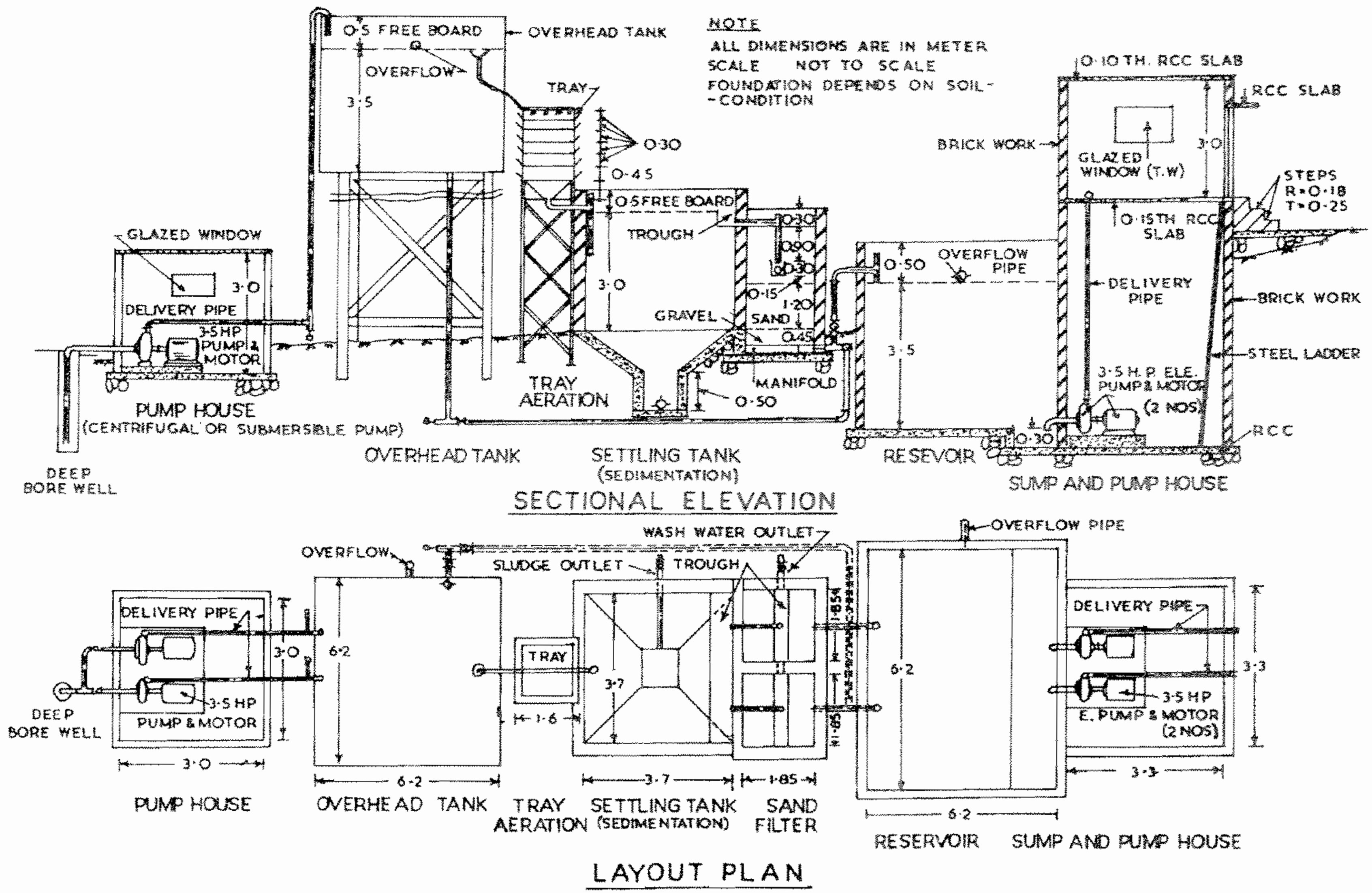
TABLE

DESIGN SPECIFICATIONS FOR CONTINUOUS IRON REMOVAL UNIT FOR COMMUNITY WATER SUPPLY

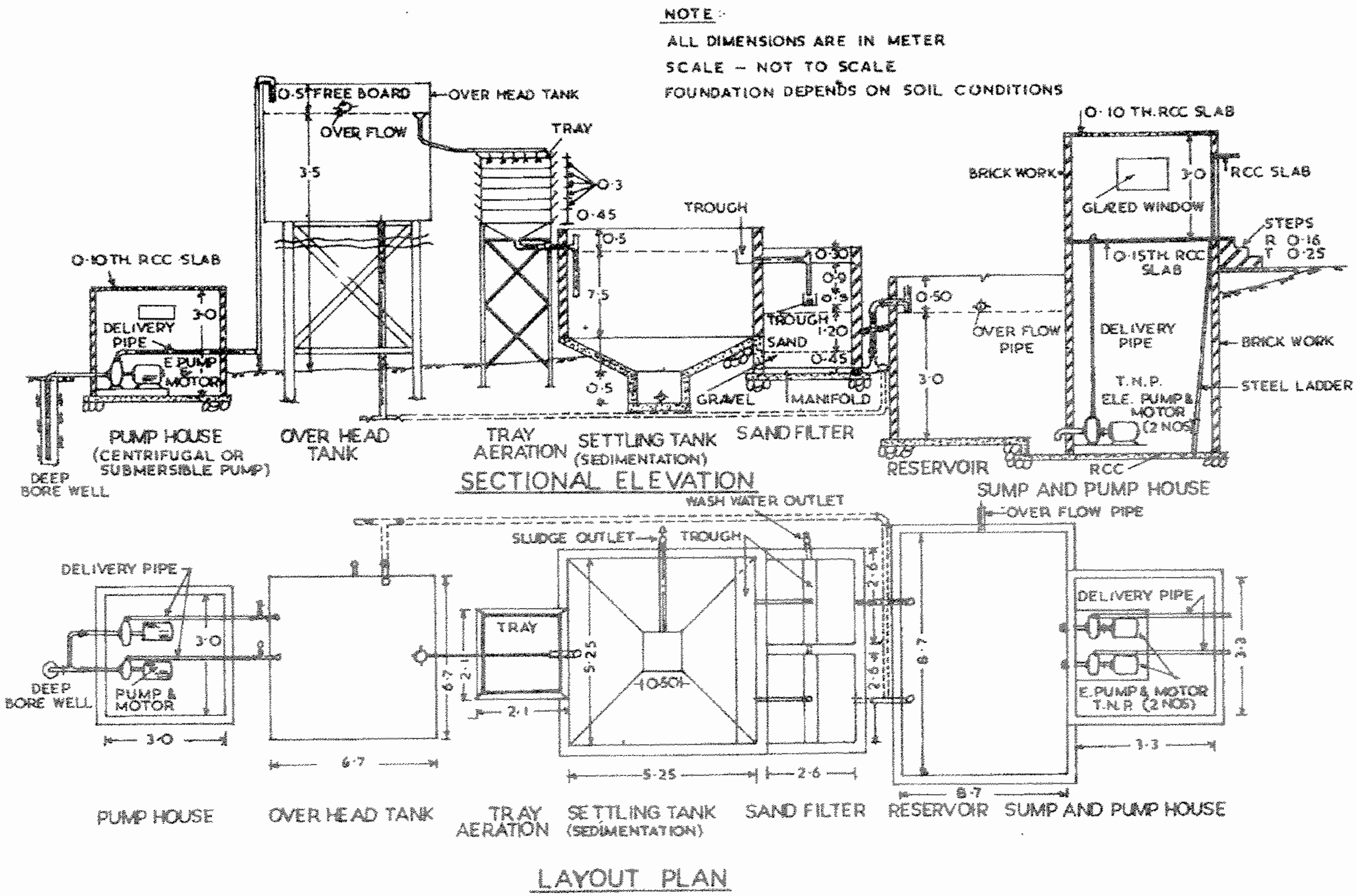
Design capacity, m^3/h	5	10	20
Raw water Pump, hp	2.0, 2 Nos	3.4, 2 Nos	7, 2Nos
Overhead Reservoir			
Capacity, m^3	66	132	264
Size	4.7m x 4.7m x 3m	6.2m x 6.2m x 3.5m	8.7m x 8.7m x 3.5m
Tray Aerators			
No. of Trays	4	5	6
Collection trough	1	1	1
Tray size	1.3 m x 1.3m	1.6 m x 1.6m	2.lm x 2. lm
Height of each tray	.03 m	0.3m	0.3 m
Sedimentation Tank			
Size	2.9m x 2.9 m x 2.5m	3.7m x 3.7m x 3.0m	5.25m x 5.25m x 3.0m
Sand Filter			
Size	1.69 m x 1.69m	1.85m x 1.85 m,	2.6m x 2.6
		2 Nos	2 Nos
Clear Water Storage Tank	4.7m x 4.7m x 3m	6.2m x 6.2m x 3.5m	8.7m x 8.7m x 3.5m



CONTINUOUS IRON REMOVAL FOR COMMUNITY WATER SUPPLY CAPACITY 5 m^3/h



CONTINUOUS IRON REMOVAL FOR COMMUNITY WATER SUPPLY CAPACITY 10 m³/h



CONTINUOUS IRON REMOVAL FOR COMMUNITY WATER SUPPLY CAPACITY 20 m³/h

APPENDIX 9.4

SOLAR RADIATION

North Lat.	range	Probable average values of insolation - Direct and Diffused on a Horizontal surface at sea level at Langley's per day.							
		January		February		March		April	
	0	Visible	Total	Visible	Total	Visible	Total	Visible	Total
34	Max[e]	114	360	160	450	204	553	254	659
	Min[f]	53	155	78	215	118	320	141	385
32	Max	126	380	169	450	212	570	258	663
	Min	63	180	87	240	126	340	146	395
30	Max	136	400	176	490	218	587	261	875
	Min	76	220	96	260	134	362	151	405
28	Max	146	420	184	510	224	603	264	683
	Min	87	250	106	290	142	373	156	415
26	Max	156	440	192	530	230	615	266	690
	Min	99	280	114	310	149	390	160	425
24	Max	166	460	200	545	236	625	268	697
	Min	111	310	123	340	156	410	164	433
22	Max	174	480	206	560	241	644	270	701
	Min	123	355	132	370	162	426	167	440
20	Max	183	500	213	575	246	652	271	703
	Min	134	360	140	390	168	440	170	447
18	Max	192	515	220	590	250	664	272	705
	Min	144	380	150	410	174	459	174	452
16	Max	200	530	226	610	255	670	272	707
	Min	154	400	159	430	180	473	177	456
14	Max	208	555	233	630	258	680	271	709
	Min	163	430	167	450	184	487	179	400
12	Max	216	572	239	645	262	690	271	710
	Min	172	455	176	450	189	500	181	462
10	Max	223	595	244	655	264	694	271	711
	Min	179	475	184	490	193	513	183	464
8	Max	230	610	249	665	267	700	270	709
	Min	187	495	192	510	196	523	185	467

APPENDIX 9.4 (Continued)

SOLAR RADIATION

North Lat.	range	Probable average values of insolation - Direct and Diffused on a Horizontal surface at sea level at Langley's per day.							
		May		June		July		August	
	0	Visible	Total	Visible	Total	Visible	Total	Visible	Total
34	Max[e]	290	743	297	775	289	763	267	696
	Min[f]	176	462	168	439	178	472	159	448
32	Max	290	744	296	772	289	761	269	700
	Min	181	475	166	731	178	472	163	458
30	Max	290	744	296	768	289	759	271	702
	Min	184	490	163	425	178	469	166	462
28	Max	289	743	294	764	288	755	272	704
	Min	187	506	161	418	178	467	169	466
26	Max	288	741	292	760	288	749	273	706
	Min	189	518	158	409	177	463	172	169
24	Max	288	738	290	753	287	742	273	708
	Min	191	525	155	403	176	459	174	471
22	Max	286	734	286	747	285	736	273	707
	Min	193	530	152	392	173	454	176	472
20	Max	284	730	284	738	282	729	272	706
	Min	194	532	148	383	172	450	177	472
18	Max	282	723	280	728	280	723	272	705
	Min	194	530	145	375	170	442	177	471
16	Max	279	718	276	720	277	715	270	702
	Min	194	528	141	363	167	435	177	469
14	Max	276	710	272	710	273	708	265	700
	Min	194	524	137	354	164	429	177	467
12	Max	273	702	267	700	269	700	267	697
	Min	193	518	133	343	161	421	176	464
10	Max	270	694	262	688	265	690	266	693
	Min	192	512	129	330	158	414	176	460
8	Max	266	685	258	678	260	680	263	688
	Min	191	506	124	320	154	405	174	456

APPENDIX 9.4 (Continued)

SOLAR RADIATION

North range Lat.		Probable average values of insolation - Direct and Diffused on a Horizontal surface at sea level at Langley's per day.							
		September		October		November		December	
	0	Visible	Total	Visible	Total	Visible	Total	Visible	Total
34	Max[e]	221	602	178	490	128	380	101	338
	Min[f]	134	368	96	250	70	202	47	158
32	Max	226	615	185	510	138	400	114	360
	Min	140	385	104	270	80	221	60	184
30	Max	321	625	192	524	148	420	126	380
	Min	147	399	113	290	90	256	70	210
28	Max	236	635	119	537	157	440	138	400
	Min	154	415	120	310	99	278	80	236
26	Max	240	652	205	552	166	460	149	420
	Min	160	429	128	332	109	300	90	260
24	Max	244	659	212	568	175	480	101	440
	Min	165	443	136	360	119	326	101	280
22	Max	248	668	218	582	183	500	172	460
	Min	170	455	143	380	128	350	110	300
20	Max	252	674	224	596	190	520	182	480
	Min	176	467	150	400	138	370	120	320
18	Max	256	680	229	605	198	538	192	500
	Min	180	479	157	418	146	390	129	340
16	Max	259	684	234	615	206	554	200	520
	Min	185	489	164	434	154	410	138	300
14	Max	262	688	240	627	214	567	209	530
	Min	189	496	170	449	162	430	146	380
12	Max	264	691	244	640	221	585	217	550
	Min	193	502	176	462	169	446	154	400
10	Max	266	693	248	650	228	600	225	570
	Min	196	510	181	474	176	462	102	420
8	Max	267	695	252	660	234	616	231	590
	Min	200	518	186	468	182	478	169	440

EXPLANATORY NOTE

(a) Calculated from data published by the United States Weather Bureau

(b) Gram Calories per square cm = Langley

(c) "Visible" Radiation of wavelengths of $4000A^0$ to $7000A^0$ penetrating a smooth water surface .

