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MANUAL ON SEWERAGE AND SEWAGE TREATMENT

(FIRST EDITION)

Prepared by
THE EXPERT COMMITTEE

Constituted by
THE GOVERNMENT OF INDIA

CENTRAL PUBLIC HEALTH
AND ENVIRONMENTAL ENGINEERING ORGANISATION

**MINISTRY OF WORKS AND HOUSING,
NEW DELHI**

1980

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AND
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NEW DELHI
1980

सत्यमेव जयते

मंत्री
निर्माण और आवास,
भारत
**Minister of
Works and Housing,
India**

New Delhi, August 18, 1980
Q-16011/2/79-CPHEEO

FOREWORD

Sanitation though motivated primarily for meeting the ends of preventive health has come to be recognised as a way of life. In this context, development of the sanitation infra-structure of any country could possibly serve as a sensitive index of its level of prosperity. It is needless to emphasise that for attaining the goals of good sanitation, sewerage and sewage treatment is a necessary concomitant. While provision of potable drinking water takes precedence in the order of provision of environmental engineering services, the importance of sewerage and sewage treatment cannot be lost sight of and cannot be allowed to lag behind, as all the water used by the community has to flow back as sewage loaded with all the wastes of community living. Unless properly collected, treated and disposed of, this would create serious water pollution problems. More prosperous communities have attempted to look at water supply and sewerage as an integral whole but such an outlook has not been possible in this country, primarily because of the paucity of resources.

2. *Nevertheless, there has been a steady effort in the recent years in our country to provide sewerage and sewage treatment facilities to cover at least the major towns particularly in the context of growing population. To facilitate the practising engineer in the work of adequate design, implementation and management of these facilities, a manual is absolutely necessary.*

3. *It is gratifying to note that the Expert Committee appointed by the Government for preparing a Manual on this subject have been able to bring out this useful volume. Its usefulness has been all the more enhanced since this Manual has been discussed at length and reviewed critically at a Workshop of Chief Engineers and Senior Engineers, specially convened for this purpose. I am happy that this volume is being brought out at an appropriate time since the nation is becoming more and more conscious of protection of the environment and a major thrust is being given by the Government towards meeting this end as evidenced by the keen interest evinced by our beloved Prime Minister on this subject. We are on the threshold of the International Decade for Drinking Water Supply and Sanitation and I am sure that this Manual would meet the professional needs of all the Public Health Engineers and Scientists in this field to discharge their task that would be demanded of them in the Decade.*

(P. C. SETHI)

PREFACE

Since the inception of the National Water Supply and Sanitation Programme and the streamlining of the Public Health Engineering Departments in the States to keep pace with the developments in this field of specialisation, there has been a consistent demand from the Engineers in the profession for a Manual on Sewerage and Sewage treatment. The Manual on Water Supply and Treatment had necessarily to precede this Manual because of the greater emphasis and resource allocations that were given to community water supplies. With the increasing importance bestowed to the sanitation aspect under the programme in recent years, the need for a Manual on the subject has been keenly felt. To achieve this objective, a Committee was set up by the Government of India in the then Union Ministry of Health and Family Planning in August, 1971, with the following composition:

1. Shri J. M. Dave, Adviser (PHE), Ministry of Health & FP, (Deptt. of Health), New Delhi. — *Chairman*
2. Prof. S. J. Arceivala, Director, Central Public Health Engineering Research Institute, Nehru Marg, Nagpur. — *Member*
3. Prof. N. Majumdar, Prof. of Sanitary Engineering, All India Institute of Hygiene and Public Health, Calcutta-12. — ,,
4. Shri A. C. Chaturvedi, Superintending Engineer, L.S.G.E.D., Govt. of Uttar Pradesh, Lucknow. — ,,
5. Shri V. D. Desai, City Engineer, Bombay Municipal Corpn. — *Member*
6. Shri D. R. Singal, Chief Engineer, P.W.D., Public Health Branch, Punjab, Patiala. — ,,
7. Shri S. Chatterjee, Chief Engineer, Calcutta Metropolitan Planning Organisation, Calcutta. — ,,
8. Shri T. Durairaj, Deputy Adviser (PHE), Ministry of Health & FP, New Delhi. — *Member Secretary*

The Committee was reconstituted by the Union Ministry of Works and Housing (to which the subject sanitation was transferred) in June, 1973, with the following:

1. Adviser (PHEE), Central Public Health & Environmental Engineering Organisation, Ministry of Works and Housing, New Delhi. — *Chairman*
2. Dr. T. R. Bhaskaran, Technical Director, Geo-Miller & Co., New Delhi. — *Member*
3. Shri A. C. Chaturvedi, Chief Engineer, LSGED (UP), Lucknow; presently, Director, Department of Ecology, Lucknow. — ,,
4. Shri D. R. Singal, Chief Engineer, P.W.D., Public Health Branch, Patiala; presently, Chairman, Punjab State Board for Prevention & Control of Water Pollution, Patiala. — ,,
5. Dr. R. H. Siddique, Scientist, National Env. Engg. Research Institute, Nagpur, presently, Associate Prof. of Civil Engineering, University of Petroleum & Chemicals, Saudi Arabia. — *Member*
6. Shri S. Chatterjee, Chief Engineer (Retd.), Calcutta Metropolitan Planning Organisation, Calcutta. — ,,
7. Shri Ajitha Simha, Director (CED), Indian Standards Institution, New Delhi. — ,,
8. Shri V. D. Desai, Special Commissioner (Engg.) Municipal Corporation, Bombay. — ,,
9. Shri T. Durairaj, Deputy Adviser (PHE) (Retd.), presently, Consultant, Shah Construction Company, Madras. — ,,

10. Prof. S. Subba Rao, All India Institute of Hygiene and Public Health Calcutta. — Member

11. Shri B. B. Rau, Deputy Adviser (PHE), Ministry of Works and Housing, New Delhi. — *Member-Secretary*

Seven meetings were held under the Chairmanship of Shri J. M. Dave and eight meetings were held when Shri B. B. Rau officiated as Adviser (PHEE). Shri T. S. Swamy took over as Adviser (PHEE) on 26-3-74 and the material was reviewed for finalisation. The Manual has been finalised after Dr. Nilay Chaudhuri took over the charge of Adviser (PHEE) in December, 1977.

In the preparation of this Manual the Committee has made extensive use of all the available literature on the subject and wishes to place on record its gratitude to the sources. The Committee thanks the Ministries of Health & Family Welfare and Works & Housing, Government of India, for affording all facilities. The Committee wishes to place on record its deep appreciation of all the assistance provided and arrangements made by the All India Institute of Hygiene and Public Health, Indian Standards Institution and National Environmental Engineering Research Institute for the meetings held outside Delhi. Special thanks are due to Sarvashri V. Raman, Head, Sewage Division, National Environmental Engineering Research Institute, Nagpur; A. Raman, Scientist Incharge, Delhi Zonal Centre, National Environmental Engineering Research Institute; Dr. R. N. Chakravarthy, Managing Director, Universal Enviroscience Pvt. Ltd., Delhi; Dr. A. M. Michael, Dr. K. V. Paliwal and Dr. S. L. Pandey of the Water Technology Centre, Indian Agriculture Research Institute, Delhi; and Dr. S. S. Ramaswamy, Deputy Director General, Central Labour Institute, Bombay, for their valuable contributions.

The Committee expresses its appreciation to Shri B. B. Rau for his untiring efforts in making possible the completion of the Manual in spite of his arduous normal duties. Special mention is made of the services of Shri M. R. Parthasarathy, Asstt. Adviser (PHE) and Dr. I. Radhakrishnan, Scientific Officer, who unstintingly devoted their time even after office hours in all phases of this work. The valuable contributions of Asstt. Advisers (PHE), Shri M. M. Datta, Dr. S. R. Shukla and Shri J. D. Sheth (until he rejoined his parent department) are gratefully acknowledged. The Committee also desires to record their appreciation of the services rendered by the different officers and staff members of the Drawing Section and Secretariat of the Central Public Health & Environmental Engineering Organisation.

The type-script of the Manual was circulated to all the State Public Health Chief Engineers and Institutions imparting public health engineering training to elicit their comments and views. The contents of the Manual were again discussed in greater detail, topic by topic, at a Special Workshop of all the Chief and Senior Public Health Engineers of the States convened at Bhubaneswar (Orissa) between 17th and 19th February, 1979, under the Chairmanship of Shri S. T. Khare, the then Adviser (PHEE), Ministry of Works & Housing, New Delhi. The Seminar was also attended by special invitees from the Educational and Research Institutions. Valuable suggestions that emerged during the discussions have been incorporated to make this Manual useful from the practising Engineers' point of view.