(d) Total Radiation of all wavelengths in solar spectrum .

(e) Value which will not normally be exceeded .

(f) Value based on or extrapolated from lowest values observed for indicated month and latitude during 10 years of record.

Approximate corrections for elevation upto 3000m

Total radiation = Total(1+0.6105El)

Visible radiation = Vis.(1+0.03053El) where El.is in thousands of metres.

Correction for cloudiness (approx.) = Min+[(Max-Min) Cl.]Where Cl is fraction of time weather is clear.

APPENDIX 10.1

CALCULATION OF CAPACITY OF SERVICE RESERVOIR

PROBLEM

Find out capacity of storage reservoir for the following two situations viz.,

(i) Power is not available from 6a.m.to 10a.m. daily

(a) 16 hrs. of pumping during 10p.m. to 6a.m. and 10a.m.to 6p.m..

(b) 8 hrs.of pumping during 4a.m. to 6a.m. and 12 noon to 6p.m.

(ii) Power is available throughout 24 hrs.

(a) 16 hrs.of pumping during 4a.m.to 12 noon and 1p.m.to 9p.m.

(b) 8 hrs. of pumping during 4a.m. to 8p.m. and 2p.m. to 6p.m.

Data given are:

1. Design population-24,000
2. Per Capita water supply-90 lpd
3. Peak factor-2.25
4. Peak hours: 6a.m. to 10a.m.,1p.m.to 2p.m.,5p.m.to 6p.m.
5. Other than peak hours, hourly demands are as follows:
   (i) 20% of average hourly demand:11p.m. to 4a.m..
   (ii) 40% of.average hourly demand: 4 a.m. to 5 a.m. and 10 p.m. to 11 p.m.
   (iii) 60% of average hourly demand: 12 noon to 1 p.m.
   (iv) 70% of average hourly demand: 2p.m. to 5 p.m. and 8 p.m. to 10 p.m.
   (v) 80% of hourly demand: 5 a.m. to 6 a.m.
   (vi) 90% of hourly demand: 6p.m. to 8p.m.
   (vii) 100% of hourly demand: 10a.m. to 12 noon
6. Water supply is continuous.

SOLUTION

1. Total demand = 24000 x 90lpd = 2.16 mld
2. Average hourly demand = 2.16/24 = 0.09ml = a
3. Peak hourly demand = 2.25 x average hourly demand = 2.25a

Tables 1 and 2 show the compilation for arriving at the capacity of the service reservoir, for 16 and 8hours of pumping.

In Table 1 data from cols.1 to 3 are applicable for both the given situations (i) and (ii). Computed data for situation (i) and (ii) are given in cols. 4,5,6,7, those inside the brackets referring to situation (ii)

Similarly in Table 2 computed data outside the brackets from cols. 2 to 5 refer to the situation (i) while those inside the brackets are for the situation (ii).

Storage required under situations:

(i) (a) 0.8460 mL or 39% of daily demand.

(b) 0.9900 mL or 46% of daily demand.

(ii) (a) 0.3285 mL or 15% of daily demand.

(b) 0.7065 mL or 33% of daily demand.

TABLE 1

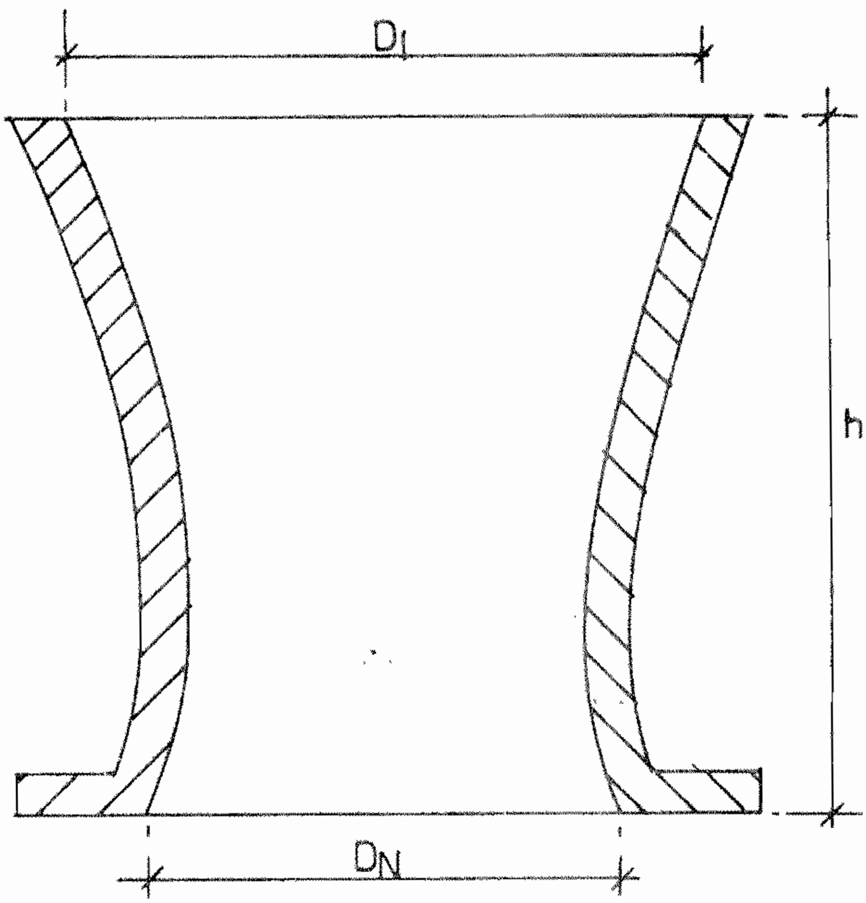
SHOWING COMPUTATION FOR CAPACITY OF SERVICE RESERVOIR

Given data			16 hours pumping; pumping rate: =24xa/6		8hours pumping; pumping rate = 24xa / 8 = 3a	
Period in hours	Hourly demand	Cumulative demand	Cumulative pumping	Cumulative deficit or surplus	Cumulative pumping	Cumulative deficit or surplus
(1)	(2)	(3) = Σ {2}	(4)	(5) = (4) - (3)	(6)	(7) = (6) - (3)
04-05	0.400a	0.40a	1.50a(1.50a)	+1.10a(+1.10a)	3.00a(3.00a)	+2.60a(+2.60a)
05-06	0.80a	1.20a	3.00a(3.00a)	+1.80a(1.80a)	6.00a(6.00a)	+4.80a(+4.80a)
06-10	2.25a	10.20a	3.00a(9.00a)	-7.20a(-1.20a)	6.00a(12.00a)	-4.20a(+1.80a)
10-12	A	12.20a	6.00a(12.00a)	-6.20a(-0.20a)	6.00a(12.00a)	-6.20a(-0.20a)
12-13	0.60a	12.80a	7.50a(12.00a)	-5.30a(-0.08a)	9.00a(12.00a)	-3.80a(-0.80a)
13-14	2.25a	15.05a	9.00a(13.50a)	-0.05a(-1.55a)	12.00a(12.00a)	-3.05a(-3.05a)
14-17	0.70a	17.15a	13.50a(18.00a)	-3.65a(0.85a)	21.00a(21.00a)	-3.85a(+3.85a)
17-18	2.25a	19.40a	15.00a(19.50a)	-4.40a(-0.10a)	24.00a(24.00a)	+4.60a(+4.60a)
18-20	0.90a	21.20a	15.00a(22.50a)	-6.20(+1.30a)	24.00a(24.00a)	+2.80a(+2.80a)
20-21	0.70a	21.90a	15.00a(24.00a)	-6.90a(+2.10a)	24.00a(24.00a)	+2.10a(-2.10a)
21-22	0.70a	22.60a	15.00a(24.00a)	-7.60a(+1.40a)	24.00a(24.00a)	+1.40a(+1.40a)
22-23	0.40a	23.00a	16.50a(24.00a)	-6.50a(+1.00a)	24.00a(24.00a)	+1.00a(+1.00a)
23-01	0.20a	23.40a	19.50a(24.00a)	-3.90a(+0.60a)	24.00a(24.00a)	+0.60a(+0.60a)
01-04	0.20a	24.00a	24.00a(24.00a)	0.00a(0.00a)	24.00a(24.00a)	0.00a(+0.00a)

TABLE 2

SHOWING CAPACITY OF SERVICE RESERVOIR FOR DIFFERENT HOURS OF PUMPING

Pumping hours	Maximum Cumulative surplus	Maximum Cumulative deficit	Capacity of Storage reservoir	Capacity of Storage Reservoir in mL. Substituting Values of a = 0.09 mL
(1)	(2)	(3)	(4) = (2) + (3)	(5)=(4) x 0.09ml
16	1.80a (2.10a)	7.60a (1.55a)	9.40a (3.65a)	0.8460(0.3285)
8	4.80a (4.80a)	6.20a (3.05a)	11.00a (7.85a)	0.9900 (0.7065)



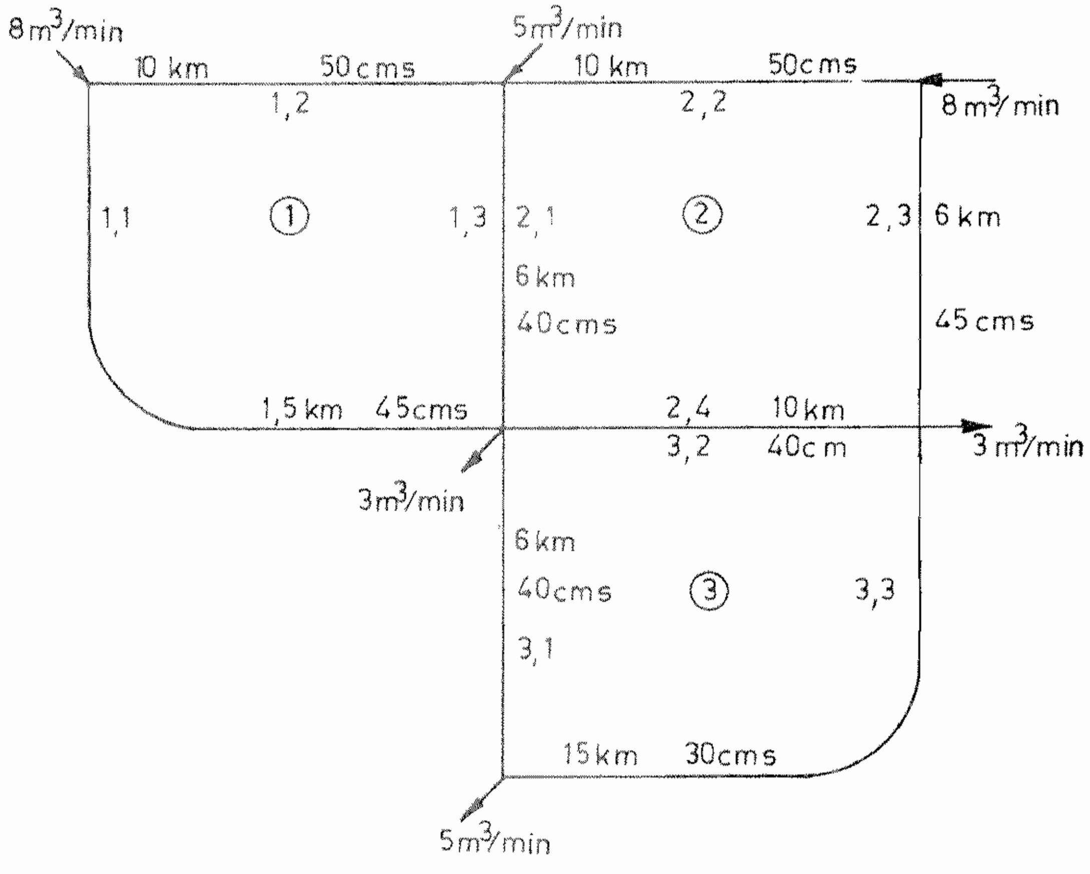
APPENDIX 10.2

NOMINAL DIAMETER D_N (mm)	ENLARGED END DIA D_1 (mm)	HEIGHT OF BELL MOUTH h (mm)	WEIGHT (APPROX.) (kg)
80	125	100	7
100	150	150	9
125	175	150	12
150	200	150	15
200	285	200	23
250	350	200	31
300	450	250	45
350	525	250	58
400	600	300	80
450	650	300	93
500	750	300	120
600	900	410	201
700	1050	470	304
800	1200	520	435
900	1350	590	575
1000	1500	650	792
1100	1650	710	965
1200	1800	770	1243
1500	2250	950	2092
1800	2700	1150	3320

DETAILS OF BELL MOUTH FOR OUTLET CONNECTIONS IN SERVICE RESERVOIRS

APPENDIX 10.3

PROBLEM TO ANALYSE THE LOOPED NETWORK GIVEN BELOW



NOTE ALL "C" VALUES ARE 100(BRITISH)

2962 UA&E/97—45A

SOLUTION TO THE PROBLEM ON HARDY CROSS METHOD OF BALANCING HEAD LOSSES BY CORRECTING ASSUMED FLOWS

Loop i	Pipe (i,j)	Length kms	Dia D cms	C	First iteration Flow m³/min	Slope s ‰	Head H.m	H/Q
1.	1.1	15	45	100	4.0	0.67	10.0	2.50
	1.2	10	50	100	4.0	0.39	3.9	0.98
	1.3*	6	40	100	3.0	0.69	4.1	1.37
							-2.0	4.85
2.	2.1*	6	40	100	-3.0	-0.69	-4.1	1.37
	2.2	10	50	100	-4.0	-0.39	-3.9	0.98
	2.3	6	45	100	4.0	0.67	4.0	100
	2.4†	10	40	100	0	0	0	0
							4.0	3.35
3	3.1	6	40	100	-4.0	-1.15	6.9	1.73
	3.2†	10	40	100	0	0	0	0
	3.3	15	30	100	1.0	0.36	5.4	5.40
							1.5	7.13

*,† indicate common pipes

	First iteration (Contd.) Flow Correction m³/min	Corrected flow m³/min	Second iteration Slope s ‰	Head H.m	H/Q	Flow correction m³/min	Corrected flow m³/min
1.1	+0.02	-3.78	-0.61	9.1	2.41	+0.19	3.59
1.2	+0.22	4.22	0.45	4.5	1.07	+0.19	4.41
1.3	+0.22 – 0.65	2.57	0.50	3.0	1.17	+0.19 – 0.01	2.75
	(-2.0)/(1.85 x 4.85)=0.22			1.6	4.65	(-1.6)/(1.85 x 4.65) = 0.19	

2.1*	+0.65 – 0.22	-2.57	-0.50	-3.0	1.17	+0.01 – 0.19	-2.75
2.2	+0.65	-3.35	-0.28	-2.8	0.84	+0.01	-3.34
2.3	+0.65	4.65	0.90	5.4	1.16	+0.01	4.66
2.4	+0.65 – 0.11	0.54	0.03	0.3	0.56	+0.01 – 0.04	0.51
(-(-4.0)/1.85 x 3.35) = 0.65				-0.1	3.73	-(-0.1)/(1.85 x 3.73) = 0.01	
3.1	+0.11	-3.89	-1.10	-6.6	1.70	+0.04	-3.85
3.2+	+0.11 – 0.65	-0.54	-0.03	0.3	0.56	+0.04 – 0.01	-0.51
3.3	+0.11	1.11	0.42	6.3	5.68	+0.04	1.15
-(-1.5)/(1.85 x 7.13) = 0.11				0.6	7.94	-(-0.6)/(1.85 x 7.94) = 3.44	

Pipe (i,j)	Third iteration		At Balance	
	Slope s, ‰	Head m	Flow m³/min	Headlosses m
1.1	-0.56	-8.4	-3.59	-8.4
1.2	0.49	4.9	4.41	4.9
1.3*	0.56	3.4	2.75	3.4
		-0.1∂		
2.1*	-0.56	-3.4	-2.75	3.4
2.2	-0.27	-2.7	-3.34	-2.7
2.3	0.91	5.5	4.66	5.5
2.4+	0.03	0.3	0.51	0.3
		-0.3∂		
3.1	-1.10	-6.6	-3.85	-6.6
3.2+	-0.03	-0.3	-0.51	0.3
3.3	0.45	6.8	1.15	6.8
		-0.1∂		

∂ Absolute values of all unbalanced headlosses are less than or equal to 0.3 m, the tolerance limit set

APPENDIX 11.1

DESIGN CALCULATIONS FOR A PUMPING PLANT

DATA OF THE SCHEME

1.1	Daily demand of water	116 mld
1.2	Hours of pumping, considering loss of one hour due to tripping and other minor interruptions	23 hrs per day
1.3	Water levels in the sump, by RLs	
	1.3.1 Maximum (High flood level)	11.0 m
	1.3.2 Mean	9 m
	1.3.3 Minimum	7 m
1.4	Rising main	
	1.4.1 Length	2575 m
	1.4.2 Diameter	1.2m
	1.4.3 Friction coefficient for m.s., mortar lined pipeline	110 m
1.5	RL of point of discharge	59.0 m
1.6	No. of pumps	
	1.6.1 Duty pumps	4
	1.6.2 Stand bye pumps	2
1.7	RL of ground level at the pumping station	8.25 m
1.8	RL of high food level	10.5 m
1.9	Altitude of the site above HSL	1250m
1.10	Ambient temperature	40° C
2.0	Size of pipes and fittings for the pumping system	
2.1	Inlet bell mouth	
	Design velocity	1.5 m/s
	Bell mouth diameter	0.545 m Say 550 mm
2.2	Column pipes	
	Design velocity	2.5 to 3 m/s Say, 2.75 m/s
	Column pipe diameter	0.402 m Say 400 mm

2.3	Delivery pipes and valves	
	Design velocity	2.5 m/s
	Diameter of delivery pipe ,delivery valve & NRV	0.422 m
		Say 450 mm
2.4	Bell mouth at discharging point	
	Design velocity	0.8 m/s
	Bell –mouth diameter	1.49 m
		Say 1500mm
3.0	Hydraulic calculations	
3.1	Combined discharge of 4 pumps	
In parallel (116 mld x 24 hrs)/23 hrs		121.01 mld
3.2	Rate of total flow with 23 hrs running of pumps per day	1.4 cubic m/s
3.3	Discharge of each pump	0.35 m^3/s
3.4	Mean static head (59m – 9m)	50 m
3.5	Frictional loss in straight pipe of rising main for combined discharge .	3.495 m
3.6	Frictional losses in bends , valves & in rising main @ 10% of (3.5)	0.3495m
3.7	Frictional loss in taper, delivery valve,NRV & individual delivery pipe of Nb 450 mm	0.35 m
3.8	Velocity head lost at atmosphere at the exit , as $v^2/2g$,where v = 0.5 m/s	0.013m
3.9	Design head = (3.4)+(3.5)+(3.6)+(3.7)+(3.8)	54.207m
3.10	System Resistance Curves	

System resistance curves are prepared by calculating total head of flows and based on following level conditions in sump at minimum, mean and maximum WL's. The head losses as they work out in example are as tabulated :-

a. Combined Q m^3/hr.	0.25	0.50	0.75	1.00	1.25	1.50
b. Max. Static Head			52 m			
c. Mean static head			50 m			
d. Min static head			48 m			

e. Friction in rising main	0.144	0.519	1.101	1.876	2.836	3.976
f. Friction in valves and fittings @ 10% of (e)	0.014	0.052	0.110	0.187	0.284	0.397
g. Velocity	0.001	0.004	0.009	0.016	0.025	0.036
h. Total friction (e)+(f)+(g)	0.159	0.575	1.220	2.079	3.145	4.109
i. Total head based on						
Min WH	52.16	52.57	53.22	54.08	55.14	56.41
Head WL	50.16	50.57	51.22	52.08	53.14	54.41
Max WL	48.16	48.57	49.22	50.08	51.14	52.41

Note:- Station losses in individual delivery pipe taper and valves should not be added for system resistance curve but should not be deducted from pumps H-Q curve. When the losses are very small , they may be neglected .

Thus , design duties and Head variations shall be as under :-

(i) Discharge $Q = 0.35\ m^3/s$ i.e. 350 l/s

(ii) Duty head $H = 54.207$ m

≈ 54.25 m

(ii) Head range :Pump should be suitable for operation in all variations from solo operation to 4 pumps in parallel and level variation in sump from min. WL to max. WL.

4.0 Selection Of Type ,Number Of Stages And Running Speed

Pump Head as per 3 (h), H= 54.25 m

Head loss at enterance to bell-mouth

$Hi = 0.05\ V^2/2g = 0.006$ m

Head loss in column pipe , assuming presently ,10 m length of column and as per Figure (3) in IS: 1710-1972

$Hc = 0.45$ m

Head loss in discharge bend/ tee =0.15 m

Hence bowl Head ,H=54.25+0.006+0.45+0.15=54.856 m

In case of horizontal centrifugal pumps, bowl head is not required to be calculated. Hence for these pumps, pump Total Head will be 54.25 m. However, since the difference between the head for horizontal Centrifugal pump and the bowl head for vertical pumps is marginal in this case, the pump head as54.856 m is considered in this exercise

For selecting the suitable pump , the following options in combinations of type of pump, number of stages as running speed are taken into accounts :-

Option	Type	Stages	Suctions	Speed
a.	Vertical turbine or Hor. End-suction	1	1	1480,980
b	Hor. Double suction	1	2	1480,980
c.	Vertical turbine	2	1	1480,980
d.	Vertical turbine	3	1	1480,980
e.	Vertical turbine	4	1	1480,980

Typical calculations for (a) and (c) are as under :

Single stage, single suction, 1480 rpm

$n_s = 3.65.N.Q^{0.5}/H^{0.75} = 3.65 \times 1480 \times 0.35^{0.5}/(54.856)^{0.75} = 158.75$

Attainable efficiency as per Figure 11.1=0.87

Suction head required as per Figure 11.3 = 1.5m @ 30 degree C.

Considering allowance of 0.5 m for field conditions, required suction

head = 2m @ 30 degree C.

Add say 0.2m for frictional losses in suction pipes.

Add 0.3 m for difference in vapour pressures at 30 degree C and site ambient 40 degree C.(ref : Table 11.2)

Add 0.75m for difference in atmospheric pressures at mean sea level and site altitude 1250m (ref : Table 3).

Hence minimum suction head required @ site condition for

a) Centrifugal pump =2+0.2+0.3+0.75=3.25m

b) VT Pump = 2+0.3+0.75=3.05m

Thus eye of impeller of centrifugal pump will have to be located 3.25m below minimum WL. As GL is 8.25m RL and min. W is 7.0m RL , impeller eye will be at 3.75 MRL i.e. 4.5 m below GL , and pump floor at approximately 0m RL. Minimum water depth will equal to min. Suction head , length of bend and bell-mouth and bottom clearance is equal to 3.25 + 1.275 = 4.525m.

In case of VT pump, eye of impeller will be above bottom of pump by distance = bottom Clearance + Length of bellmouth & bowl upto impeller eye

=D/2 + 0.75 = 1.025m above bottom of sump

Thus minimum water depth required upto minimum WL to satisfy NPSHR for VT pump= 1.035 + 3.05 = 4.075

(c) Two stage, single suction, 1480 r.p.m.

H,head per stage=54.856/2=27.428m

$n_q = 3.65 \times N \times Q^{0.5}/H^{0.75} = 3.65 \times 1480 \times 0.35^{0.5}/(27.428)^{0.75}$

=266.65

Attainable efficiency as per figure 11.1=0.87

Suction head required as per figure 11.3 = 0.5m at 30^0C

Working out as for (a) above for field condition allowance, head loss in suction appurtenances, difference in vapour pressures at 30^0C and site ambient and difference in vapour atmospheric pressure at mean sea level and site altitude, suction head required at site condition =2.05m

Location of eye of impeller below minimum WL and minimum water depth required to satisfy NPSHR can be worked out as for (a) above .

The final value are tabulated in the table attached .

Observations : Possible feasible choices considering excavation cost etc. are

(a) Double suction horizontal centrifugal pump with depth of excavation of 3.0m but added construction cost of pump house (and land)which is required to be located at site of pump.

(b) 2/3 stage VT pump with depth of excavation of 4.325m but reduced construction cost of pump house which will be located above sump.

(c) Difference between efficiency of pumps a & b is very insignificant.

From observations and remarks it is seen that final choice is limited to either double suction horizontal centrifugal pump with pump house at site but with some risk of flood as HFL is at RL 10.50 m,CL 8.25 m and pump house floor will be at RL 8.5 m (approx).

2 or 3 stage VT pump with pump house above sump but with 1.25 m extra excavation .

Cost of two alternative will be almost at par .Considering flood risk , alternative with VT pump is selected .In order to keep operating floor free from obstruction and pipe work, delivery is taken below floor level .The pump shall be self water lubricated .

5. Sump Dimensions

(a) Clearance between bottom of sump and lip of suction bellmouth,

C=D/3=550/3=185.3 mm Say 185 mm

(b) Distance between rear well and center of bell mouth,

B=3D/4=3/4x550=412.5=400 mm

(c) Spacing between pumps

Desirable spacing between pumps is 2.5 D i.e. 1375 mm. However, size of lower flange of headgear /discharge head (accommodating stuffing box, thrust bearing and flexible coupling) would be approximately 3.5 times column pipe diameter i.e. 1400 mm. Keeping about 600 mm clearance,spacing will be 2000 mm

(d) Slope

As seen minimum depth of water required is 3.075m below minimum WL.In order to minimize excavation cost, permissible slope of 14 degree is taken .The slope will terminate upstream of pump at a distance equal to 3 D i.e. 1650 mm from pump center.

(e)Straight Approach

The portion under the pump will be flat from line of termination of slope upto atleast rear false wall.

(f) Rear False Wall

Size of base of discharge head will be 1400 mm. i. e. 700 mm from center of pump, whereas dimension B is 400 mm (max). Therefore, column and rear wall of sump will have to be located at least 1000 mm away from pump center keeping 300 mm margin for nut fastening, etc. Therefore, rear false wall is necessary at a distance of 400 mm(clear) from pump center.Top or false wall will be upto maximum water level.

(g) Baffles/ Dividing Walls

Dividing walls will be constructed between pumps to above mutual interference. Both ends of each dividing wall shall be rounded . Front edge of dividing wall shall be in line with front edge of suction bellmouth. At rear end opening 150-200 mm size shall be kept at least upto minimum WL. Top of dividing wall will be upto maximum WL.

5. Sizes Of Important Components/Equipment

(a) As calculated in 2 above

Column Pipe	400 mm
Inlet bellmouth	550 mm

(b) Line shaft diameter using empirical formula

$$KW = \frac{fNd^3}{5.01 \times 10^8}$$

Where f = Safe stress in Kgf/cm^2

= 400 kgf/cm^2 for EN 8/c – 40 shaft

d = 55.68 mm

Adding corrosion allowance of 3.4 mm

Minimum line shaft diameter = 59 mm

(c) Thickness of column pipe

The column pipe will act as a closed pressure vessel when pump is started under shut off condition. Considering specific speed and pattern of pump characteristics, shut off head is likely to be 80 m.

Hence design pressure(@ 1.5 times shut off pressure)

$P = (80 / 10) \times 1.5 = 12$ kgf/cm^2

For pressure vessel as per IS 2825-1969

$t = (PD_i)/(200f.j - p)$

where p, design pressure =12 kgf /cm^2

D_i, internal diameter =400mm

f, safe stress =10kgf /mm^2

j, welding factor =0.7

Hence t = 3.45mm

Adding 4mm corrosion allowance as pipe is subject to corrosion from both inside and outside.

Thickness of column pipe = 7.5mm = 8mm

(d)Motor

Lowest bowl efficiency as per 4 above is 0.87

Allowing 3% margin, quoted bowl efficiency is 0.84

Input to bowl assembly (clause 3.17 of IS 1710)

$= (54.856 \times 350 \times 60)/ 6120 \times 0.84 = 224$kw

Power loss in thrust bearing and line shaft bearing	3 KW
Input to pump	227 KW
Considering 10% margin of power in motor , rating of motor required	249.7 KW
i.e.	250 KW

Note: Calculations for motor rating is to be done to enable detailing specifications for associated electrical equipment .Motor rating should not be specified in the specifications.