The following attended the deliberations of the Workshop :—

1. Shri N. Ramachandra Rao,
   Chief Engineer (Rural Water Supply),
   Panchayati Raj, Andhra Pradesh.
2. Shri S.K. Shah,
   Chief Engineer (Public Health),
   Gujarat.
3. Shri A.T. Upadhyay,
   Superintending Engineer (Public Health),
   Gujarat.
4. Shri L. M. Chaudhuri,
   Engineer-in-Chief
   Public Health Branch, Haryana.
5. Shri A.N. Mehendale,
   Chief Engineer,
   Haryana Urabn Development
   Authority.
6. Shri Gurdia Singh,
   Superintending Engineer,
   Public Health,
   Haryana.
7. Shri I. D. Mirchandani,
   Chief Engineer,
   Public Works Department,
   Himachal Pradesh.
8. Shri B. Subbaiah,
   Chief Engineer,
   Karnataka.
9. Shri D.R. Jagannatha Rao,
   Chief Engineer,
   Public Health Engineering Deptt.
   Madhya Pradesh.
10. Shri W. D. Bhide,
    Addl. Chief Engineer,
    Urban Development and
    Public Health Deptt. and
    Member Secretary,
    Water Research Management Board,
    Maharashtra.
11. Shri G.F. Khambati,
    Chief Engineer, Bombay,
    Municipal Corporation, Bombay.
12. Shri Y. V. Damle,
    Deputy Chief Engineer (Sewerage)
    Planning and Design, Municipal
    Corporation of Greater Bombay.
13. Shri B. K Singh,
    Chief Engineer,
    Public Health Engineering,
    Manipur, Imphal.
14. Shri P. Arunachalam,
    Chief Public Health Engineer,
    Meghalaya.
15. Shri Akangameron Ao,
    Superintending Engineer,
    Nagaland.
16. Shri D. N. Singh Deo,
    Chief Engineer,
    Public Health, Orissa.
17. Shri M. S. Hyder,
    Superintending Engineer,
    Public Health, Orissa.
18. Shri A. K. Patnaik,
    Superintending Engineer,
    Public Health, Orissa.
19. Shri N. Mishra,
    Superintending Engineer,
    Public Health, Orissa.
20. Shri G. N. Patra,
    Superintending Engineer,
    Public Health, Orissa.
21. Shri R. N. Pattnaik,
    Superintending Engineer,
    Public Health, Orissa.
22. Shri H. S. Manocha,
    Managing Director,
    Punjab Water Supply and
    Sewerage Board.
23. Shri M. S. Sandhu,
    Chief Engineer,
    Punjab Water Supply and
    Sewerage Board.
24. Shri H. S. Puri,
    Chief Engineer (Rural Water Supply),
    Punjab Public Health
    Deptt.
25. Shri R. Krishnaswamy, Chief Engineer,
    Tamilnadu Water supply and
    Drainage Board.
26. Shri George Mathew,
    Project Engineer,
    Tamilnadu Water supply and
    Drainage Board.
27. Shri V. Varadarajan,
    Engineering Director, Madras
    Metropolitan Water Supply and
    Sewerage Board.
28. Shri K. N. Dwivedi,
    Managing Director,
    U. P. Jal Nigam, Lucknow.
29. Shri B. P. Varma,
    General Manager,
    Uttar Pradesh Jal Nigam.
30. Shri P. K. Chatterjee,
    Director, Water Supply
    and Environmental Hygiene,
    Calcutta Metropolitan Development
    Authority.
31. Shri S. K. Das Gupta,
    Director, Sewerage and Drainage,
    Calcutta Metropolitan Development
    Authority.
32. Shri A. K. Poddar,
    Chief Engineer,
    Public Health Engineering Directorate,
    West Bengal.

33. Shri Satyabrata Kundu,
Superintending Engineer,
Public Health Engineering Directorate,
West Bengal.

34. Shri S. K. Roy,
Chief Engineer,
Irrigation and Flood Control Deptt.,
Tripura.

35. Shri Mallinath Jain,
Chief Engineer (Water),
Delhi Municipal Corportation.

36. Shri J. Deivasigamany,
Director,
Public Works Department,
Pondicherry-1.

37. Shri D. Guin,
Executive Engineer,
Panchayat and Rural Works Deptt.,
Sikkim.

38. Prof. S. Subba Rao,
Professor of Sanitary Engineering,
All India Institute of Hygiene and
Public Health, Calcutta.

39. Prof. S. V. Patwardhan,
Prof. of Environmental Engineering,
University of Roorkee.

40. Shri V. Raman,
Head,
Sewage Treatment Division,
National Environmental Engineering
Research Institute,
Nagpur.

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INTRODUCTION

With the march of civilization and the growing social consciousness of the communities in the country, there has been an incessant demand for doing away with the existing dry latrines and replacing the same with sanitary water seal latrines. Further, other liquid wastes generated by the communities are now carried in open drains leading to severe environmental problems. To dispose of these liquid wastes in a proper and sanitary manner, it would be necessary to collect them in a system of sewers.

The sewage thus collected has perforce to be disposed of in a body of water, like stream, river, lake or sea. However, proper care has to be ensured that it does not interfere with the other uses of these water bodies, like water supply, recreation, pisciculture etc. Pollution caused by sewage is perhaps more important from the point of view of the health of the community as it contains a number of causative agents for many dangerous diseases, which can easily spread through the water supply systems.

The concern for crying a halt to the menace of pollution of our water environment before it reaches unmanageable proportions has already culminated in the enactment of the Central legislation "Water (Prevention and Control of Pollution) Act, 1974". The most logical solution to arrest the deterioration of the environment lies in the provision of appropriate and adequate treatment facilities for the sewage.

The desirability of viewing water supply and sewerage as an integrated whole is quite obvious in Public Health Engineering practice. There has been a long felt need for forging uniformity in planning, design and construction of sewerage and sewage treatment facilities. With this objective in view, this Manual on Sewerage and Sewage treatment lays emphasis on the various pertinent aspects for execution of a comprehensive sewage collection and treatment scheme to serve the interest of the practising Public Health Engineer.

This Manual has considered the recent technical advances and trends in the field of sewerage and sewage treatment and has incorporated them as appropriate to Indian conditions.

As adopted in the Manual on Water Supply and Treatment, all units of measurements, operational parameters and design criteria have been furnished in the metric system. A table of conversion factors has been included in the appendices for easy verification wherever needed. Other appendices contain useful information which will serve as a guide for solving the problems encountered in the field.

PART I
SEWERAGE

CHAPTER 1

SURVEY AND INVESTIGATION

Survey and investigation are prerequisites both for the framing of the preliminary report and the preparation of a detailed sewerage project. The engineering and policy decisions taken are dependent on the correctness of the data collected and its proper evaluation.

1.1 BASIC INFORMATION

For an effective investigation, a broad knowledge of the problems likely to be faced during the various phases of implementation of the project is essential. Information on physical, developmental, fiscal and other aspects has to be collected.

1.1.1 Physical Aspects

These would necessitate the collection of information relating to :

(i) topography or elevation differences needed for design of sewers and location of outfall and disposal works;

(ii) subsoil conditions, such as types of strata likely to be encountered, depth of ground water table and its fluctuations. In the absence of any records, data should be collected by putting at least 3 trial bores or trial pits per hectare;

(iii) underground structures like storm drains and appurtenances, city survey stones and utility services like house connections for water supply and sewerage, electric and telephone cables and gas lines; and

(iv) location of streets and adjoining areas likely to be merged or annexed.

Possible sources of information are existing maps and plans showing streets from revenue or town surveys or Survey of India maps. Other sources are topographical maps from Survey of India if available with existing spot levels, aerial photographs and photographs of complex surfaces for supplementing the existing instrumental surveys by concerned authorities like Municipalities and Road Departments.

1.1.2 Developmental Aspects

The following considerations should be taken into account :

(i) types of land use, such as commercial, industrial, residential and recreational;

(ii) density of population, trends of population growth and demographic studies;

(iii) type and number of industries for determining quantity and nature of wastes and location of their discharge points;

(iv) existing drainage and sewerage facilities and data relating to them;

(v) flow in existing sewers and sewers of similar areas to assess the flow characteristics;

(vi) historical and socio-economical data; and

(vii) basis of design and information on the maintenance of existing sewers.

Possible sources of information are census records, town and metropolitan master plans regional planning records, land use plans, flow gauging records, stream flow records, meteorological data, industrial survey records and reports of the Water pollution prevention Boards.

1.1.3 Fiscal Aspects

The various factors that will have an important bearing are:

(i) existing policies or commitments of obligations which may affect the financing of the project;

(ii) outstanding loan amounts and instalments of repayments;

(iii) availability of Central and State loans, grants-in-aid, loans from other financing bodies such as Life Insurance Corporation, Industrial Development Corporation, International Bank for Reconstruction and Development and other Banks;

(iv) present water rates and sewer-tax and revenue realised from them, size of property plots and land holdings, the economic condition of the community with respect to their tax-paying capacity; and

(v) factors affecting the cost of construction, operation and maintenance.

Some of the information can be obtained from the records relating to Municipal and State Tax Levies, Acts and Rules governing loans, procedures for financing projects and registers and records of the authorities, maintaining water supply and sewerage systems.

1.1.4 Other Aspects

The considerations that are likely to influence are:

(i) changes in political boundaries by physical acquisition or merger of adjacent communities or by possible extension of limits;

(ii) feasibility of multi-regional or multi-municipal systems ;

(iii) prevailing water pollution prevention statutes, other rules and regulations relating to discharge of industrial and domestic wastes ;

(iv) present status of the governmental, semi-governmental or municipal authority sponsoring the project, its capacity, adequacy and effectiveness ; and the desirability of its modification or necessity of a new organisation to satisfactorily implement and maintain the project; and

(v) the inconveniences likely to be caused to the community during execution and the feasibility of minimising them by suitable alignment or location of the components of the system.

Possible sources of information are National Acts, State and Municipal Laws and Byelaws, minutes of the past meetings of the municipal or other governing bodies and discussions with officials, municipal councillors and other local leaders.

1.2 PROJECT SURVEYS

1.2.1 Preliminary Project Surveys

This is concerned with the broad aspects of the project. Data on aspects such as capacity required, basic arrangement and size, physical features affecting general layout and design, availability of effluent disposal facilities, probable cost and possible methods of financing shall be collected to prepare an engineering report describing the scope and the cost of the project with reasonable accuracy. In framing such estimates, due consideration must be given to the escalation of prices of basic materials and their availability. While extreme precision and details are not required in this phase, all the basic data obtained must be reliable.

1.2.2 Detailed Project Surveys

Surveys for this phase form the basis for the engineering design as well as for the preparation of plans and specifications for incorporation in the detailed project report. In contrast to preliminary survey, this survey must be precise and contain all the details that will facilitate the designer to prepare design and construction plans suiting the field conditions. It should include, *inter-alia,* net work of bench marks and traverse surveys to identify the nature as well as extent of the existing underground structures requiring displacement, negotiation or clearance. Such detailed surveys are necessary to establish rights of way, minimise utility relocation costs, obtain better bids and prevent changing and rerouting of lines.

1.3 CONSTRUCTION SURVEYS

All control points such as base lines and bench marks for sewer alignment and grade should be established by the engineer along the route of the proposed construction. All these points should be referenced adequately to permanent objects.

1.3.1 Preliminary layouts

Before starting the work, rights-of-way, work areas, clearing limits and pavement cuts should be laid out clearly to ensure that the work proceeds smoothly. Approach roads, detours, bypasses and protective fencing should also be laid out and constructed prior to undertaking sewer construction work. All layout work must be completed and checked before construction begins.

1.3.2 Setting line and Grade

The transfer of line and grade from control points, established by the engineer, to the construction work should be the responsibility of the executing agency till work is completed.

The methods generally used for setting the line and grade of the sewers are discussed in 7.1.3.

The procedure for establishing line and grade where tunnels are to be employed in sewer systems are discussed in 7.1.2.3.

1.4 INVESTIGATIONS

Investigations may take many forms but generally are directed towards determining the most feasible and practical method of achieving a desired result. On small sewer projects, they may involve not more than an on the spot decision to use conventional minimum standards for a simple gravity extension of an existing system. Larger projects, on the other hand, may have several alternatives, all of which must be considered. Projects envisaging relief to existing systems demand extensive studies for deciding the design capacity and the most appropriate solution.

Proper investigation is a prerequisite for resolving, *inter alia,* the following :

(i) extent of area to be served, pattern of present and future land use, zonal plans of the area with reference to regional sewer plan;

(ii) general arrangement of the system needed and easements required for this arrangement;

(iii) proportion of combined flow to be intercepted for treatment from an existing combined system and possibility of reducing the combined flow;

(iv) estimated present and future flows;

(v) storm frequency or pattern to be adopted for storm sewer design;

(vi) multiplicity of discharge and treatment points for the whole project;

(vii) requirements of other agencies (national, state or local highway departments and railways) particularly with reference to specific locations for crossing, rights-of-way, installation and details of materials of constructions;

(viii) deep gravity sewers along circuitous routes versus shallow sewers with large number of lift stations;

(ix) alternative materials to be used for sewer construction;

(x) cost of construction and operation of the project and ways and means of financing it like taxes, levies and debentures; and

(xi) necessity of establishing a new authority, such as an autonomous board.

CHAPTER 2

PROJECT PREPARATION

Preparation of sewerage projects is normally done by the State public health engineering authority on behalf of the local bodies excepting the metropolitan towns and large industrial undertakings that have separate design organisations for the purpose. The sewerage project needs approval from the competent government authority either at the State or Central level before it is taken up for execution. It is essential, for efficient and speedy execution, that the project is prepared in detail after proper investigation and includes the technical, financial and administrative aspects giving due consideration to economy, accuracy and soundness.

An accurate estimate of both capital and running costs of the project is particularly necessary as the sewerage projects are not directly revenue yielding and are normally funded by loans. At all stages of the preparation of the project and its scrutiny, it is very essential that technical and financial implications of possible alternatives are borne in mind. This will entail examination of alternative routes for sewers, the choice of suitable materials, different diameters of conduits and their gradients. The choice may be between open cut (with or without shoring) and tunnelling or deep gravity sewers and pumping. The design engineer should keep himself abreast of uptodate costs of materials and construction so that the alternatives can be compared both from the technical and economic considerations.

2.1 PROJECT REPORTS

The project reports are prepared in two stages, the preliminary and the detailed ones. The former is undertaken when the local body concerned decides to provide sewerage and sewage treatment facilities, either as a new system or as an improvement to an existing system and makes the necessary authorisation to the public health engineering authority through the appropriate channels. After the appropriate authority has scrutinised and selected the best alternative from the preliminary report, the public health engineering authority is directed to prepare the detailed project report.

2.1.1 Preliminary Project Report

This report, based on the survey and investigations referred to in Chapter 1, should contain briefly :

(a) a historical retrospect leading to the justification of the project;

(b) description of existing facilities, if any;

(c) population studies, analysis and prediction based on a critical appraisal of available demographic data for at least five preceding decades;

(d) location of the water supply headworks for this area and the neighbouring communities;

(e) availability of piped water supply in the area, present and anticipated per capita water supply;

(f) total sewage flow—present and prospective;

(g) discussion of different possible disposal points and their comparative merits;

(h) review of the different possible methods of treatment;

(j) engineering features and economical aspects of the sewer system, an index and a general layout plan with contours at intervals of 2m alongwith a schematic diagram;

(k) basis for computation of surface runoff, drainage area and siting of storm sewage overflow, wherever necessary;

(l) number and location of pumping stations with static and frictional heads on pumps, pump duties and velocities in rising mains;

(m) a comparison of total costs of the alternatives involved—both capital and maintenance, under major subheads;

(n) the most satisfactory alternative for the project as recommended;

(p) probable stages of construction, procurement of proprietary materials for the project and any special problems relating thereto; and

(q) fiscal aspects of the problem including the annual maintenance charges together with the annuities on the capital loan within the assumed period of repayment, financial commitments to the community, repaying potential and methods of raising the capital for the project.

2.1.2 Detailed Project Report

This report is drawn up after a detailed field survey and investigation is carried out (1.2.2 and 1.4). In addition to the data included in the preliminary project report, this would contain detailed zonal plans with contours of 1m intervals, detailed design calculations and working drawings for various structures and other components, detailed hydraulic calculations for the sewers, longitudinal sections of all peripheral, main and trunk sewers and layout plans for the different treatment units.

2.1.2.1 *Capital cost*

The detailed estimates of capital costs would include the following:

a detailed bill of quantities for the different components of the project; an abstract estimate of cost showing priced schedule of quantities with inclusive rates for the several items under each component; and

a general abstract of estimate for the entire project based on the individual estimates, based on an itemised schedule of costs dealing with each component of the project, viz.,

(i) cost of sewerage system, zonewise;

(ii) cost of pumping stations and pumping equipments;

(iii) cost of pumping mains;

(iv) cost of treatment works, unitwise;

(v) cost of power lines and telephone facilities to be laid;

(vi) cost of approach roads;

(vii) cost of land acquisition;

(viii) provision for special tools and plant and all ancillary items and equipments contingent on the proper execution of the project;

(ix) contingencies and unforeseen works; and

(x) centage charges.

2.1.2.2 *Recurring costs*

The estimated cost of the annual maintenance of the project should also be worked out separately to include the cost of necessary technical and non-technical staff for the operation and maintenance, energy charges for the running of pumpsets and other machinery, the cost of spares, consumable stores and replacements and the cost of chemicals envisaged in the treatment of the sewage as also the annuity on the capital loans based on the assumed period of repayment and the rate of interest. Contribution towards depreciation of the plant and machinery should also be indicated.

2.1.3 Plans

The following procedure is recommended for the nomenclature of sewers :

The trunk sewer should be selected first and drawn and other sewers should be considered as branches. The trunk sewer should be the one with the largest dia that would extend farthest from the outfall works. Whenever two sewers meet at a point, the main sewer is the larger of the incoming sewers. The manholes of the trunk sewer are designated as 0, 1, 2, 3, etc., commencing at the lower end (outfall end) of the line and finishing at the top end. Manhole on the mains or submains are again numbered 1, 2, 3, etc., prefixing the number of the manhole on trunk/main sewer where they join (e.g. 3.2 represents the second manhole on the main sewer from the manhole no. 3 on the trunk sewer). When all the sewer lines connected to the main line have thus been covered by giving distinctive numbers to the manholes, the manholes on the further branches to the branch mains are similarly given distinctive numbers, again commencing with the lower end. If there are two branches, one on each side meeting the main sewer or the branch sewer, letter 'L' (to represent left) or letter 'R' (to represent right) is again prefixed to the numbering system, reckoning against the direction of flow. If there is more than one sewer either from the left or right, they are suitably designated as L_1, L_2, R_1, R_2, the subscript referring to the line near to the sewer taking away the discharge from the manhole.

Thus L_2.R.4.2.3 (Figure 2.1) will pinpoint a particular manhole on the submain from which the flow reaches manhole number no. 4 on the trunk sewer through a submain and a main. The first numeral (from the left) is the number of the manhole on the trunk sewer. The numerals on the right of this numeral, in order, represent the manhole numbers in the main, submain etc. respectively. The first letter immediately preceding the numeral denotes the main and that it is to the right of the trunk sewer. Letters to the left in their order represent submain, branch respectively. The same nomenclature is used for representing the sections e.g. Section L_2.R.4.2.3 identifies the section between the manhole L_2.R. 4.2.3 and the adjoining downstream manhole.