As motors are to be installed indoors, SPDP motors with IP 23 protection shall be suitable. As rating is 250 kW, as seen from article E. 2.3 either 415 v or 3.3 kV can be adopted. However, as maintenance problems are less in 3.3 kV installation, 3.3 kV motors are selected.

(e) Transformer

Total load of 4 pump motor sets 250x4 = 1000 KW

Hence transformer KVA required at 0.85 P.F. and 10% margin.

= (1000 x 1.1)/0.85 = 1294 KVA

Hence provide next commercial rating 1600 KVA

(f) Motor Control Gear

As motors are of 3.3 KV, either MDCB or vacuum contactors can be selected.

(i) Current at 0.85 P.F. and lowest voltage 3.3 KV-10% i.e. 2.97 KV

$$= \frac{250}{0.85 \times \sqrt{2.97}} = 57.17A$$

As minimum available rating is 100/200 A, a 100 A breaker shall be specified.

(ii) Short Circuit current rating

Normal impedence for 1600 KVA transformer, Z=6%

Minimum impedence with 10% tolerance in impedence as per IS 2026,

Z_{min} = 5.4%

Therefore short circuit MVA

= (1600 x 100)/(5.4 x 1000) = 29.62 MVA

As motor contributes 10 times its normal full load current during fault, contribution of 4 motors.

S.C. current = 57.17 x 10 x 4 = 2.28 KA

S.C. MVA = 11.76 MVA

Hence total S.C. MVA = 41.38 MVA

say , 50 MVA

Hence breaking capacity of breaker

= 41.38/ { 2.97 x (3)½} = 8.04 KA

(g) Incoming Breaker to HT Panel

Normal current = 57.17 x 4 = 228.68 A

say, 400 A

S.C. MVA = 41.38 MVA, say, 50 MVA, as before.

(h) Breaker On Incoming to Transformer

Say power supply authorities supply system is 22 nv and characteristic is of 500 MVA.

Therefore S.C. current $\frac{500}{\sqrt{3} \times (22 - 2.2)} = 14.57KA$

The breaker shall be suitable for 500 MVA at 22 KV.

Table Showing The Various Alternatives

Sl. No.	Type	No.	Suction stages	Speed N	n_q	n	Suction Head/Lift	Min Water Depth	Depth of Excavation below GL for sump	RL of location of impeller eye	Remarks
1	Centrifugal	1	Single	1480	158.55	0.87	+3.25	4.525	5.775	3.75	Excavation very deep
2	VT	1	Single	1480	158.55	0.87	+3.05	4.075	5.325	-	-do-
3	Centrifugal	1	Double	1438	158.55	0.88	-2.25	1.750	3.0	9.250	Min required vortex free operation
4	VT	2	Single	1480	266.65	0.87	+2.05	3.075	4.325	-	Excavation deeper than case 2.
5	VT	3	Single	1480	361.44	0.87	+1.75	2.775	4.025	-	-do-
6	VT	4	Single	1480	440.48	0.85	+7.0	8.025	9.275	-	Excavation abnormal

Note : 980 r.p.m. is not considered further as sump with 1480 r.p.m. are suitable@

\+ indicates suction head required

\- indicates suction lift permissible.

APPENDIX 13.1

RECOMMENDED MINIMUM OPERATION AND MAINTENANCE STAFF PATTERN SURFACE SOURCE: TYPICAL STAFF PATTERN (UPTO 5 MLD SYSTEM) WITH CONVENTIONAL TREATMENTS

System component as per flow line		1	2	3	4	5	6	7
		Pump house	Raw water rising main	Treatment works and clear water pump	Clear water rising main	Service reservoir	Gravity main	Distribution system
Sl. No.	Category of staff							
1	Superintendent Manager (A.E.E)	-	-	-	-	-	-	-
2	Supervisor/ Asstt Manager(A.E)	-	-	1	-	-	-	-
3	Assistant Supervisor/Junior Manager.	-	-	-	-	-	-	-
4	Operators	4	-	3	-	-	-	-

System component as per flow line		1	2	3	4	5	6	7
		Pump house	Raw water rising main	Treatment works and clear water pump	Clear water rising main	Service reservoir	Gravity main	Distribution system
5	Helpers/Fitters	2	1* (for every 8Km.)	2	1* (for every 8 Km.)	-	-	Fitter –1 Helper –2 (for every 10-15 Km.)
6	Electrician/ Mechanic	-	-	2	-	-		
7	Watchman	1	-	3	-	1	1	-

Note : 1 The above staffing pattern does not include personnel for billing, collection and accounting for water charges.

2. Above staffing pattern includes the operating staff required for one off-day in a week for staff. Suitable adjustments may have to be made between personnel in pump House and Treatment works.

3. *In case the total length of the pipe line has been less than 8 Km. Under 2 and 4 one Helper/Fitter would be adequate.

APPENDIX 13.2

RECOMMENDED MINIMUM OPERATION AND MAINTENANCE STAFF PATTERN SURFACE SOURCE: TYPICAL STAFF PATTERN (FOR 5 TO 25 MLD SYSTEM) WITH CONVENTIONAL TREATMENTS

System component as per flow line		1	2	3	4	5	6	7
		Pump house	Raw water rising main	Treatment works and clear water pump	Clear water rising main	Service reservoir	Gravity main	Distribution system
Sl. No.	Category of staff							
1	Superintendent Manager (A.E.E)	-	-	-	-	-	-	-
2	Supervisor/ Asstt Manager(A.E)	-	-	1	.		.	.
3	Assistant Supervisor/Junior Manager.	-	-	-	-	-	-	-
4	Operators	3	-	4	-	-	.	.

System component as per flow line		1	2	3	4	5	6	7
		Pump house	Raw water rising main	Treatment works and clear water pump	Clear water rising main	Service reservoir	Gravity main	Distribution system
5	Helpers/Fitters	4	1* (for every 8Km.)	3-1	1* (for every 8 Km.)	-	-	Fitter –1 Helper –2 (for every 10-15 Km.)
6	Electrician/ Mechanic	-	-	2	-	-	-	-
7	Watchman	1	-	3	-	1	1	-

Note : 1. The above staffing pattern does not include personnel for billing, collection and accounting for water charges.

2. Above staffing pattern includes the operating staff required for one off-day in a week for staff. Suitable adjustments may have to be made between personnel in pump House and Treatment works.

3. *In case the total length of the pipe line has been less than 8 Km. Under 2 and 4 one Helper/Fitter would be adequate.

APPENDIX 13.3

RECOMMENDED MINIMUM OPERATION AND MAINTENANCE STAFF PATTERN SURFACE SOURCE: TYPICAL STAFF PATTERN (FOR 25 TO 50 MLD SYSTEM) WITH CONVENTIONAL TREATMENTS

System component	1	2	3	4	5	6	7
as per flow line	Pump house	Raw water rising main	Treatment works and clear water pump	Clear water rising main	Service reservoir	Gravity main	Distribution system
Sl. No. Category of staff							
1 Superintendent Manager (A.E.E)	-	-	1	-	-	-	-
2 Supervisor/ Asstt Manager(A.E)	-	-	-	-	-	-	-
3 Assistant Supervisor/Junior Manager.	-	-	1	-	-	-	-
4 Operators	7	-	7	-	-	-	-

	System component as per flow line	1 Pump house	2 Raw water rising main	3 Treatment works and clear water pump	4 Clear water rising main	5 Service reservoir	6 Gravity main	7 Distribution system
5	Helpers/ Fitters	3	1* (for every 8Km.)	3+1(Lab.)	1* (for every 8 Km.)	-	-	Fitter –1 Helper –2 (for every 10-15 Km.)
6	Electrician/ Mechanic	-	-	3 Electrician -1 Mechanic - 2	-	-	-	-
7	Watchman	1	-	3	1	1	-	-

Note : 1. The above staffing pattern does not include personnel for billing, collection and accounting for water charges.

2. Above staffing pattern includes the operating staff required for one off-day in a week for staff. Suitable adjustments may have to be made between personnel in pump House and Treatment works.

3. The personnel for Sl.1 & 2 should preferably be one from the Civil Engg. and other from the electrical & mechanical Engg. disciplines.

4. *In case the total length of the pipe line has been less than 8 Km. Under 2 and 4 one Helper/Fitter would be adequate.

APPENDIX 13.4

RECOMMENDED MINIMUM OPERATION AND MAINTENANCE STAFF PATTERN SURFACE SOURCE: TYPICAL STAFF PATTERN (FOR 50 TO 75 MLD SYSTEM) WITH CONVENTIONAL TREATMENTS

System component as per flow line		1	2	3	4	5	6	7
		Pump house	Raw water rising main	Treatment works and clear water pump	Clear water rising main	Service reservoir	Gravity main	Distribution system
Sl. No.	Category of staff							
1	Superintendent Manager (A.E.E)	-	-	1	-	-	-	-
2	Supervisor/ Asstt Manager(A.E)	-	-	1	-	-	-	-
3	Assistant Supervisor/Junior Manager.	-	-	1	-	-	-	-
4	Operators	7	-	7	-	-	-	-

	System component as per flow line	1 Pump house	2 Raw water rising main	3 Treatment works and clear water pump	4 Clear water rising main	5 Service reservoir	6 Gravity main	7 Distribution system
5	Helpers/ Fitters	6	1* (for every 8Km.)	6+2(Lab.)	1* (for every 8 Km.)	-		Fitter –1 Helper –2 (for every 10-15 Km.)
6	Electrician/ Mechanic	-	-	3 Electrician –1 Mechanic –2		-	-	-
7	Watchman	1	-	3	1	1	-	-

Note : 1. The above staffing pattern does not include personnel for billing, collection and accounting for water charges.

2. Above staffing pattern includes the operating staff required for one off-day in a week for staff. Suitable adjustments may have to be made between personnel in pump House and Treatment works.

3. From among three categories of personnel indicated at S1.1, 2, & 3 at least one should be from the Electrical & Mechanical Engg. disciplines.

4. *In case the total length of the pipe line has been less than 8 Km. Under 2 and 4 one Helper/Fitter would be adequate.

APPENDIX 13.5

RECOMMENDED MINIMUM OPERATION AND MAINTENANCE STAFF PATTERN ABOVE 75 MLD UPTO 150 MLD

System component as per flow line		1	2	3	4	5	6	7	8
		Intake works	Raw water Pump House	Raw water rising main or Gravity main	Treatment works and clear water pump House	Clear water rising main	Clear water reservoir	Gravity main	Distribution system
Sl. No.	Category of staff								
1	Superintendent Manager Dy. Exe. Engr.	-	-	-	1	-	-	-	-
2	Supervisor/ Asstt Manager(A.E.E)	1	-	1	4	1	1	1	-
3	Assistant Supervisor/Junior Manager.	-	-	-	4	-	-	-	-
4	Operators	-	4	-	12+4	-	-	-	-

System component as per flow line		1 Intake works	2 Raw water Pump House	3 Raw water rising main or Gravity main	4 Treatment works and clear water pump House	5 Clear water rising main	6 Clear water reservoir	7 Gravity main	8 Distribution system
5	Helpers/Fitters	-	4	2	16 (for every 6 Km)	2 (for every 6 kms)	4	2 (for every 6 kms)	Fitter –1 Helper –2 (for every 10-16 Km.)
6	Electrician	-	1	-	4	-	-	-	-
7	Mechanic	-	1	-	1	-	-	-	-
8	Electrical Helper	-	4	-	4	-	1	-	-
9	Watchman	-	4	-	4	-	4	-	-

Note : 1. The above staffing pattern does not include personnel for billing, collection and accounting for water charges.

2. Above staffing pattern includes the operating staff required for one off-day in a week for staff. Suitable adjustments may have to be made between personnel in pump House and Treatment works.

3. From among three categories of personnel indicated at Sl 1, 2, & 3 at least one should be from the Electrical & Mechanical Engg. disciplines.

4. In case the total length of the pipe line has been less than 6 Km. Under 5 one Helper/Fitter would be adequate.

5. Separate staff may be provided for sub-stations(on the pattern of respective Electricity Boards) if there are owned and maintained by the waterworks authority.

APPENDIX 13.6

RECOMMENDED MINIMUM STAFFING PATTERN FOR OPERATION AND MAINTENANCE

SOURCE : BATTERY OF BOREWELLS/TUBEWELLS, OPENWELLS

(EACH WELL YIELDS 5000 GPH MAXIMUM)

System component as per flow line		1		2	3	4	5	6
		Water Works		Pump House	Rising main	Service reservoir	Gravity main	Distribution system
		Less than 10 wells	10 wells & above					
Sl. No.	Category of staff							
1	Supervisor	-	1	-	-	-	-	-
2	Asst. Supervisor	1	1 for every additional 10 wells	-	-	-		-
3	Operators	-	-	1 for every 5 wells/shift	-	-	-	-

System component as per flow line		1 Water Works		2 Pump House	3 Rising main	4 Service reservoir	5 Gravity main	6 Distribution system
		Less than 10 wells	10 wells & above					
4	Helpers Fitters	-	-	1(for every 5 wells/shift)	1* (for every 8 Km.)	-	1* (for every 8 Km.)	3 nos. Fitter –1 Helper –2 (for every 10-15 Km.)
5	Electrician/ Mechanic	-	-	1	-	-	-	-
6	Chowkidar/ Watchman	-	-	3	-	1	-	-
7	Chemist	1	1	-	-	-	-	-
8	Lab. Assistant	1	1	-	-	-	-	-

Note : 1. The above staffing pattern does not include personnel for billing, collection and accounting for water charges.

2. Suitable additional operating staff to be included for one off-day/week for staff.

3. *In case the total length of the pipe line is less than 8 Km. Under 3 and 5 one helper/Fitter would be adequate.

APPENDIX 13.7

RECOMMENDED MINIMUM STAFFING PATTERN FOR OPERATION AND MAINTENANCE

SOURCE : LARGE DIA, HIGH YIELDING TUBEWELL

System component as per flow line		1 Water Works: Less than 5 wells	1 Water Works: 5 wells & above	2 Pump House	3 Rising main	4 Service reservoir	5 Gravity main	6 Distribution system
Sl. No.	Category of staff							
1	Supervisor	-	1	-	-	-	-	-
2	Asst. Supervisor	1	1 for every additional 5 wells	-	-	-	-	-
3	Operators	-	-	1 for every 5 wells/shift	-	-	-	-

System component as per flow line		1 Water Works: Less than 5 wells	1 Water Works: 5 wells & above	2 Pump House	3 Rising main	4 Service reservoir	5 Gravity main	6 Distribution system
Sl. No.	Category of staff							
4	Helpers Fitters	-	-	1(for every 5 wells/shift)	1* (for every 8 Km.)	-	1* (for every 8 Km.)	3 nos. Fitter –1 Helper –2 (for every 10-15 Km.)
5	Electrician/ Mechanic	-	1(for every 5 wells)	-	-	-	-	-
6	Chowkidar/ Watchman	-	-	1(for each well)	-	1(for each well)	-	-
7	Chemist	1	1	-	-	-	-	-
8	Lab. Assistant	1	1	-	-	-	-	-

Note : 1. The above staffing pattern does not include personnel for billing, collection and accounting for water charges.