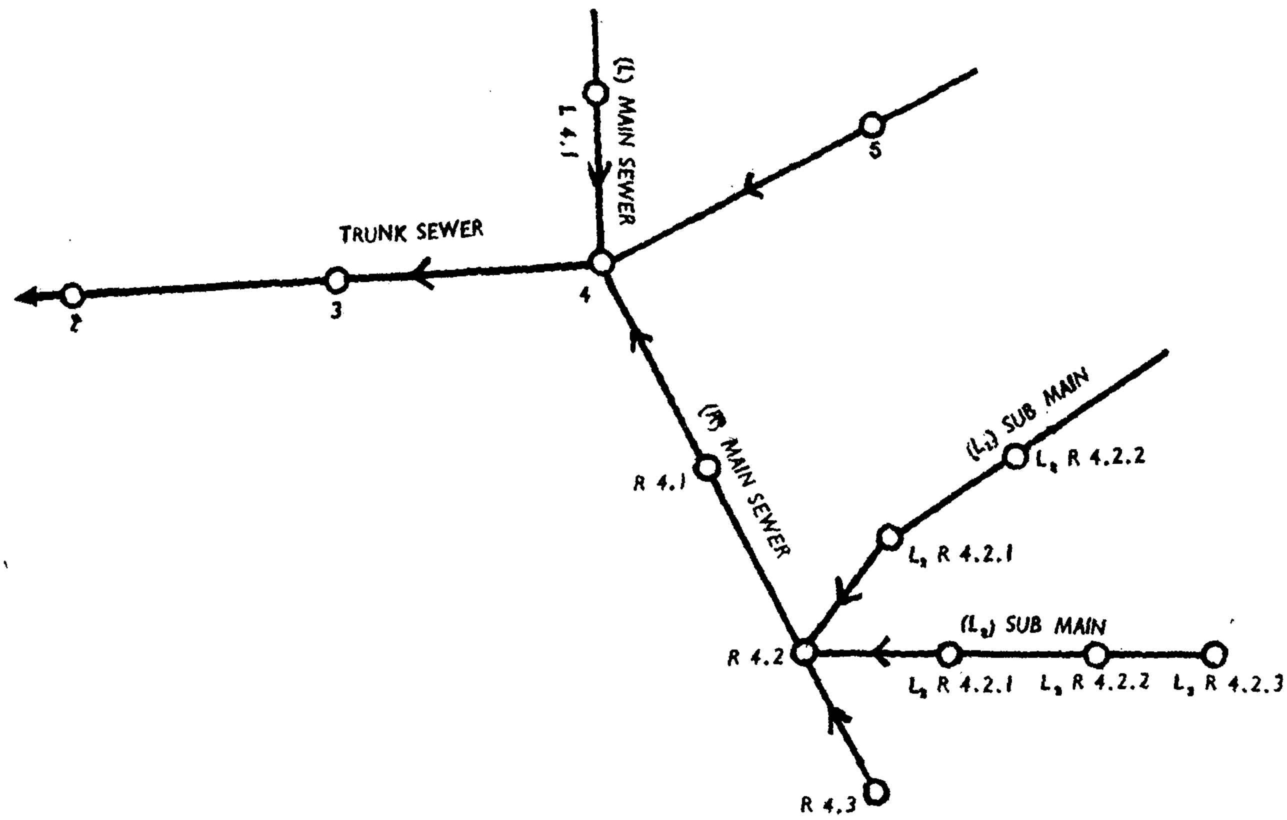


FIG 2.1 NOMENCLATURE OF SEWERS

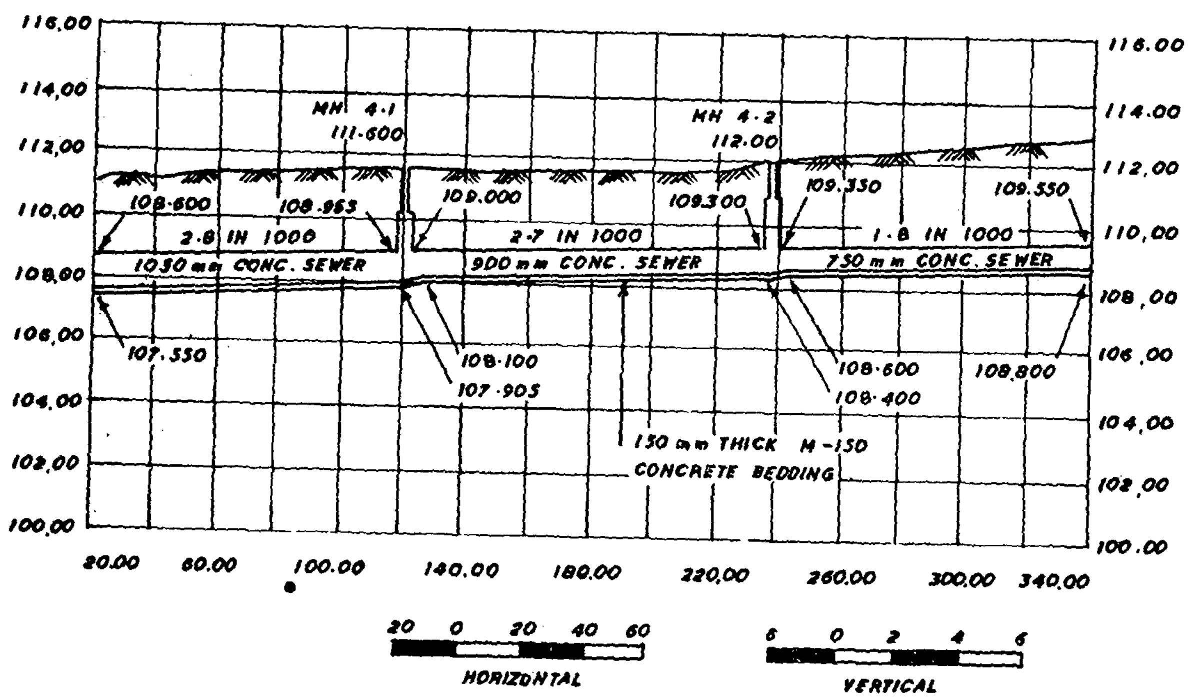


FIG. 2.2 A TYPICAL SEWER SECTION

All longitudinal sections should be indicated with reference to the same datum line. The vertical scale of the longitudinal sections should be magnified ten times the horizontal scale.

Once the rough sections have been prepared, the designer should go over the work for improving the spacing of manholes, the sizes and gradients of the sewers and so forth, economising on materials and excavation to the extent possible but at the same time making sure that the sewer will serve all users and that they can be actually laid according to the alignments shown in the drawing and have sufficient gradients. The sewers should have a minimum cover of 1 m at the starting point or otherwise adequately protected with cement concrete encasing.

The following scales may be adopted for the various plans and drawings:

(a) Index plan . . .	1:100,000 or 1:200,000
(b) Keyplan & general layout plan.	1:10,000 or 1:20,000
(c) Zonal plans . . .	1:2,500 or 1:5,000
(d) Longitudinal sections of sewers.	1:600 or 1:1,250 or 1:2,500
(e) Structural drawings . .	1:20 or 1:50 or 1:100 or 1:200.

The sewers should be shown as thick lines and manholes as small circles in plan. In section the sewer may be indicated by a line or two lines depending upon the diameters and scales adopted. Grade, size and material of pipe, ground and invert levels and extent of concrete protection should be indicated as shown in Figure 2.2.

Standard vertical plan filing systems are now available and are very convenient for storing of plans and taking them out quickly for reference. Normally, size A0 and AI (trimmed size 841 × 1189 mm and 594 × 841 mm respectively) should be used while submitting the project drawings for approval.

All documents including drawings, design calculations, measurement sheets of estimates, etc., should be in metric system. In drawings, length should be indicated either entirely in metres correct upto two decimals or entirely in millimetres (for thickness etc.). If this practice is followed, units would be obvious and in certain cases writing of m or mm with the figure can be omitted. The flow should normally be indicated in litres per second (lps) or cubic metres per hour (m^3/hr) except for very large flows which may be indicated in cubic metres per second (cumec). For uniformity, lps for sewage flows and cumec for storm flows is recommended. For all practical purposes one cubic metre may be taken as 1,000 litres. Similarly, areas in sewer plans and design calculations may be indicated in hectares (ha).

While writing figures they should be grouped into groups of three with a single space between each group and without comma. In case of a decimal number, this grouping may be on either side of the decimal (e.g. 47 342.294 31).

CHAPTER 3

DESIGN OF SEWERS

Sewerage systems can compose of a system of separate sanitary sewers and storm sewers, a system of combined sewers or a system consisting of sanitary sewers, part storm sewers and part combined sewers.

Combined sewerage system invariably suffers from the disadvantages of sluggish flow during most part of the year leading to deposition of sewage solids and creating foul and offensive conditions. In view of this, the combined system is normally not recommended in modern designs.

3.1 ESTIMATE OF SANITARY SEWAGE

Sanitary sewage is mostly the spent water of the community draining into the sewer system with some ground water and a fraction of the storm run-off from the area, draining into it. The sewers should be capable of receiving the maximum discharge expected at the end of design period. The provision, however, should not be much in excess of the actual discharge in the early years of its use to avoid deposition in sewers. The estimate of flow, therefore, requires a very careful consideration and is based upon the contributory population and the per capita flow of sewage, both the factors being guided by the design period.

3.1.1 Design Period

Since it is both difficult and uneconomical to augment the capacity of the system at a later date, sewers are usually designed for the maximum expected discharge to meet the requirements of the ultimate development of the area. Thus, the population estimate is guided by the anticipated ultimate growth rates which may differ in the different zones of the same town.

A design period of 30 years for all type of sewers is recommended.

3.1.2 Population Estimate

There are several methods used for forecasting the population of a community. The most suitable approach is to base the estimation either on anticipated ultimate density of population or on Floor Space Index.

In case the desired information on population is not available in the Master Plan of the town, the following densities are suggested for adoption :

Size of town (Population)	Density of population per hectare
Upto 5,000	75—150
5,000 to 20,000	150—250
20,000 to 50,000	250—300
50,000 to 1,00,000	300—350
Above 1,00,000	350—1000

In cities where Floor Space Index (FSI) or Floor Area Ratio (FAR) limits are fixed by the municipality, this approach may be used for working out the population density. FSI or FAR is the ratio of total floor area (of all the floors) to the plot area. The densities of population on this concept may be worked out as in the following example :

Assume that a particular Development Plan Rules provide for the following reservations for different land uses.

Roads	20%
Gardens	15%
Schools (including playgrounds)	5%
Markets	2%
Hospital and Dispensary	2%
	44%

Area available for Residential Development (100-44). $=56\%$

Actual total floor area $=$ Area for residential development $\times$ FSI.

Assuming an FSI of 0·5 and floor area of 9 m^2/person,

Number of persons or density per hectare. $= \dfrac{0{\cdot}56 \times 10000 \times 0{\cdot}5}{9} = 311$

3.1.3 Area

The tributary area for any section under consideration needs to be marked on a key plan. The topography, layout of buildings, legal limitations, etc., determine the tributary area draining to a sewer section. The area is to be measured from the map.

3.1.4 Per Capita Sewage Flow

Although the entire spent water of a community should contribute to the total flow in a sanitary sewer, it has been observed that a small portion is lost in evaporation, seepage in ground, leakage, etc. In some arid areas, the fraction reaching the sewers may be as low as 40% while for an intensely developed area, it may be as high as 90%. Generally, 80% of the water supply may be expected to reach the sewers unless there is data available to the contrary.

The sewers should be designed for a minimum of 150 litres per capita per day.

Industries and commercial buildings often use water other than from the municipal supply and more often discharge their liquid wastes into the sanitary sewers. In such cases, deviations from the general values may occur and estimates of such flows have to be made separately. It is, however, desirable that when the industrial waste is fairly large, it is segregated and disposed of in a suitable manner or treated suitably before discharge into sewers.

3.1.5 Storm Runoff

Sanitary sewers are not expected to receive storm water. Strict inspection and vigilance and proper design and construction of sewers and manholes should eliminate this flow or bring it down to a very insignificant quantity.

3.1.6 Ground Water Infiltration

Estimate of flow in sanitary sewers may include certain flows due to infiltration of ground water through joints. The quantity will depend on workmanship in laying of sewers and height of the ground water table. Since sewers are designed for peak discharges, allowance for ground water infiltration for the worst condition in the area should be made. Suggested estimates for ground water infiltration for sewers laid below ground water table are as follows :

	Minimum	*Maximum*
lpd/hectare	5,000	50,000
lpd/km of sewer/cm dia . .	500	5,000
lpd/manhole	250	500

With improved standards of workmanship and quality and availability of various construction aids, these values should tend to the minimum rather than the maximum. These values should not mean any relaxation on the water tightness test requirements in 7.1.5.

3.2 ESTIMATE OF STORM RUNOFF

Storm runoff is that portion of the precipitation which drains over the ground surface. Estimation of such runoff reaching the storm sewers therefore is dependent on intensity and duration of precipitation, characteristics of the tributary area and the time required for such flow to reach the sewer. The storm water flow for this purpose may be determined by using the rational method, hydrograph method, rainfall-runoff correlation studies, digital computer models, inlet method or empirical formulae.

The empirical formulae that are available for estimating the storm water runoff can be used only when comparable conditions to those for which the equations are derived initially can be assured.

A rational approach, therefore, demands a study of the existing precipitation data of the area concerned to permit a suitable forecast. Storm sewers are not designed for the peak flow of rare occurence such as once in 100 years or more but it is necessary to provide sufficient capacity to avoid too frequent a flooding of the drainage area. There may be some flooding when the precipitation exceeds the design value which has to be permitted. The frequency of such permissible flooding may vary from place to place, depending on the importance of the area. Flooding at any time, however, causes inconvenience to the citizens but they may accept it once in a while considering the savings affected in storm drainage costs.

The maximum runoff which has to be carried in a sewer section should be computed for a condition when the entire basin draining at that point becomes contributory to the flow and the time needed for this is known as the time of concentration (tc) with reference to the concerned section. Thus for estimating the flow to be carried in the storm sewer, the intensity of rainfall which lasts for the period of time of concentration is the one to be considered contributing to the flow of storm water in the sewer. Of the different methods, the rational method is more commonly used.

3.2.1 Rational Method

3.2.1.1 *Runoff-rainfall intensity relationship*

The entire precipitation over the drainage district does not reach the sewer. The characteristics of the drainage district such as imperviousness, topography including depressions and water pockets, shape of the drainage basin and duration of the precipitation determine the fraction of the total precipitation which will reach the sewer. This fraction known as the coefficient of runoff needs to be determined for each drainage district. The runoff reaching the sewer is given by the expression,

$$Q = 10\, CiA \qquad (3\text{—}1)$$

where Q is the runoff in m^3/hr;

'C' is the coefficient of runoff;

'i' is the intensity of rainfall in mm/hr; and

'A' is the area of drainage district in hectares.

3.2.1.2 *Storm frequency*

The frequency of storm for which the sewers are to be designed depends on the importance of the drainage area. Commercial and industrial areas should be subject to less frequent flooding than the residential areas. In view of the present economic conditions, the suggested frequency of flooding in the different aeas is as follows :

(a) Residential areas—

- (i) Peripheral areas . . . twice a year;
- (ii) Central and comparatively high priced areas. . . . once a year.

(b) Commercial and high priced areas. once in 2 years.

3.2.1.3 *Intensity of precipitation*

The intensity of rainfall decreases with duration. Analysis of past records over a period of years of the observed data on intensity-duration of rainfall in the area is necessary to arrive at a fair estimate of the intensity-duration for given frequencies. The longer the record available, the more dependable is the forecast.

Table 3.1 gives the analysis of the frequency of storms of stated intensities and durations during 26 years for which rainfall data were available for a given town.

Table 3.1

Duration in Minutes	No. of storms of stated intensity or more for a period of 26 years								
	30	35	40	45	50	60	75	100	125 mm/hr
5					100	40	18	10	2
10			90	72	41	25	10	5	1
15		82	75	45	20	12	5	1	
20	83	62	51	31	10	9	4	2	
30	73	40	22	10	8	4	2		
40	34	16	8	4	2	1			
50	14	8	4	3	1				
60	8	4	2	1					
90	4	2							

The stepped line indicates the location of the storm occurring once in 2 years, i.e. 13 times in 26 years. The time-intensity values for this frequency are obtained by interpolation and given in Table 3.2 :

Table 3.2

i (mm/hr)	t (min)
30	51·67
35	43·75
40	36·48
45	28·57
50	18·50
60	14·62
75	8·12

The relationship may be expressed by a suitable mathematical formula, several forms of which are available. The following two equations are commonly used :

$$\text{(i)}\ i = \frac{a}{[t^n)} \quad \ldots \quad \ldots \quad (3\text{—}2)$$

$$\text{(ii)}\ i = \frac{a}{t+b} \quad \ldots \quad \ldots \quad (3\text{—}3)$$

Where

i=intensity of rainfall (mm/hr);

t=duration of storm (minutes); and

a, b & n are constants.

The available data on i and t are plotted on arithmetic paper (the second equation permits a straight line plot with the reciprocal of 'i' plotted against 't'). The values of the intensity, (i), can then be determined for any given time of concentration, (t_c).

3.2.1.4 *Time of concentration*

It is the time required for the rain water to flow over the ground surface from the extreme point of the drainage basin and reach the point under consideration. Time of concentration (t_c) is equal to inlet time (t_i) plus the time of flow in the sewer (t_f). The inlet time is dependent on the distance of the farthest point in the drainage basin to the inlet manhole, the shape, characteristics and topography of the basin and may generally vary from 5 to 30 minutes. In highly developed sections, the inlet time may be as low as 3 minutes. The time of flow is determined by the length of the sewer and the velocity of flow in the sewer. It is to be computed for each length of sewer as it is designed.

3.2.1.5 *Coefficient of runoff*

The portion of rainfall which finds its way to the sewer is dependent on the imperviousness and the shape of tributary area apart from the duration of storm.

(a) IMPERVIOUSNESS

The percent imperviousness of the drainage area can be obtained from the records of a particular district. In the absence of such data, the following may serve as a guide :

Type of area	Percentage of imperviousness
Commercial and Industrial areas	70 to 90
Residential Area :	
(i) High density	60 to 75
(ii) Low density	35 to 60
Parks & undeveloped areas	10 to 20

The weighted average imperviousness of drainage basin for the flow concentrating at a point may be estimated as

$$I = \frac{A_1.I_1 + A_2.I_2 + \ \ldots\ldots}{A_7 + A_2 + \ \ldots\ldots}$$

where,

A_1, A_2,=drainage areas tributary to the section under consideration;

I_1, I_2, =imperviousness of the respective areas; and

I =weighted average imperviousness of the total drainage basin

(b) TRIBUTARY AREA

For each length of storm sewer, the drainage area should be indicated clearly on the map and measured. The boundaries of each tributary are dependent on topography, land use, nature of development and shape of the drainage basins. The incremental area may be indicated separately on the compilation sheet and the total area computed.

(c) DURATION OF STORM

Continuously long light rain saturates the soil and produces higher coefficient than that due to heavy but intermittent rain in the same area because of the lesser saturation in the latter case. Runoff from an area is significantly influenced by the saturation of the surface nearest the point of concentration rather than the flow from the distant area. The runoff coefficient of a larger area has to be adjusted by dividing the area into zones of concentration and by suitably decreasing the coefficient with the distance of the zones.

(d) COMPUTATION OF RUNOFF COEFFICIENT

The weighted average runoff coefficients for rectangular areas of length four times the width as well as for sector shaped areas with varying percentages of impervious surface for different times of concentration are given in Table 3.3. Although these are applicable to particular shapes of area, they also apply in a general way to the areas which are usually encountered in practice. Errors due to difference in shape of drainage are within the limits of accuracy of the rational method and of the assumptions on which it is based.

A typical example of the computation of storm runoff is given in Appendix-4.

3.3 SEWER DESIGN

Sewers while carrying the waste water discharge for which they are designed have also to transport suspended solids in such a manner that deposition and odour nuisance therefrom are kept to a minimum. Sewers are almost exclusively designed for flows with free water surface and self cleansing velocities. Pressure sewers, including siphons, should be avoided as far as practicable.

3.3.1 Flow Assumptions

The flow in sewers varies considerably from hour to hour and also seasonally but for purposes of hydraulic design it is the estimated peak flow that is adopted.

Table 3.3

Runoff coefficients (After Horner)

Duration, t, minutes	10	20	30	45	60	75	90	100	120	135	150	180
Weighted average Coefficients												
(1) Sector concentrating in stated time												
(a) Impervious	·525	·588	·642	·700	·740	·771	·795	·813	·828	·840	·850	·865
(b) 60% Impervious	·365	·427	·477	531	·569	·598	·622	·641	·656	·670	·682	·701
(c) 40% Impervious	·285	·346	·395	·446	·482	·512	·535	·554	·571	·585	·597	·618
(d) Pervious	·125	·185	·230	·277	·312	·330	·362	·382	·399	·414	·429	·454
(2) Rectangle (length=4×width) concentrating in stated time												
(a) Impervious	·550	·648	·711	·768	·808	·837	·856	·869	·879	·887	·892	·903
(b) 50% Impervious	·350	·442	·499	·551	·590	·618	·639	·657	·671	·683	·694	·713
(c) 30% Impervious	·269	·360	·414	·464	·502	·530	·552	·572	·588	·601	·614	·636
(d) Pervious	·149	·236	·287	·334	·371	·398	·422	·445	·463	·479	·495	·522

The peak factor or the ratio of maximum to average flows, depends upon the contributory population and the following values are recommended :

Contributory Population	*Peak Factor*
Upto 20,000	3·5
20,000 to 50,000	2·5
50,000 to 7,50,000	2·25
Above 7,50,000	2·0

3.3.2 Self-Cleansing Velocity

It is necessary to maintain a minimum velocity or 'self-cleansing velocity' in a sewer to ensure that suspended solids do not deposit and cause nuisance. Self-cleansing velocity is determined by considering the particle size and the specific weight of the suspended solids in sewage (refer to 11.2.5.4. Grit Removal). A minimum velocity of 0.8mps at design peak flow in the sanitary sewers is recommended subject to a minimum velocity of 0.6 mps for present peak flows as discussed in 3.3.3.

3.3.3 Velocity at Minimum Flow

To avoid steeper gradients which will require deeper excavations, it has been the practice to design sewers for the self-cleansing velocity at ultimate peak flows. This is done on the assumption that although silting might occur at minimum flow, the silt would be flushed out during the peak flows. However, the problem of silting may have to be faced in the early years, particularly for smaller sewers which are designed to flow half-full, as the actual depth of flow then is only a small fraction of the full depth. Similarly upper reaches of laterals pose a problem as they flow only partially full even at the ultimate design flow, because of the necessity for adopting the prescribed minimum size of sewer.

It has been shown that for sewers running partially full, for a given flow and slope, velocity is little influenced by pipe diameter. It is, therefore, recommended that for present peak flows upto 30 lps, the slopes given in Table 3.4 may be adopted, which would ensure a minimum velocity of 0.60 mps in the early years.