2. Suitable additional operating staff to be included for one off-day/week for staff.

3. *In case the total length of the pipe line is less than 8 Km. Under 3 and 5 one helper/Fitter would be adequate.

APPENDIX 13.8

SCHEDULE OF PREVENTIVE MAINTENANCE CLARIFLOCCULATORS & THEIR DRIVE

Sl. No.	Name of section or part to be attended	Maintenance to be carried out	Frequency/time interval at which inspection & maintenance to be done	Remarks
1	Trolley Wheels	Lubrication(greasing)	One Month	
2	Reduction Gear Box	Checking & topping of oil level	Three Months	
3	Turn Table Mechanism	Checking & topping the oil level	Three Months	
4	Verticalslip Ring Motor	Dust blowing checking of carbon brushes bearings etc.	Four Months	
5	Rail/Track	Adjustment of gap between two rails & its aligning etc.	Four Months	
6	Reduction Gear Box	Checking of helical or spurgears condition	Six Months	
7	Rubber Type Wheels Iron Wheels	Checking of wear & tear alignment & its positioning	Six Months	More frequently in the old installations
8	M./S. Scrapers	Tightening of nuts & bolts, replacement of broken parts	Year	
9	Turn Table Mechanism	Checking of its sprockets chains, steel balls, gear boxes etc.	Two year	

APPENDIX 14.1

SUGGESTED STAFFING PATTERN FOR SUPERVISORY ENGINEERING DIVISION (WORKLOAD RS. 200 LAKHS ANNUALLY 1988) AND SUBDIVISION (WORKLOAD RS. 50 LAKHS ANNUALLY 1988) FOR O. & M. OF WATERWORKS

Sl. No.	Category of Staff	Division Office	Sub-Division Office
A) Engineering			
1	Ex. Engineer	1	-
2	Dy. Engineer (Civil)	-	1
3	Dy. Engineer (Elec. Mech.)	1*	-
4	Junior Engineer (Civil)		
	a) Diploma holders	2	3
	b) Degree holders	2	2
5	Junior Engineer (Elec. Mech.)	2*	1*
6	Draughtsman	1	-
7	Tracer	2	1
B) Correspondence & Estt. Section			
8	Head Clerk	1	-
9	Senior Clerk	4	1
10	Junior clerk/Typist	4	2
C) Account Section			
11	Senior Accountant	1	-
12	Junior Accountant	4	1
13	Store-keeper	1	-
14	Assistant Store-keeper	-	1
D) Class IV			
15	Peons	6	3
16	Chowkidars	As required	As required
		32[c]	16[c]

* Preferably with degree in Elec. & Mech. Engineering

C Excluding posts of chowkidars

APPENDIX 14.2

REQUIREMENT OF STAFF FOR - O & M

1.	Operation & Maintenance	Recommended Staffing pattern for operation & maintenance of water works for various capacities is given in Appendix 13.1 to 13.7 in the chapter on Operation & Maintenance of waterworks.
2.	Billing & Collecting Water charges	
	a) Meter Reader b) Bill Clerk	One for every 500 connections to be read monthly or a minimum of one if less than Bill 500 connections (includes leave reserve/shift duty also)
	c) Water Rate collectors	One for every 1500 monthly billed connections
	d) Water rate superintendent	One for every 6000 billed connections monthly.
	e) Meter repairer	One for every 80 meters per month to be repaired.
	f) Assistant meter repairer	-do-
3.	Laboratory Personnel	Recommended laboratory personnel is suggested in Chapter 15.

APPENDIX 15.1

MINIMUM STAFF RECOMMENDED FOR WATER WORKS LABORATORIES

		Greater than 7.5 mld	upto 7.5 mld
(i)	Water Analyst (Chemist)	1	-
(ii)	Water Analyst (Bacteriologist)	1	-
(iii)	Water analyst	-	1
(iv)	Laboratory Technician	3	1
(v)	Typist-cum clerk	1	-
(vi)	Sample takers	3	1
(vii)	Laboratory cleaners	3	2

APPENDIX 15.2

PARTICULARS TO BE SUPPLIED WITH THE SAMPLES

1. Name and address of person requesting the examination.
2. Date and time of collection and despatch.
3. Purpose of examination.
4. Source of water and its location (well, tubewell, stream, river etc.).
5. Exact place and depth below surface from which sample was taken.
6. Weather at the time of collection and particulars of recent rainfall, if any.
7. Does the water become affected in taste or odour after rainfall or under any particular circumstances.
8. Are there any complaints from the consumer? If so, the nature of the complaint.
9. Character of surroundings, and proximity to drains, cess pools, cattlesheds, manure heaps, grave yard, bathing ghats and other sources of pollution.
10. Methods of purification and disinfection if any, details, dose of chemicals and points of applications.
11. If from a dug well or a bore well.
    (a) Whether an old source or newly constructed.
    (b) Whether open or covered: nature and material of cover.
    (c) Nature of steining or casing and depth to which constructed and whether it is in good condition
    (d) Height and condition of parapet and apron.
    (e) Method of pumping or other means of raising water.
    (f) Depth of well and of water surface from ground level.
    (g) Whether the water is clear as it flows out of tubewell and remains clear if exposed to air (4-6 hours) or becomes discoloured and turbid.
12. If from a river or stream.
    (a) Nature of flow and weather floods are common or rare.
    (b) Whether level of water is above or below normal.
    (c) Is there any bathing ghat, boat jetty, burial ground or sewer outfall If upstream, give distance from sampling point.

13. If from lakes, impounded reservoirs and tanks.
    (a) How supplied (channel, stream, rain).
    (b) Nature of catchment, whether conserved or not.
    (c) Nature of extent of weed growth.
14. Size and number of service reservoirs.
    (a) Whether open or covered.
    (b) How often cleaned and method of cleaning.
    (c) Date of last cleaning.
15. Number of hydrants and sewers on the distribution system.
16. Hours of pumping and supply.
17. Population served.
18. Any other particulars.

Station *Signature and name in block letters of the person Collecting and forwarding the samples.*

Date

APPENDIX 15.3

SPECIMEN FORM FOR SHORT PHYSICAL AND CHEMICAL EXAMINATION

Name and Address of the Laboratory:

Name and Address of Sender	Sender's No.	Date of Collection................................
		Date and time of receipt at laboratory ....
	Laboratory Ref. No.	Date and time of commencing of examination ....

1. Raw water
2. Coagulated water
3. Filtered water
4. Water after specific treatment
5. Distribution system.

Time of collection of sample			1	2	3	4	5
	Physical	Expressed as					
1	Temperature	^{0}C					
2	Turbidity	JTU/NTU					
3	Colour	Units of Pt-Co- scale					
4	Taste & odour	Qualitative					
Chemical							
5	PH						
6	Conductivity	Micromhos/cm					

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Time of collection of sample			1	2	3	4	5
7	Free CO_2	(mg/l) CO_2					
8	Alkalinity	(mg/l) $CaCO_3$					
	a) Phenolphthalein						
	b) Total						
9	Chlorides	(mg/l) Cl					
10	Nitrites	(Qualitative)					
11	Dissolved Oxygen	(mg/l) O_2					
12	Hardness	(mg/l) $CaCO_3$					
	Carbonate						
	Non-Carbonate						
	Total						
13	Iron	(mg/l) Fe					
14	Fluorides	(mg/l) F					
15	Residual Chlorine	(mg/l) Cl_2					
16	Alumina in Alum	(%) Al_2O_3					
17	Available chlorine in Bleaching Powder	(%) Cl_2					
18	Coagulant Dose-Jar Test	(mg/l)					
19	Chlorine Demand	(mg/l) Cl_2					

Remarks:

Date: *Officer-in-charge*

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APPENDIX 15.4

SPECIMEN FORM FOR COMPLETE PHYSICAL, CHEMICAL AND BIOLOGICAL EXAMINATION

Name and Address
of the Laboratory:

Name and Address of Sender	Sender's No.	Date of Collection................................
		Date and time of receipt at laboratory ....
	Laboratory Ref. No.	Date and time of commencing of examination ....

1. Raw water
2. Coagulated water
3. Filtered water
4. Water after specific treatment
5. Distribution system.

Time of collection of sample			1	2	3	4	5
	Physical	Expressed as					
1	Temperature	^0C					
2	Turbidity	JTU/NTU					
3	Colour	Units of Pt-Co- scale					
4	Taste & odour	Qualitative					
Chemical							
5	pH						
6	Conductivity	Micromhos/cm					
7	Free CO_2	(mg/l) CO_2					

Time of collection of sample			1	2	3	4	5
8	Alkalinity	(mg/l) $CaCO_3$					
	c) Phenolphthalein						
	d) Total						
9	Chlorides	(mg/l) Cl					
10	Amonia	(mg/l)/N					
	a) Free and Saline						
	b) Albuminoid						
11	Nitrites	(mg/l) N					
12	Nitrates	(mg/l) N					
13	Dissolved oxygen	(mg/l) O_2					
14	Oxygen absorbed at 27^0C	(mg/l) O_2					
	a) 3 minutes						
	b) 4 hours						
15	C.O.D	(mg/l) O_2					
16	B.O.D	(mg/l) O_2					
17	Hardness	(mg/l) $CaCO_3$					
	a) Carbonate						
	b) Non-Carbonate						
	c) Total						
18	Iron	(mg/l) Fe					
19	Manganese	(mg/l) Mn					
20	Fluorides	(mg/l) F					
21	Calcium	(mg/l) Ca					
22	Magnesium	(mg/l) Mg					
23	Residual chloride	(mg/l) Cl_2					
24	Sulphates	(mg/l) SO_4					
25	Total solids						
	a) Dissolved						
	b) Suspended						
	c) Volatile						

Time of collection of sample			1	2	3	4	5
26	Alumina in Alum	(%) Al_2O_3					
27	Available chlorine in bleaching powder	(%) Cl_2					
28	Coagulant Dose – Jar test	(mg/l)					
29	Langelier Index	(mg/l)					
30	Chlorine demand	(mg/l)Cl_2					
31	Total silica	(mg/l) SiO_2					
32	Phenolic compounds	(mg/l) Phenol					
33	Synthetic detergents	(mg/l) MBAS					
34	Sulphide	(mg/l) S					
35	Arsenic	(mg/l) As					
36	Cadmium	(mg/l) Cd					
37	Hexavalent Chromium	(mg/l) Cr					
38	Copper	(mg/l) Cu					
39	Cyanide	(mg/l) CN					
40	Lead	(mg/l) Pb					
41	Selenium	(mg/l) Se					
42	Zinc	(mg/l) Zn					
43	Mercury	(mg/l) Hg					
44	Oil and grease	(mg/l)					
45	Polynuclear Aromatic Hydrocarbon	(mg/l) PAH					
46	Radio activity	(pci/l)					
	a) Gross alpha Activity						
	b) Gross Beta Activity						

Time of collection of sample			1	2	3	4	5
BIOLOGICAL							
47	Total count of planktone	(Total count of SAU Organisms/ml)					

Remarks:

Date: *Officer-in-charge*

APPENDIX 15.5
SPECIMEN FORM FOR SHORT BACTERIOLOGICAL EXAMINATION OF WATER

Name and Address
of the Laboratory:

Name and Address of Sender	Sender's No.	Date of Collection..................................
		Date and time of receipt at laboratory ....
	Laboratory Ref. No.	Date and time of commencing of examination ....

1. Raw water
2. Filtered water
3. Chlorinated Water
4. Distribution system.

Time of collection of sample			1	2	3	4
	Bacteriological	*Expressed as*				
1	Plate count	Colonies/ml				
	a) 20^0 C					
	b) 35^0 C					
2	Coliform Organisms	MPN/100ml				

Remarks:

Date: *Officer-in charge*

In addition, common glassware and accessories like beaker, conical flask, burette, pipette, volumetric flask etc. will be required.