Table 3.4

Present peak flow in lps	*Slopes per 1,000*
2	6·0
3	4·0
5	3·1
10	2·0
15	1·3
20	1·2
30	1·0

After arriving at slopes for present peak flows, the pipe size should be decided on the basis of ultimate design peak flow and the permissible depth of flow.

3.3.4 Erosion and Maximum Velocity

Erosion of sewers is caused by sand and other gritty material in the sewer and also by excessive velocity. Velocity in a sewer is recommended not to exceed 3.0. mps.

3.3.5 Minimum Size

Minimum diameter for a public sewer shall be 150mm. However, recommended practice is to provide 200mm minimum size. Minimum size for hilly areas where extreme slopes are prevalent may be 100 mm.

3.3.6 Hydraulic Formulae

For open channel flow, Kutter's or Manning's formula may be used for designing slope and diameter of a sewer line to carry the design flow at a stated velocity. Manning's formula which is simpler and used more commonly is—

$$V = \frac{1}{n} \; r^{\frac{2}{3}} \cdot s^{\frac{1}{2}}$$

For circular conduits—

$$v = 3.968 \times 10^{-3} \times d^{\frac{2}{3}} \times \frac{1}{n} \times s^{\frac{1}{2}}$$

and $Q = 3.118 \times 10^{-6} \times d^{8/3} \times \dfrac{1}{n} \times s^{\frac{1}{2}}$

where
Q=discharge in lps
s=slope of hydraulic gradient
d=dia of pipe in mm
r=hydraulic radius in m
v=velocity in mps
and n=Manning's coefficient of roughness.

A Chart for Manning's formula is given in Appendix 5.

The values of Manning's coefficient 'n' for different materials are given in Table 3.5. Usually, the values corresponding to fair condition of the interior surface are used in design.

Table 3.5

Manning's Coefficients

Conduit Material	Condition of Interior Surface	
	Good	Fair
Salt glazed stoneware . .	0·012	0·014
Cement Concrete . . .	0·013	0·015
Cast Iron	0·012	0·013
Brick, unglazed . . .	0·013	0·015
Asbestos Cement . . .	0·011	0·012
Plastic (Smooth) . . .	0·011	0·011

A reduction in the value of n has been reported with increase in dia. For cement concrete pipes of dia 600mm and above, 0.013 may be used.

3.3.7 Depth of Flow

From considerations of ventilation in waste water flow, sewers should not be designed to run full; upto 400mm dia sewers may be designed to run at half depth; 400 to 900 mm at two-thirds depth; and larger sewers at three-fourths depth at ultimate peak flows. The chart for Manning's formula in the Appendix 5 gives the discharges and the velocities when sewers are running full. These figures require to be modified for partial flow conditions. The relation between flow ratio and velocity ratio with the depth ratio is given in Fig. 3.1 and Table 3.6.

Table 3.6

Hydraulic Properties of Circular Sections

	Constant n		Variable n		
d/D	v/V	q/Q	nd/n	v/V	q/Q
1·0	1·000	1·000	1·00	1·000	1·000
0·9	1·124	1·066	1·07	1·056	1·020
0·8	1·140	0·988	1·14	1·003	0·890
0·7	1·120	0·838	1·18	0·952	0·712
0·6	1·072	0·671	1·21	0·890	0·557
0·5	1·000	0·500	1·24	0·810	0·405
0·4	0·902	0·337	1·27	0·713	0·266
0·3	0·776	0·196	1·28	0·605	0·153
0·2	0·615	0·088	1·27	0·486	0·070
0·1	0·401	0·021	1·22	0·329	0·017

where,
D=Full depth of flow (Internal dia)
d=Actual depth of flow
V=Velocity at full depth
v=Velocity at depth 'd'
Q=Discharge at full depth
q=Discharge at depth 'd'
n=Manning's coefficient at full depth
n_d=Manning's coefficient at depth d

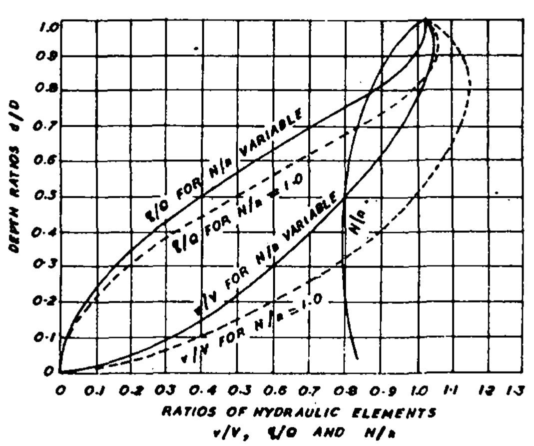


FIG. 3·1 BASIC HYDRAULIC ELEMENTS OF CIRCULAR SEWERS FOR ALL VALUES OF ROUGHNESS AND SLOPE.

3.3.8 Design Approach

For the present peak discharge and velocity of 0.6 mps, the slope and dia are chosen. For this dia and velocity of 0.8 mps, the discharge at full depth is found. Depending upon the dia, the depth ratio is fixed and the corresponding flow and velocity ratios are found: Q is then determined and a check is applied to verify whether this exceeds the actual ultimate peak discharge. Otherwise, the next higher dia or steeper slope is selected and adjusted to satisfy the velocity and flow requirements.

Appendix 6 gives a worked out example.

3.3.9 Sewer Transitions

Sewer transitions include change in size slope, alignment, volume of flow, free and submerged discharge at the end of sewer lines, passage through measuring and diversion devices and sewer junctions. Allowance for the headloss that occurs at these transitions has to be made in the design.

Manholes should be located at all such transitions and a drop should be provided where the sewer is intercepted at a higher elevation for streamlining the flow, taking care of the headloss and also to help in maintenance. The vertical drop may be provided only when the difference between the elevations is more than 60 cm, below which it can be avoided by adjusting the slope in the channel in the manhole connecting the two inverts.

The following invert drops are recommended:

(a) For sewers less than 400 mm. ...... Half the difference in dia.

(b) 400 mm to 900 mm...... 2/3 the difference in dia.

(c) Above 900 mm ...... 4/5 the difference in dia.

Transition from larger to smaller diameters should not be made. The crowns of sewers are always kept continuous. In no case, the hydraulic flowline in the larger sewers should be higher than the incoming one.

3.3.10 Backwater Curves

Backwater or drawdown curves resulting from abrupt changes in sewer slopes or when there is a free fall or an obstruction to the flow may be calculated from the following formula:—

$$L = \frac{(d+h_y)}{s_e - s_o}$$

where d and h_y are the changes in the water depth and velocity in a length L; Se and So being the slopes of energy grade line and the invert respectively.

The computations are started from a point where depth and velocity of flow are known and L is worked out for different depths of flow upto the normal depth.

3.3.11 Force Mains

Sewage may have to be carried to higher elevations through force mains. The size of the main should be determined by taking into account the initial cost of pipeline and cost of operation of pumping for different sizes. Velocities may range from 0.8 to 3 mps. Hazen & Williams formula is generally used for computing the frictional losses which is expressed as:

$$v = 0.849\ c r^{0.63} s^{0.54}$$

where, v is velocity in mps;

r is hydraulic radius in m;

s is slope of hydraulic gradient; and

c is Hazen and Williams coefficient for the material of the pipe.

The following values of c may be adopted for design purposes :

(i) Cast Iron	100
(ii) Steel	100
(iii) Asbestos Cement	120
(iv) Cement Concrete	110
(v) Plastic (smooth)	120

Losses in valves, fittings, etc., are dependent upon the velocity head $v^2/2g$. Loss in bends and elbows depend upon the ratio of absolute friction factor to dia of pipe, besides velocity head. Loss due to sudden enlargement depends upon the ratio of diameters. The losses in bends, enlargements and tapers are given in 5.2.3 of the companion volume, Manual on water supply and treatment (second Edition). In the actual design of the force mains, it may not be necessary to compute the losses individually but the same may be assumed arbitrarily as 5 to 15% of the total frictional losses depending upon the number of bends, tapers and other fittings. However, for shorter mains with a large number of bends etc., the actual loss may be computed and expressed as equivalent lengths of pipes.

3.3.12 Inverted Siphon

When a sewer line dips below the hydraulic grade line, it is called an inverted siphon. The purpose is to carry the sewer under the obstruction and regain as much elevation as possible after the obstruction is passed. They should be resorted to only where other means of passing the obstruction are not feasible as they require considerable attention in maintenance. As the siphons are depressed below the hydraulic grade line, maintenance of self-cleansing velocity at all flows is very important. It is necessary to ascertain the minimum flows and the peak flows for design. To ensure self-cleansing velocities for the wide variations in flows, generally, two or more pipes not less than 200 mm dia are provided in parallel so that upto the average flows, one pipe is used and when the flow exceeds the average, the balance flow is taken by the second and subsequent pipes. The design criteria for inverted siphons are given in IS:4111 Part III. Some of the important criteria are given below.

3.3.12.1 *Hydraulic calculations*

As the inverted siphon is a pipe under pressure, the difference in the water levels at the inlet and outlet is the head under which the siphon operates. This head should be sufficient to cover the entry, exit and friction losses in pipes.

3.3.12.2 *Velocity*

It is necessary to have a self-cleansing velocity of 1.0 mps for the minimum flow to avoid deposition in the line.

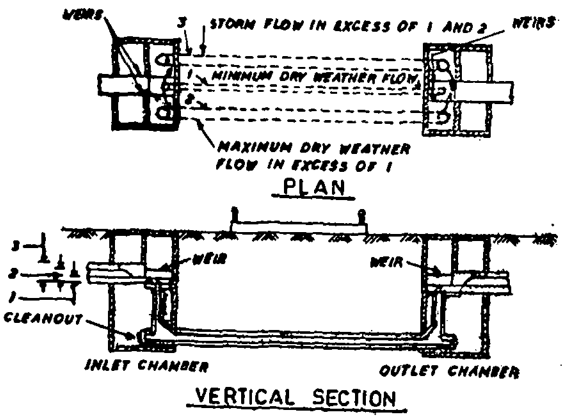


FIG. 3.2 INVERTED SIPHON OR SUPPRESSED SEWER FOR COMBINED SEWAGE.

3.3.12.3 *Size and arrangement of pipes*

In the multiple pipe siphon, the inlet should be such that the pipes come into action successively as the flow increases. This may be achieved by providing lateral weirs with heights kept in accordance with the depth of flow at which one or more siphon pipes function. Fig. 3.2 gives the general arrangement for a three-way siphon. In the two-pipe siphon, the first pipe should take 1.25 to 1.5 times the average flow and second should take the balance of the flow.

3.3.12.4 *Inlet and outlet chambers*

The design of inlet and outlet chambers should allow sufficient room for entry for cleaning and maintenance of siphons. The outlet chambers should be so designed as to prevent the backflow of sewage into pipes which are not being used at the time of minimum flow.

3.3.12.5 *General requirements*

Provision should be made for isolating the individual pipes as well as the siphon to facilitate cleaning. This can be done by providing suitable penstocks or stopboards at the inlet and outlet of each pipe and by providing stopvalve at its lower point if it is accessible. A manhole at each end of the siphon should be provided with clearance for rodding. The rise, out of the siphon for small pipes should be on a moderate slope so that sand and other deposits may be moved out of the siphon. It is desirable to provide a coarse screen to prevent the entry of rags etc., into the siphon.

Proper bypass arrangements should be provided from the inlet chamber to a nearby stream if permitted by the pollution control authority; otherwise special arrangements should be made for pumping the sewage to the lower reach of the sewer line.

CHAPTER 4

SEWER APPURTENANCES

Sewer appurtenances are devices necessary, in addition to pipes and conduits, for the proper functioning of any complete system of sanitary, storm or combined sewers. They include structures and devices such as various types of manholes, lamp-holes, gully traps, intercepting chambers, flushing tanks, ventilation shafts, catch-basins, street inlets, regulators, siphons, grease traps, side flow weirs, leaping weirs, venturi-flumes and outfall structures.

4.1 MANHOLES

4.1.1 Ordinary Manholes

A manhole is an opening constructed on the alignment of a sewer for facilitating a person access to the sewer for the purpose of inspection, testing, cleaning and removal of obstructions from the sewer line.

4.1.1.1 *Spacing*

Manholes are generally provided on straight reaches at convenient spacings which depend on the size of the sewers. The larger the diameter of the sewer, the greater may be the spacing between two manholes. The spacing between the mahnoles will also depend upon the nature of sewer cleaning devices in use. The straight runs between manholes are limited in length to 30m for sewers upto 300 mm in dia where manual rodding is adopted. For large sewers, they may go up to 100 m or more. These limits can be considerably relaxed for sewers sufficiently large permitting entry for inspection, cleaning or repair with access manholes placed quite far apart either symmetrically above the sewer or tangentially to one side. Apart from these manholes on straight reaches, manholes shall also be provided at the start of a sewer, at all junctions, at all points of change of alignment and at all points of change of gradient. When twin or multiple box sections are used, separate manholes for each conduit shall be provided.

4.1.1.2 *Shape and size*

Manholes are generally circular, square or rectangular in shape. The inside dimension should be adequate to permit inspection and cleaning operations without difficulty. A minimum inside dimension of 120cm × 90 cm for manholes is usually recommended except for shallow manholes upto depths of 1.35 m, where minimum allowable width may be reduced from 90cm to 75cm.

4.1.1.3 *Construction details*

Manholes are usually constructed directly over the centre line of the sewer. For larger sewers the manhole is preferably constructed at a tangent to the side of the sewer for better accessibility. The manholes for very large sewers may be located over the centre line of the sewer with a suitable landing platform offsetting from an opening in the sewer itself.

The opening for entry into the manhole should be of such minimum dimensions as to allow a workman with the cleaning equipments to get access into the interior of the manhole without difficulty. A circular opening is generally preferred. A minimum clear opening of 50 cm is recommended. Suitable steps usually of malleable cast iron shall be provided for entry.

A slab, generally of plain cement concrete at least 150 mm thick should be provided at the base to support the walls of the manhole and to prevent the entry of ground water. The thickness of the base slab shall be suitably increased upto 300 mm, for manholes on large dia sewers, with adequate reinforcement provided to withstand excessive uplift pressures, In the case of larger manholes, the flow in the sewer should be carried in U-shaped smooth channel constructed integrally with the concrete base of the manhole. The side of the channel should be equal to the dia of the largest sewer pipe. The adjacent floor should have a slope of 1 in 10 draining to the channel. Where more than one sewer enters the manhole the flow through channel should be curved smoothly and should have sufficient capacity to carry the maximum flow.

It is desirable to place the first pipe joint outside the manhole as close as practicable. The pipe shall be built inside the wall of the manhole flush with the internal periphery protected with an arch of masonry or cement concrete to prevent it from being crushed.

The sidewalls of the manhole are usually constructed of cement brickwork 250 mm thick and corbelled suitably to accommodate the frame of the manhole cover. The inside and outside of the brickwork shall be plastered 20 mm thick with 1:2 cement mortar.

4.1.1.4 *Cover and frame*

The cover and frame may be of C. I. or reinforced concrete. IS:1726 should be adhered to when these are of C. I. The manhole frames shall be 53 cm and not be less than 160 mm thick and be set conforming accurately to the grade of the pavement. The frame shall rest on concrete band and be set in appropriate concrete mix so that the space between the top of the manhole masonry and the bottom flange of the frame shall be completely filled and made watertight. A thick ring of mortar extending to the outer edge of the masonry shall be placed all around the bottom flange. Heavy reinforced concrete covers with suitable lifting arrangements could be used instead of C. I. manhole covers.

4.1.2 **Special Manholes**

4.1.2.1 *Junction chambers*

Where two sewers particularly of large dia intersect, the intersection is made by means of a brick or concrete structure known as a junction chamber. The junction chamber provides access to the sewer and also allows the flow from the sewers entering the chamber to be combined without excessive turbulence and loss of head.

The principal objective in the design of a junction chamber is to provide a safe and economical structure which will combine the flow smoothly without decreasing the velocities appreciably and without causing backwater conditions in the sewers entering the chamber.

4.1.2.2 *Drop Manholes*

These are provided when the difference in elevation of the invert levels of the incoming and outgoing sewers of a manhole is more than 60 cm. Details are discussed in 3.3.9 and 7.1.8.

4.1.2.3 *Flushing manholes*

Flushing manholes are located generally at the head of a sewer. The sewers are flushed once or twice a day. Sufficient velocity shall be imparted in the sewer to wash away the deposited solids. The flush is usually effective upto a distance of about 300 m after which the imparted velocity gets dissipated.

Flushing operation should preferably be automatic. In case of hard chokages in the sewers, care should be exercised to ensure that there is no possibility of backflow of sewage into the water supply mains.

The automatic systems which are operated by mechanical units get often corroded by the sewer gases and do not generally function satisfactorily and hence are not recommended.

Approximate quantities of water needed for flushing are as follows :

Slope	Quantity of water (litres) 200 mm dia	250 mm dia	300 mm dia
0·0050	2300	2500	3000
0·0075	1500	1800	2300
0·0100	1300	1500	2000
0·0200	500	800	1000
0·0300	400	500	700

4.2 INVERTED SIPHONS

Inverted siphons although grouped as a separate head of appurtenances form a type of diversion structure meant for the purpose of bifurcation of the flow in stipulated proportions. The details are discussed in 3.3.12.

4.3 HOUSE SEWER CONNECTIONS

House sewer connections should preferably be 150 mm or more in dia with a minimum slope of 0.025 laid, as far as possible, to a straight line and grade. Connections to the main street sewer should normally be made with Y branches. For sewers deeper than 5 m, tees are preferable to facilitate connections at higher elevations, particularly where simultaneous discharge of house sewers into the street sewer is not expected and also prevent damage while rodding.

The Y or tee may be installed with the branch turned about 45° from the horizontal so that back-flooding of the house connection will not occur when the collecting sewers flow full. Connections to large sewers are for the same reason made above the spring line of the main sewer. The house connection for deep sewers, where made by means of a vertical pipe riser, shall be encased in concrete at least 75 mm thick and upto the full length of the pipe to prevent damage during backfilling.

All possible practical provision should be made for future connections in the original construction. Connections to existing sewers, particularly those of small diameter, should wherever possible be made with these tees or Ys. The free end of the service lines or branches should be closed with a carefully fitted stopper, when service lines are not yet connected to buildings or where intermediate connections are not yet made with the tee or the Y branches.

The recent practice is to make the house connections directly without providing intercepting traps. The deletion of the intercepting traps at the sewer connections provides effective ventilation of the sewer system without the use of ventilators. The intercepting traps do not serve any useful purpose and are more a nuisance. The direct connection is therefore recommended.

4.4. STORM WATER INLETS

These are devices meant to admit the surface runoff to the sewers and form a very important part of the system. Their location and design should therefore be given careful consideration.

Storm water inlets may be categorised under three major groups viz., curb inlets, gutter inlets and combination inlets, each being either depressed or flush depending upon their elevation with reference to the pavement surface.

The actual structure of an inlet is usually made of brickwork. Normally, cast iron gratings conforming to IS: 961 shall be used. In case there is no vehicular traffic, fabricated steel gratings may be used. The clear opening shall not be more than 25 mm. The connecting pipe from the street inlet to the main street sewer should not be less than 200 mm in dia and should have sufficient slope.

Maximum spacing of inlets would depend upon various conditions of road surface, size, and type of inlet and rainfall. A maximum spacing of 30 m is recommended.

4.4.1 **Curb Inlets**

Curb inlets are vertical openings in the road curbs through which the storm water flows and are preferred where heavy traffic is anticipated. They are termed as deflector inlets when equipped with diagonal notches cast into the gutter along the curb opening to form a series of ridges or deflectors. This type of inlet does not interfere with the flow of traffic as the top level of these deflectors lie in the plane of the pavement.

4.4.2 **Gutter Inlets**

These consist of horizontal openings in the gutter which is covered by one or more gratings through which the flow passes.

4.4.3 **Combination Inlets**

These are composed of a curb and gutter inlet acting as a single unit. Normally, the gutter inlet is placed right in front of the curb inlet but it may be displaced in an overlapping or end-to-end position. Figure 4 shows different types of inlets.