APPENDIX 15.8

EQUIPMENT NEEDED FOR BACTERIOLOGICAL EXAMINATION

1. Hot Air Oven — Upto 200^0C
2. Autoclave Or Pressure Cooker
3. Incubator 37^0C or 44^0C (Water/Air-Jacketed)
4. pH Meter
5. Pipette Box (Stainless Steel)
6. Wooden Racks/Aluminium Racks
7. Wire Baskets
8. Cotton/Aluminium Foils
9. Brown paper
10. Twine
11. Burners (Bunsen) With Pilot Lamp
12. Suction Flask (1 Litre Cap)
13. Suction Pump
14. Sampling Bottles (Reagent Bottles Of 250 ml. Capacity)

BACTERIOLOGICAL MEDIA

1. M. Endo Broth (dehydrated)
2. Lactose or Lauryl Tryptose broth
3. Mac Conkey broth
4. Brilliant Green Bile Lactose Broth
5. Total Plate Count Agar
6. Peptone/Triyptone Water

APPENDIX 15.9

TEST TO BE DONE BY WATER WORKS LABORATORIES

Sl. No.	Name of Test	Category of water works laboratory		
		I	II	III
1	Turbidity	√	√	√
2	Colour	√	√	√
3	Odour	√	√	√
4	Conductivity	√	√	√
5	Alkalinity	√	√	√
6	Residual Chlorine	√	√	√
7	pH	√	√	√
8	Iron	√	√	√
9	Chloride	√	√	√
10	Hardness	√	√	X
11	Total solids	√	X	X
12	*Volatile solids	√	X	X
13	*Suspended solids	√	X	X
14	*Free and saline ammonia	√	X	X
15	Albuminiod nitrogen	√	X	X
16	Nitrites(qualitative)	√	√	√
17	Nitrates	√	X	X
18	*Fluorides	√	√	√
19	Metals other than iron	√	√	√
20	Jar test for determining alum dose	√	√	X
21	Chlorine demand	√	√	√
*22	Complete mineral analysis	√	X	X
23	Total count in nutrient agar	√	X	X
24	Presumptive coliforms	√	√	√
25	Confirmed test , BGB	√	√	√
*26	Completed test	√	X	X
*27	Research into media, etc	√	X	X
28	Microscopy	√	X	X

* where applicable

APPENDIX 17.1

AVERAGE INCREMENT COST PER 1000 LITERS

Year	Quantity Of water Sold	Discounted Value (at 8.5%)	Capital Cost	O & M Cost	Total Cost	Discounte d Value (at 8.5%)
	(....kld.....)			(.Rs.million)		
1985-86	-	-	1.0050	-	1.0050	0.9063
86-87	-	-	0.6770	-	0.6770	0.5761
87-88	396	310	-	0.0679	0.0679	0.6532
88-89	411	297	-	0.0683	0.0683	0.0493
89-90	423	231	-	0.0674	0.0674	0.0448
90-91	435	267	-	0.0677	0.0677	0.0415
91-92	470	266	-	0.0686	0.0686	0.4386
92-93	474	247	-	0.0687	0.0687	0.0353
93-94	479	230	-	0.0689	0.0689	0.0331
94-95	484	214	-	0.0690	0.0690	0.0365
95-96	491	200	-	0.0692	0.0692	0.0232
1996-97	2485	784	0.1669(A)	0.3475	0.5144	0.1723
to 2000-01	(497 x 5)			(0.0695 x 5)		
2001-02	2780	594	-	0.3560	0.3560	0.0751
to 2005-06	(556 x 5)			(0.0712 x 5)		
2006-07	3080	438	-	0.3655	0.3655	0.0519
to 2010-11	(616 x 5)			(0.0731 x 5)		
2011-12	656	72	-	0.0741	0.0741	0.0962
2013-14	656	67	-	0.0741	0.0741	0.0076
2013-14	656	67	-	0.0741	0.0741	0.0070
2014-15	25230	762	2.7520(B)	2.3940	4.5660	0.3122
to 2043-44	(841 x 30)			(0.0798 x 30)		
Total		5111				2.6919

*excludes price contingencies and land cost.
(A) Cost of replacement of pumpsets and chlorinator in the year 1996-97
(B) Project cost at 31[st] year
Average Incremental Cost per 1000 liters = Rs. 24,91,911 / (5111 x 365)= Rs. 1.34

APPENDIX 17.2

NET PRESENT WORTH AND BENEFIT COST RATIO OF THE PROJECT AT DISCOUNT RATE 8.5% AND INTERNAL RATE OF RETURN

Sl No	Year	Capital cost * + O & M Cost	Discounted Value		Revenue	Discounted Value	
			at 8.5%	at 2%		at 8.5%	at 2%
		(.......................Rs. million.........................)					
1	1985-86	0.9490	0.8747	0.9304	-	-	-
2	86 -87	0.6370	0.5411	0.6123	-	-	-
3	87-88	0.0679	0.0532	0.0640	0.1176	0.0921	0.1100
4.	88-89	0.0683	0.0493	0.0631	0.1220	0.0880	0.1127
5	89-90	0.0674	0.0448	0.0610	0.1259	0.0837	0.1140
6.	90-91	0.0677	0.0415	0.0601	0.1289	0.0790.	0.1145
7.	91-92	0.0686	0.0388	0.0597	0.1409	0.0796	0.1227
8.	92-93	0.0687	0.0358	0.0586	0.1418	0.0738	0.1210
9.	93-94	0.0689	0.0331	0.0577	0.1430	0.0686	0.1197
10.	94-95	0.0690	0.0305	0.0566	0.1444	0.0639	0.1185
11.	95-96	0.0692	0.0282	0.0557	0.1467	0.0590	0.1180
12.	1996-97	0.4893	0.1629	0.3753	0.7421	0.2340	0.5627
16	to 2000-01	(0.0695*5+0.1418(A)					
17.	2001-02	0.3560	0.0761	0.2445	0.8316	0.1777	0.5711
21.	to 2005-06	(0.0712*5)					
22.	2006-07	0.3655	0.0519	0.2273	0.9255	0.1315	0.5757
26.	to 2010-11	(0.0731*5)					
27.	2011-12	0.0741	0.0082	0.0434	0.1949	0.0215	0.1142
28.	2012-13	0.0741	0.0076	0.0426	0.1950	0.0199	0.1120
29.	2013-14	0.0741	0.0070	0.0417	0.1951	0.0183	0.1099
	Total	-	2.0847	3.0540	-	1.2914	3.0975

* excludes price `contingencies , land cost and taxes and duties .

(A)Cost of replacement of pumpsets and chlorinator in the year 1996-97

Benefit Cost Ratio = 0.62

Net Present worth (Rs. million) = (-) 0.7933

Internal rate of Return (%) = (2 x (8.5 - 2.0) x 0.0435)/ (0.0435 + 0.7933)
= 2.34%

APPENDIX 17.3

ASSUMPTIONS FOR FINANCIAL FORECASTS

A. INCOME AND EXPENDITURE STATEMENT (APPENDIX 17.4)

1. The number of anticipated house service connections after 1987-88 have been estimated on the basis of new houses constructed in the area during the last three years.
2. Fees for service connections are not taken into account.
3. Revenue includes water tax of 25% of house tax. Income under sanitation is also included in the Revenue.
4. The house tax is based on the annual rental value of the houses which are normally revised every five years under the present administrative procedure. In respect of this town, the last revision was made in the year 1975-76 .An increase of 25% over the previous year's demand has been assumed once in five years for the future period beginning from the revision made in 1983-84. It is also assumed that there will be 1% increase in the house tax due to new houses every year and that 25% house tax will be made over to the water supply account as water tax. The demand of house tax for the last year available viz., 1981-82 was Rs. 21,497.
5. The tariff for water is assumed to be Rs. 12.60 per month per connection for domestic consumption from the year 1987-88 and Rs. 1.90 and Rs. 2.85 per 1,000 liters for Commercial & Industrial purposes respectively .The domestic tariff is also assumed to be increased by 10% once in 5 years and commercial and industrial tariffs are assumed to increase by 10% once in three years.
6. Water supplied through public fountains are charged at 25% of the domestic water tariff.
7. The revenue collected by the local body under "Sanitation" is assumed to increase by 1% per annum on the basis of new houses expected to be constructed every year.
8. Operation and maintenance expenditure also includes establishment charges and other expenses, if any, under Sanitation.
9. Establishment charges relating to water supply and sanitation are assumed to increase by 3% annually.
10. Costs of power and chemicals are calculated in proportion to the volume of water produced. The tariff for power is assumed to increase by 10% once in three years. The cost of chemicals is assumed to increase by 7.5% annually in 1984-85 and by 7% annually in 1985-86 and by 6% annually from 1986-87 onwards.
11. Repairs and renewal charges are calculated as 0.5% of total project cost and assumed to be increased by 6% annually.

12. Administrative charges are calculated as 2% of the total establishment charges, power, chemicals and repairs and renewals for the first two years.

13. Depreciation is calculated as 2.5% of the total project cost (including interest).

14. Other charges are calculated as 1% of the total establishment charges, power, chemicals, and repairs and renewals.

(B) SOURCES AND APPLICATION OF FUND STATEMENTS (APPENDIX 17.6)

1. Increase in account payable is the difference of the amount in the two consecutive years as shown in the projected balance sheet.

2. The loan period is assumed to be 24 years inclusive of moratorium period of 4 years during which interest is to be paid but capital repayment is deferred.

3. It is assumed that the government will have to pay 75% of total sub project cost as grant.

4. Compound interest at 8.5% per annum during the moratorium period viz., 1985-86, 1986-87,1987-88 and 1988-89 is calculated for the loan and added to it. The interest thus calculated every year is added to fixed assets and shown in balance sheet.

5. The first loan is received during the year 1985-86 and 6 months' interest is calculated for this year.

6. Repayment of annuity at 8.5% (Rs. 0.0602 million every year) begins in 1989-1990. The capital recovery factor for 20 years at 8.5% is 0.1057.

7. Increase in accounts is receivable in the difference of amount in two consecutive years as given in the projected balance sheet.

(C) BALANCE SHEET (APPENDIX 17.9)

1. No provision is made for the bed debts.

2. 20 percent of the sales revenue is shown as accounts receivables every year.

3. No figures are assumed for inventories.

4. 10 percent of operating and maintenance expenditure is shown as accounts payables every year.

5. Approximate values are given to old assets under the sectors.

APPENDIX 17.4

INCOME AND EXPENDITURE STATEMENT OF WATER SUPPLY AND SEWERAGE/SANITATION PROJECT

	HISTORICAL					FORECAST				
	1982-83	83-84	84-85	85-86	86-87	87-88	88-89	89-90	90-91	91-92
1. Water produced (kld)	-	-	-	-	-	495	514	529	544	588
2. Water sold (kld)	-	-	-	-	-	396	411	423	435	470
3. Revenue (Rs. million)										
A. Water Supply	-	-	-	-	-	0.1326	0.1375	0.1437	0.1513	0.1655
B. Sanitation	-	-	-	-	-	-	-	-	-	-
Total Revenue (A+B)	-	-	-	-	-	0.1326	0.1375	0.1437	0.1513	0.1655
4. Operating Expenses (Rs. million)										
A. Water Supply										
i. Estt. Charges	-	-	-	-	-	0.0510	0.0625	0.0541	0.0557	0.0574

	HISTORICAL					FORECAST				
	1982-83	83-84	84-85	85-86	86-87	87-88	88-89	89-90	90-91	91-92
ii. Power	-	-	-	-		0.0116	0.0131	0.0135	0.0139	0.0168
iii. Chemicals	-	-		-	-	0.0011	0.0012	0.0013	0.0013	0.0014
iv. Repair and Renewals	-	-	-		-	0.0088	0.0093	0.0099	0.0105	0.0111
v. Royalty charges	-	-		-	-	0.0004	0.0004	0.0004	0.0004	0.0004
vi. Other (specify)		-	-	-	-	0.0007	0.0008	0.0008	0.0008	0.0009
vii. Administrative Expenses				-		0.0015	0.0015			
Sub Total (A)	-		-	-	-	0.0751	0.0782	0.0800	0.0826	0.0860
B. Sewerage/Sanitation	-	-			-			-	-	
i. Estt. charges	-		-	-				-		-
ii. Power	-	-	-	-	-	-	-			-

	HISTORICAL					FORECAST				
	1982-83	83-84	84 85	85-86	86-87	87-88	88-89	89-90	90-91	91-92
iii. Chemicals	-	-	-	-	-	-	-	-	-	-
iv. Administrative expenses	-	-	-	-	-	-	-	-	-	-
v. Others(specify)	-	-	-	-	-	-	-	-	-	-
Sub Total (B)	-	-	-	-	-	-	-	-	-	-
Total Operating Expenses (A+B)	-	-	-	-	-	0.0751	0.0788	0.0800	0.0826	0.0880
5. Income before Depreciation and interest (Rs. million)	-	-	-	-	-	0.0575	0.0587	0.0637	0.0687	0.0775

	HISTORICAL					FORECAST				
	1982 83	83-84	84-85	85-86	86-87	87-88	88-89	89-90	90-91	91-92
6. Depreciation (Rs. million	-	-	-	-	-	0.0452	0.0463	0.0474	0.0474	0.0474
7. Income before interest (Rs. million)	-	-	-	-	-	0.0123	0.0124	0.0163	0.0213	0.0301
8. Interest (Rs. million)	-	-	-		-	-	-	0.0434	0.0474	0.0463
9. Income after interest (Rs. million)	-	-	-	-	-	0.0123	0.0124	0.0321	0.0261	0.0162
10. Operating Ratio**	-	-	-	-	-	0.57	0.57	0.89	0.86	0.81

**Operating Ratio = Total O & M Cost and Interest/ Total Revenue

APPENDIX 17.4(Contd.)

	FORECAST									
	92-93	93-94	94-95	95-96	96-97 to 2000-01	01-02 to 05-06	06-07 to 10-11	11-12	12-13	13-14
1. Water produced (kld)	593	599	605	614	621	695	770	820	820	820
2. Water sold (kld)	474	479	484	491	497	556	615	656	656	656
3. Revenue (Rs. million)										
A. Water supply	0.1772	0.1836	0.1876	0.1908	1.0619	1.3410	1.6937	0.3809	0.4027	0.4030
B. Sanitation	-	-	-	-	-	-	-	-	-	-
Total Revenue(A+B)	0.1772	0.1836	0.1876	0.1908	1.0619	1.3410	1.6937	0.3809	0.4027	0.4030
4. Operating Expenses(Rs. million)										
A. Water Supply										
i. Estt. Charge	0.0591	0.0609	0.0627	0.0646	0.3533	0.4097	0.4748	0.1037	0.1068	0.1100
ii. Power	0.0168	0.0170	0.0189	0.0191	0.1060	0.1380	0.1823	0.0413	0.0453	0.0453
iii. Chemicals	0.0015	0.0016	0.0017	0.0018	0.0134	0.0178	0.0286	0.0068	0.0072	0.0076
iv. Repairs & Renewals	0.0118	0.0125	0.0132	0.0140	0.0839	0.1122	0.1500	0.0356	0.0378	0.0401
v. Royalty charges	0.0004	0.0004	0.0004	0.0004	0.0025	0.0025	0.0030	0.0006	0.0006	0.0006
vi. Others (specify)	0.0009	0.0009	0.0010	0.0010	0.0055	0.0069	0.0084	0.0019	0.0020	0.0020

	FORECAST									
	92-93	93-94	94-95	95 96	96-97 to 2000-01	01-02 to 05-06	06-07 to 10-11	11-12	12-13	13-14
vii. Administrative expenses	-	-	-	-	-	-	-	-	-	-
Sub total (A)	0.0905	0.0932	0.0979	0.1009	0.5646	0.6871	0.8471	0.1899	0.1997	0.2056
B. Sewerage / Sanitation										
i. Estt. charge	-									
ii. Power	-	-	-	-	-	-	-	-	-	-
iii. Chemicals	-	-	-	-	-	-	-	-	-	-
iv. Administrative expenses	-	-	-	-	-	-	-	-	-	-
v. Others (specify)	-	-	-	-	-	-	-	-	-	-
vi. Sub Total (B)	-	-	-	-	-	-	-	-	-	-
Total Operating Expenses (A+B)	0.0905	0.0932	0.0979	0.1009	0.5646	0.6371	0.8471	0.1849	0.1997	0.2056
5. Income before depreciation and interest(Rs. Million)	0.0867	0.0904	0.0897	0.0899	0.4973	0.6539	0.8466	0.1910	0.2030	0.1974

	FORECAST									
	92-93	93 94	94-95	95 96	96-97 to 2000-01	01-02 to 05-06	06-07 to 10-11	11-12	12-13	13-14
6. Depreciation (Rs.million)	0.0474	0.0474	0.0474	0.0474	0.2370	0.2370	0.2370	0.0474	0.0474	0.0474
7. Income before interest(Rs.million)	0.0393	0.0430	0.0423	0.0425	0.2603	0.4169	0.6096	0.1436	0.1556	0.1500
8. Interest (Rs. million)	0.0451	0.0438	0.0424	0.0409	0.1771	0.1148	0.0284		-	-
9. Income after interest(Rs. Million)	0.0058	0.0008	0.0001	0.0016	0.0832	0.3021	0.5812	0.1436	0.1556	0.1500
10. Operating Ratio**	0.77	0.75	0.75	0.74	0.70	0.60	0.52	0.50	0.50	0.51

** Operating Ratio = Total O &M Cost and Interest /Total Revenue

APPENDIX 17.5

Funding Pattern

	Description	Ist year	II year	Total
		(1985-86)	(1986-87)	
			(Rs. in million)	
1	Loan Component	0.2585	0.1835	0.4420
2	Grant in aid from GTN	0.7755	0.5505	1.3260
	Total	1.0340	0.7340	1.7680

APPENDEIX 17.6

PROJECT SOURCES AND APPLICATION OF FUNDS (CASH FLOW) STATEMENT

	Historical							Forecast				
	82-83	83-84	84-85	85-86	86-87	87-88	88-89	89-90	90-91	91-92	92-93	93-94
						(Rs. million)						
I. Sources												
1. Net income after depreciation	-	-	-	-	-	0.0123	0.0124	0.0163	0.0213	0.0301	0.0393	0.0430
2. Depreciation	-	-	-	-	-	0.0452	0.0463	0.0474	0.0474	0.0474	0.0474	0.0474
3. Increase in accounts payable	-	-	-	-	-	0.0076	0.0003	0.0001	0.0003	0.0005	0.0002	0.0003
4. Increase in other current liabilities(interest capitalised)	-	-	-	0.0140	0.0309	0.0410	0.0445	-	-	-	-	-

	Historical						Forecast					
	82 83	83-84	84-85	85-86	86-87	87-88	88-89	89-90	90-91	91-92	92-93	93-94
	(Rs. million)											
5. Increase in long term loan	-	-	-	0.2585	0.1835	-	-	-	-	-	-	-
6. Increase in grant in aid	-	-	-	0.7755	0.5505	-	-	-	-	-	-	-
7. Others(specify)	-		-									
Total Sources	-	-	-	1.0450	0.7649	0.1061	0.1035	0.0638	0.069[illegible]	0.0780	0.0869	0.0907
II. Applications												
1. Increase in fixed assets	-	-	-	1.0450	0.7449	0.0410	0.0445				-	-
2. Increase in current assets	-	-	-	-	-	-	-	-	-		-	-
3. Increase in accounts receivables	-	-	-	-	-	0.0265	0.0010	0.0012	0.0016	0.0028	0.0023	0.0013
4. Decrease in current liabilities	-	-	-	-	-	-	-	-	-	-	-	-

	Historical							Forecast				
	82-83	83-84	84-85	85-86	86-87	87-88	88-89	89-90	90-91	91-92	92-93	93-94
	(Rs. million)											
5. Interest capitalised	-	-	-	-	-	-	-	-	-	-	-	-
6. Debt service** interest	-	-	-	-	-	-	-	0.0484	0.0474	0.0463	0.0451	0.0438
7. Debt service principal**	-	-	-	-	-	-	-	0.0118	0.0128	0.0139	0.0151	0.0164
Total Application	-	-	-	1.0450	0.7649	0.0675	0.0455	0.0614	0.0618	0.0630	0.0625	0.0615
III. Cash Surplus(Defic-it) for the year	-	-	-	-	-	0.0386	0.0580	0.0024	0.0072	0.0150	0.0244	0.0292
IV. Cash Surplus(Deficit) at the beginning of the year	-	-	-	-	-	-	0.0386	0.0966	0.0990	0.1062	0.1212	0.1456

		Historical					Forecast						
		82-83	83-84	84-85	85-86	86-87	87-88	88-89	89-90	90-91	91-92	92-93	93-94
		(Rs. million)											
V.	Cash Surplus(Deficit) at the end of the year	-	-	-	-	-	0.0386	0.0966	0.0990	0.1862	0.1212	0.1456	0.1748

APPENDIX 17.7

INTEREST ADDED TO THE CAPITAL DURING MORATORIUM PERIOD

		Interest(Rs. in million)			
		1985-86	1986-87	1987-88	1988-89
I.	Loan amount received in 1985-86(Rs. 0.2585 million)	*0.0110	0.0229	0.0249	0.0269
II.	Loan amount received in 1986-87(Rs. 0.1835 million)	-	0.0080	0.0161	0.0176
III.	Loan amount received in 1987-88(Rs.million)	-	-	-	-
IV.	Loan amount received in 1988-89(Rs. – million)	-	-	-	-
Total		0.0110	0.0309	0.0410	0.0445
			0.1274		

* Interest at 8.5 percent for six months is calculated for the year of receipt of loan.

APPENDIX 17.8

CALCULATION OF ANNUITY

	(Rs. Million)
Loan amount received in 1985-86	0.2585
Loan amount received in 1986-87	0.1835
Loan amount received in 1987-88	-
Loan amount received in 1988-89	-
Interest due (added to the capital) during the moratorium period viz.	
1985-86 to 1988-89	0.1274
Total	0.5694
Capital Recovery Factor at 8.5% for 20 years	= 0.1057
Annuity	= 0.5694 x 0.1057
	= 0.0602

CALCULATION OF PRINCIPAL & INTEREST IN ANNUITY

Year	Annuity	Interest	Principal(Annuity interest)	Loan outstanding
		(Rs. in million)		
1989-90	0.0602	0.0484	0.0118	0.5576
1990-91	0.0602	0.0474	0.0128	0.5448
1991-92	0.0602	0.0463	0.0139	0.5309
1992-93	0.0602	0.0451	0.0151	0.5158
1993-94	0.0602	0.0438	0.0164	0.4994

APPENDIX 17.9

PROJECT BALANCE SHEET (WATER SUPPLY & SANITATION) AS ON 31ST MARCH

	Historical							Forecast				
	1983 (82-83)	1984 (83-84)	1985 (84-85)	1986 (85-86)	1987 (86-87)	1988 (87-88)	1989 (88-89)	1990 (89-90)	1991 (90-91)	1992 (91-92)	1993 (92-93)	1994 (93-94)
	(Rs. million)											
Assets												
I. Current assets												
i. Cash	-	-	-	-	-	0.0386	0.0966	0.0990	0.1062	0.1212	0.1456	0.1748
ii. Accounts receivable	-	-	-	-		0.0265	0.0275	0.0287	0.0303	0.0331	0.0354	0.0367
iii. Inventories (including chemicals, pipes, spare parts	-	-	-	-	-	-	-	-	-	-	-	-

	Historical								Forecast			
	1983 (82-83)	1984 (83-84)	1985 (84-85)	1986 (85-86)	1987 (86-87)	1988 (87-88)	1989 (88-89)	1990 (89-90)	1991 (90-91)	1992 (91-92)	1993 (92-93)	1994 (93-94)
	(Rs. million)											
etc.)												
Total current assets	-	-	-	-	-	0.0651	0.1241	0.1277	0.1365	0.1543	0.1810	0.2115
II. Fixed assets												
A. Water supply												
i. Gross fixed assets in operation	-	-	-	1.0450	1.8099	1.8509	1.8502	1.8039	1.7565	1 7091	1.6617	1.6143
ii. Depreciation	-	-	-	-	-	0.0452	0.0463	0.0474	0.0474	0.0474	0.0474	0.0474
iii. Net fixed assets in operation	-	-	-	1.0450	1.8099	1.8057	1.8039	1.7565	1.7091	1.6617	1.6143	1.5669

	Historical						Forecast					
	1983 (82-83)	1984 (83-84)	1985 (84-85)	1986 (85-86)	1987 (86-87)	1988 (87-88)	1989 (88-89)	1990 (89-90)	1991 (90-91)	1992 (91-92)	1993 (92-93)	1994 (93-94)
	(Rs million)											
B. Sewerage/ sanitation												
i. Gross fixed assets in operation	-	-	-	-	-		-	-		-	-	
ii. Depreciation	-	-	-	-	-		-	-	-	-	-	-
iii. Net fixed assets in operation	-	-	-	-	-	-	-	-	-	-	-	-
Total fixed assets	-	-	-	1.0450	1.8099	1.8057	1.8039	1.7565	1.7091	1.6617	1.6143	1.5669

	Historical							Forecast				
	1983 (82-83)	1984 (83-84)	1985 (84-85)	1986 (85-86)	1987 (86-87)	1988 (87-88)	1989 (88-89)	1990 (89-90)	1991 (90-91)	1992 (91-92)	1993 (92-93)	1994 (93-94)
	(Rs. million)											
III. Expenditure to be written off:	-	-	-	-	-	-	-	0.0074	0.0335	0.0497	0.0555	0.0563
Total Assets (I+II+III)	-	-	-	1.0450	1.8099	1.8708	1.9280	1.8916	1.8791	1.8657	1.8508	1.8347

APPENDIX 17.9 (CONTD.)

PROJECT BALANCE SHEET (WATER SUPPLY & SANITATION) AS ON 31ST MARCH

	Historical							Forecast				
	1983 (82-83)	1984 (83-84)	1985 (84-85)	1986 (85-86)	1987 (86-87)	1988 (87-88)	1989 (88-89)	1990 (89-90)	1991 (90-91)	1992 (91-92)	1993 (92-93)	1994 (93-94)
						(Rs. million)						
Liabilities												
I. Current Liabilities												
i) Account payable	-	-	-	-	-	0.0076	0.0079	0.0080	0.0083	0.0088	0.0090	0.0093
II. Equity												
A. Water supply scheme												
i) Grant-in-aid	-	-	-	0.7755	1.3260	1.3260	1.3260	1.3260	1.3260	1.3260	1.3260	1.3260
ii) Surplus	-	-	-	-	-	0.0123	0.0247	-	-		-	-
B. Sewerage/ Sanitation Scheme												

			Historical						Forecast			
	1983	1984	1985	1986	1987	1988	1989	1990	1991	1992	1993	1994
	(82-83)	(83-84)	(84-85)	(85-86)	(86-87)	(87-88)	(88-89)	(89-90)	(90-91)	(91-92)	(92-93)	(93-94)
						(Rs. million)						
i).Localbody contribution (for capital expenditure)		-	-	-	-	-	-	-	-	-	-	-
ii)Retained earnings or surplus	-	-	-	-	-	-	-	-	-	-	-	-
Total equity	-	-	-	-	-		-	-	-	-	-	-
III. Long Term Loan												
A. Water Supply Scheme	-	-	-	0.2585	0.4420	0.4420	0.4420	0.4420	0.4302	0.4174	0.4035	0.3889
B.Sewerage/ Sanitation Scheme	-	-	-	-	-	-	-	-	-	-	-	-
C. Interest added to the Capital	-	-	-	0.0110	0.0419	0.0829	0.1274	0.1274	0.1274	0.1274	0.1274	0.1274

	Historical							Forecast				
	1983	1984	1985	1986	1987	1988	1989	1990	1991	1992	1993	1994
	(82-83)	(83-84)	(84-85)	(85-86)	(86-87)	(87-88)	(88-89)	(89-90)	(90-91)	(91-92)	(92-93)	(93-94)
	(Rs. million)											
D. Current maturities	-	-	-	-	-	-	-	0.110	0.0128	0.0139	0.0151	0.0164
Total long term Loan(A+B+C-D)	-	-	-	0.2695.	0.4833	0.5249	0.5694	0.5576	0.5440	0.5309	0.5158	0.4994
Total liabilities	-	-	-	1.6450	1.8099	1.8708	1.9280	1.8916	1.8791	1.8657	1.8508.	1.8347

* Interest accrued during the year added to the fixed assets.

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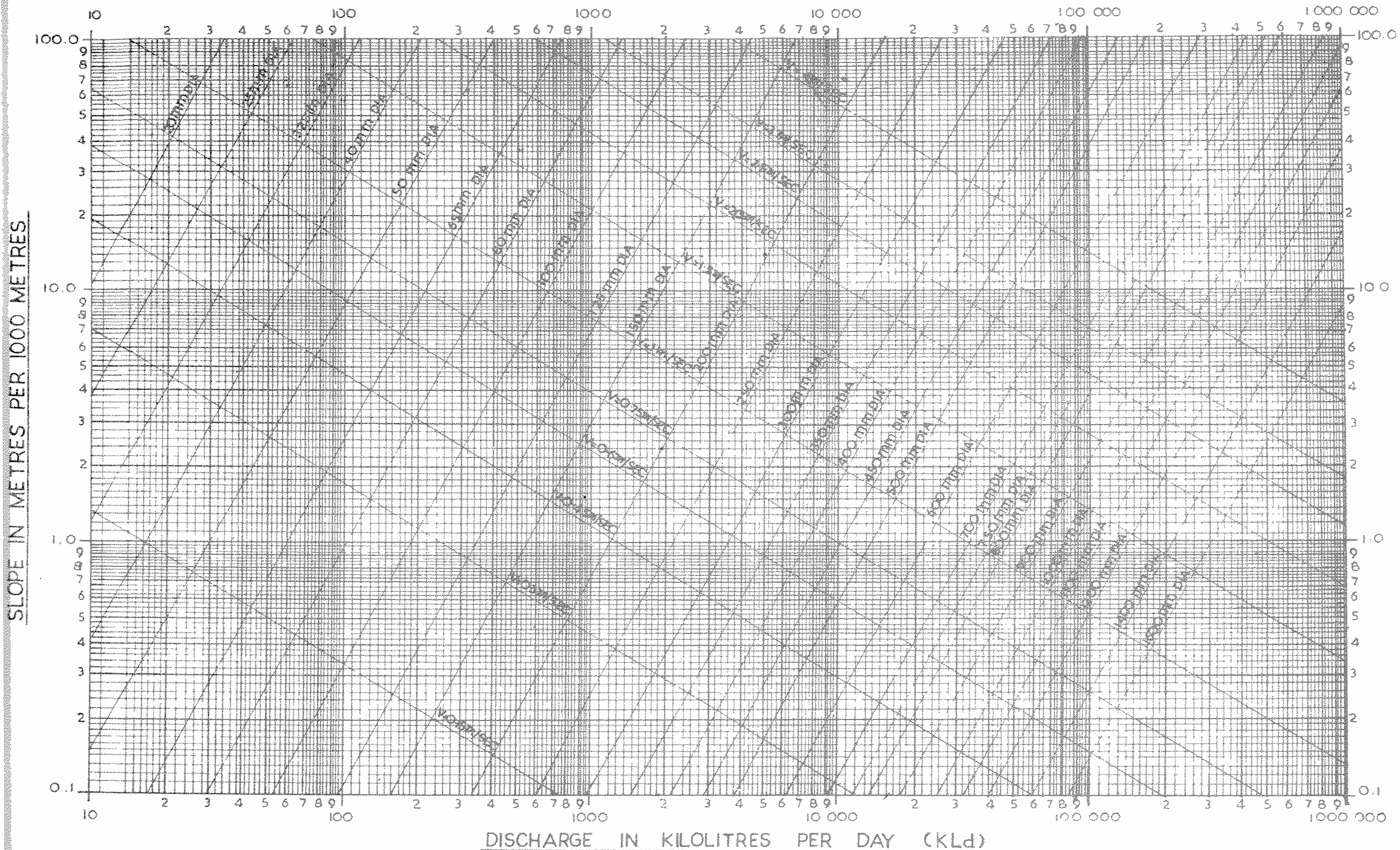

NOTE :- FOR VALUES OF C_R LESS THAN 1
MULTIPLY GIVEN 'Q' OR 'V' BY (C_1/C_R) TO FIND 'S' OR
MULTIPLY FIND 'Q' OR 'V' BY $C_R/1$ FOR GIVEN 'S'

CHART FOR DISCHARGE ETC ACCORDING TO
MODIFIED HAZEN & WILLIAMS FORMULA FOR $C_R = 1$
(FOR FLOW OF WATER AT 20° C)

$V = 143.534\ C_R\ r^{0.6575}\ S^{0.5525}$

V = VELOCITY OF WATER IN METRES FOR SECOND

r = HYDRAULIC RADIUS OF PIPE IN METRES

S = SLOPE LOSS OF HEAD IN METRES PER 1000 METRES

APPENDIX 6.1

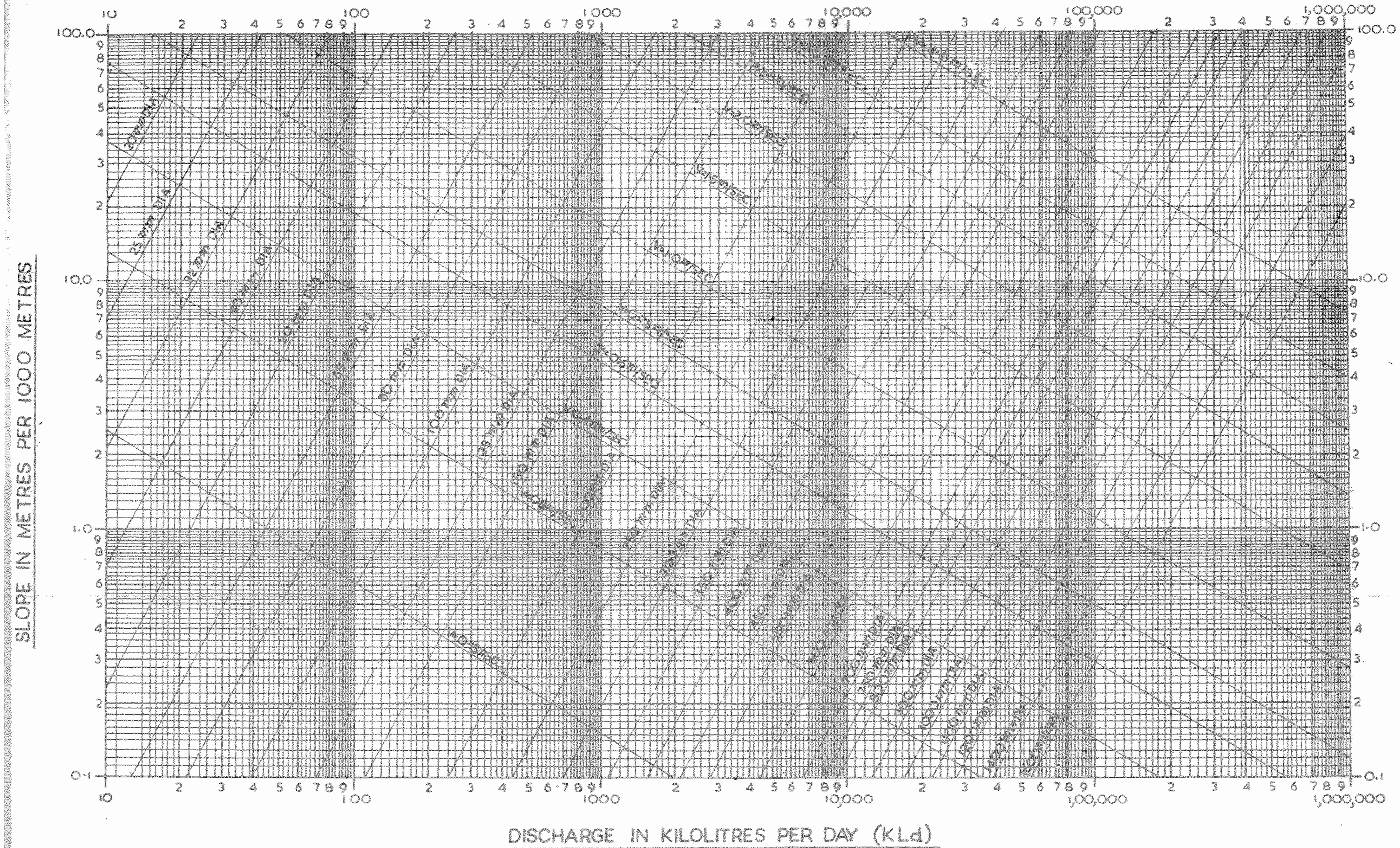

CHART FOR DISCHARGE ETC. ACCORDING TO HAZEN & WILLIAMS FORMULA

1. $v = 4.567 \times 10^{-3} \times C \times d^{0.63} \times s^{0.54}$
2. $Q = 3.1 \times 10^{-4} \times C \times d^{2.63} \times s^{0.54}$
3. $V = 0.849\, C\, r^{0.63}\, s^{0.54}$

WHERE

- v = VELOCITY IN METRES PER SECOND
- Q = DISCHARGE IN KILOLITRES PER DAY (KLd)
- d = DIA OF CIRCULAR PIPE IN mm
- s = SLOP
- C = HAZEN AND WILLIAMS COEFFICIENT OF 100 ADOPTED
- r = HYDRAULIC RADIUS IN METRES

NOTE:-

FOR ANY OTHER VALUE OF 'C' SAY 'C_1' THE VALUES OF v AND Q AS FOUND FOR A GIVEN d AND s FROM THE CHART ARE TO BE MULTIPLIED BY A FACTOR $K = \left(\frac{C_1}{100}\right)$ AND FOR A GIVEN d AND Q OR v, THE VALUE OF SLOP AS FOUND FROM THE CHART HAS HAS TO BE MULTIPLIED BY A FACTOR $K_2 = \left(\frac{100}{C_1}\right)^{1.85}$

VALUE OF C	70	80	90	100	110	120	130	140
K_1	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
K_2	1.94	1.51	1.22	1.0	0.84	0.71	0.62	0.54

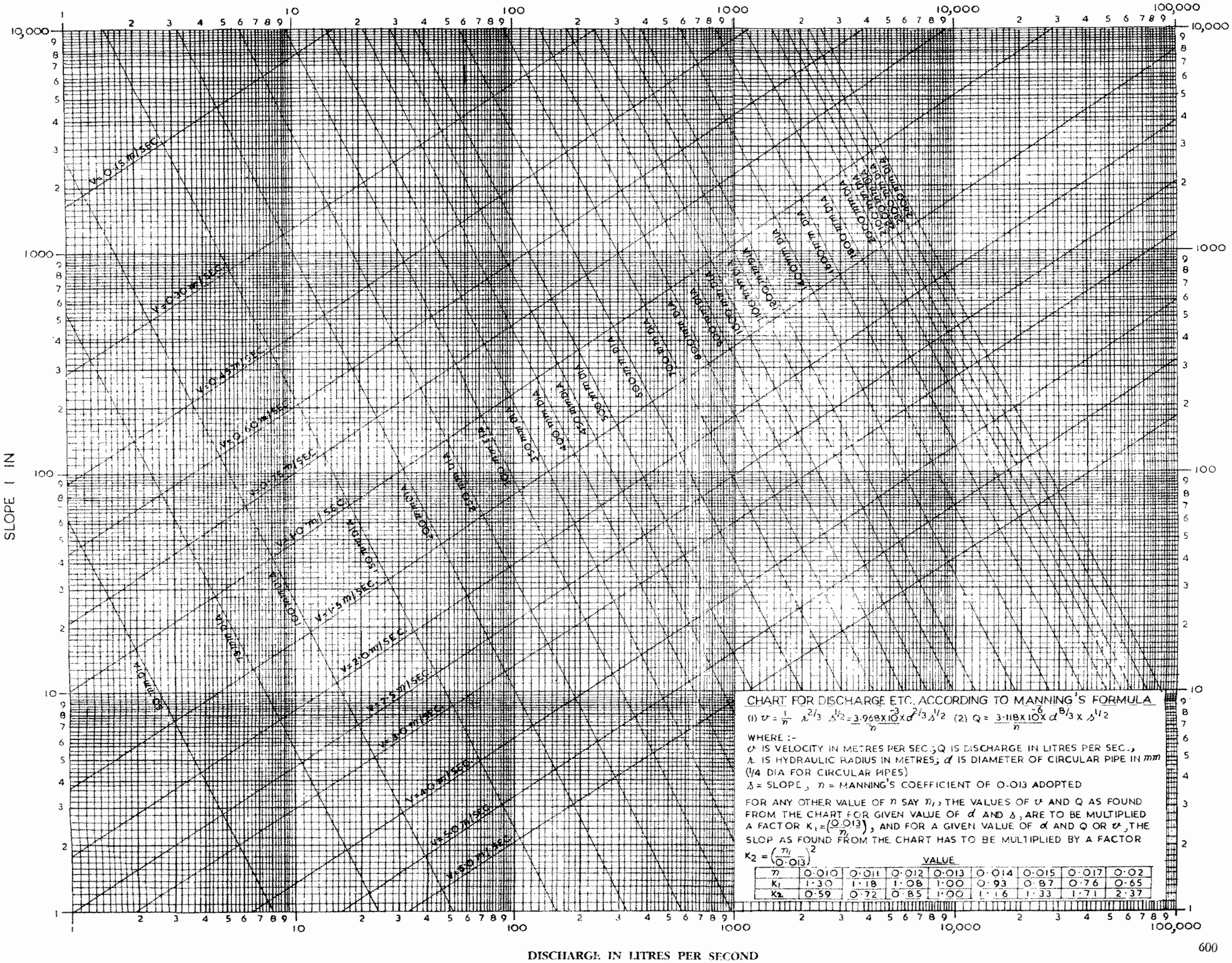
CHART FOR DISCHARGE ETC. ACCORDING TO MANNING'S FORMULA
(1) $v = \frac{1}{n} r^{2/3} s^{1/2} = \frac{3.968 \times 10^{-3}}{n} \times d^{2/3} s^{1/2}$ (2) $Q = \frac{3.118 \times 10^{-6}}{n} d^{8/3} \times s^{1/2}$
WHERE :-
v IS VELOCITY IN METRES PER SEC.; Q IS DISCHARGE IN LITRES PER SEC.,
r IS HYDRAULIC RADIUS IN METRES; d IS DIAMETER OF CIRCULAR PIPE IN mm
(1/4 DIA FOR CIRCULAR PIPES)
s = SLOPE, n = MANNING'S COEFFICIENT OF 0.013 ADOPTED
FOR ANY OTHER VALUE OF n SAY n_1, THE VALUES OF v AND Q AS FOUND FROM THE CHART FOR GIVEN VALUE OF d AND s, ARE TO BE MULTIPLIED A FACTOR $K_1 = \left(\frac{0.013}{n_1}\right)$, AND FOR A GIVEN VALUE OF d AND Q OR v, THE SLOP AS FOUND FROM THE CHART HAS TO BE MULTIPLIED BY A FACTOR $K_2 = \left(\frac{n_1}{0.013}\right)^2$
VALUE
n 0·010 0·011 0·012 0·013 0·014 0·015 0·017 0·02
K_1 1·30 1·18 1·08 1·00 0·93 0·87 0·76 0·65
K_2 0·59 0·72 0·85 1·00 1·16 1·33 1·71 2·37
SLOPE 1 IN
DISCHARGE IN LITRES PER SECOND
V=0.15 m/SEC.
V=0.30 m/SEC.
V=0.45 m/SEC.
V=0.60 m/SEC.
V=0.75 m/SEC.
V=1.0 m/SEC.
V=1.5 m/SEC.
V=2.0 m/SEC.
V=2.5 m/SEC.
V=3.0 m/SEC.
V=4.0 m/SEC.
V=5.0 m/SEC.
V=6.0 m/SEC.
50 mm DIA
75 mm DIA
100 mm DIA
150 mm DIA
200 mm DIA
250 mm DIA
300 mm DIA
350 mm DIA
400 mm DIA
450 mm DIA
500 mm DIA
600 mm DIA
700 mm DIA
800 mm DIA
900 mm DIA
1000 mm DIA
1200 mm DIA
1400 mm DIA
1600 mm DIA
1800 mm DIA
2000 mm DIA
2200 mm DIA
2400 mm DIA
2600 mm DIA

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