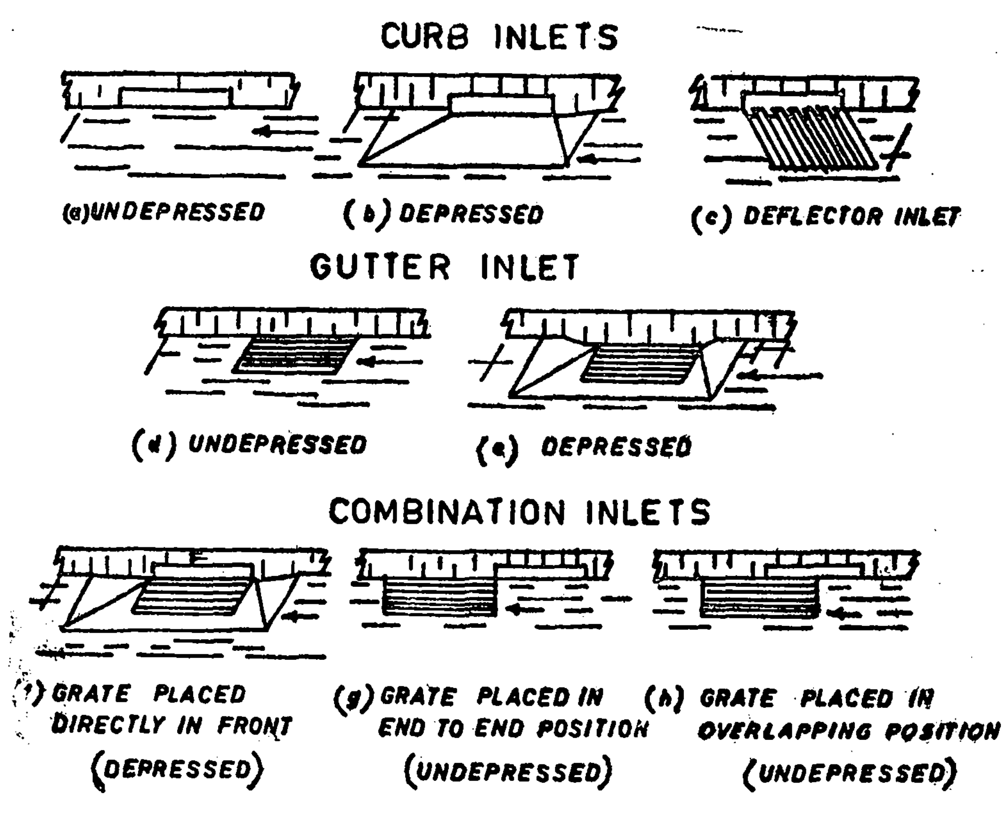


FIGURE: 4.1 INLETS

4.5 CATCH BASINS

Catch basins are structures meant for the retention of heavy debris in storm water which otherwise would be carried into the sewer system. Their use is not recommended since they are more of a nuisance and a source of mosquito breeding apart from posing substantial maintenance problems.

4.6 REGULATORS OR OVERFLOW DEVICES

These are used for preventing overloading of sewers, pumping stations, treatment plants or other disposal arrangements, by diverting the excess flows to relief sewers etc.

The overflow devices may be sideflow or *leaping weirs according to the position of the* weir, siphon spillways, or float actuated gates and valves.

4.6.1 Sideflow Weir

A sideflow weir constructed along one or both sides of a combined sewer delivers excess flows during storm periods to relief sewers or natural drainage courses. The crest of the weir is set at an elevation corresponding to the desired depth of flow in the sewer. The weir length must be sufficiently long for effective regulation.

The length of the sideflow weir is given by the formula devised by Babbitt.

$$L=7.6\times10^{-3} \ V \ D \log \frac{h_1}{h_2}$$

where L is the required length in m;

V is the velocity of approach in mps;
D is the dia of the sewer in mm and;
h_1 and h_2 are the heads above the crest of the weir upstream and downstream.

The formula is limited to conditions in which the weir is placed in the side of a circular pipe at a distance above the bottom greater than $\frac{d}{4}$ and less than $\frac{d}{2}$ where 'd' is the diameter of the pipe, and the edge of the weir is sharp and parallel to the invert of the channel. Its usefulness is limited in that it was devised for pipes between 450 and 600 mm in dia and in that the depth of flow above the weir should not exceed 3d/4.

4.6.2 **Leaping Weir**

This is designed to take the sanitary sewage through an opening in the invert of the combined sewer to an intercepting sewer. Depending upon the design, all or part of storm water leaps across the opening and is thus diverted from the intercepting sewer.

Some formulae, based on empirical findings are available for design of leaping weirs. However from practical considerations it is desirable to design the weirs with moving crests to make the opening adjustable as indicated in Figure 4.2.

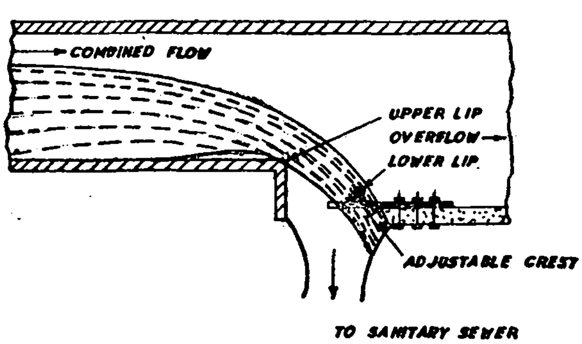


FIG. 4.2 LEAPING WEIR

4.6.3 **Float Actuated Gates and Valves**

Control of the flow in sewers can also be regulated by means of automatic mechanical regulators. These are actuated by the water level in the sump interconnected to the sewers. These regulators involve moving parts which are actuated by the varying depths of flow in the sewers. They require periodic inspection and maintenance.

4.7 FLAP GATES AND FLOOD GATES

Flap gates or backwater gates are installed at or near sewer outlets to prevent backflow of water during high tide or at high stages in the receiving stream. Such gates should be designed so that the flap should open at a very small head differential. With a properly operated flap gate it is possible to continue to pump, a quantity equivalent to the sanitary sewage flow from the combined sewer to the treatment plant even though flood conditions prevail in the stream at the sewer outlet.

In case of sea and estuary outfalls, the outfall sewer should be able to discharge at full rate *when the water level in the estuary or sea is* 3/4ths the mean annual tide level. Adequate storage to prevent backflow into the system due to the closure of these gates at the time of high tides is also necessary if pumping is to be avoided. To control the flow from the storage tank, flood gates or penstocks, which can be opened and closed quickly at the predetermined states of tide, are provided. The gates are generally electrically operated and are controlled by a lunar clock.

Many flap or backwater gates are rectangular and may consist of wooden planks. Circular or rectangular metallic gates are commercially available. Flap gates may be of various metals or alloys as required by the design conditions.

Flap gates are usually hinged by a link-type arrangement that makes it possible for the gate shutter to get seated more firmly. Hinge pins, linkages and links should be of corrosion resistant material.

The maintenance of flap gates requires regular inspection and removal of debris from the pipe and outlet chamber, lubrication of hinge pins and cleaning of seating surfaces.

4.8 MEASURING DEVICES

For gauging the flow in sewers, a venturi flume may be used, since notches or weirs would cause deposition because of heading up of sewage. In the Palmer-Bowlus flume, flow is made to converge into a throat so that a control section is created. The floor is flat and bent for a length equal to the pipe dia and flow conditions are such that the stream generally leaves the throat at supercritical velocity. The Parshall flume is generally rectangular in section with a sloping floor with all the dimensions in rigidly specified proportions (Fig. 11.2 and Table 11.2).

Weirs are mostly used for gauging the flows at treatment plants. The shape of the opening varies but generally the triangular weirs are used for smaller flows (upto 1250 lps) and rectangular weirs are used for larger flows (3.2 of companion volume of water supply and treatment, 1976). The weirs are generally installed in a manhole. The reading for the depth should be taken at the weir section where the liquid level is parallel to the bed and at sufficient frequency so as to ensure that all major changes in flow rates are recorded.

4.9 SEWER VENTILATORS

In a modern, well designed, sewerage system, there is no need to provide ventilation on such elaborate scale considered necessary in the past, specially with the present day policy to omit intercepting traps in house connections. The ventilating columns are not necessary where intercepting traps are not provided. It is necessary, however, to make provision for the escape of air to take care of the exigencies of full flow. In the case of storm sewers this can be done by providing ventilating manhole covers. Ventilating columns should be provided on large pumping mains and they should be taken above the hydraulic grade line.

CHAPTER 5

MATERIALS FOR SEWER CONSTRUCTION

Factors influencing the selection of materials for sewer constructions are flow characteristics, availability in the sizes required including fittings and connections, availability and ease of handling and installation, watertightness and simplicity of assembly, physical strength, resistance to acids, alkalies, gases, solvents etc., resistance to scour, durability and cost including handling and installation.

No single material will meet all the conditions that may be encountered in sewer design. Selection should be made for the particular application and different materials may be selected for parts of a single project. Cost of pipe is usually a minor factor for smaller sewers, the differences in cost among the various pipe materials being a small fraction of the total project outlay.

5.1 TYPES OF MATERIAL

5.1.1 Brick

Brickwork is often used for construction of sewers, particularly for larger diameters. Many old brick sewers are still in use; the failures are mainly due to the disintegration of the bricks or the mortar joints. Because of the comparatively higher cost, larger space requirement, slower progress of work and other factors, brick is now used for sewer construction only in special cases. The advantage of brick sewers is that these could be constructed to any required shape and size.

Brick sewers shall have cement concrete or stone for invert and 12·5 mm thick cement plaster with neat finish for the remaining surface. To prevent ground water infiltration, it is desirable to plaster the outside surface. Under special conditions protection against corrosion may be necessary.

5.1.2 Concrete

Concrete pipes may be manufactured to any reasonable strength required by varying the wall thickness and the percentage of reinforcement and shape of the reinforcing cage. A number of jointing methods are available depending on the tigthness required and the operating pressure within the sewer line.

The advantages of concrete pipes are the relative ease with which the required strength may be provided, feasibility of adopting a wide range of pipe sizes and the rapidity with which the trench may be opened and backfilled.

However, these pipes are subject to corrosion where acid discharges are carried in the sewer or where velocities are not sufficient to prevent septic conditions or where the soil is highly acidic or contains excessive sulphates. Protective linings or coatings as discussed in 5.3.1 should be used inside and outside where excessive corrosion is likely to occur. Only high alumina cement concrete should be used when it is exposed to corrosive sewage or industrial wastes. When specifying concrete pipe, the pipe diameter, class or strength, the method of jointing and the type of protective coating and lining, if any, should be stipulated. Structural requirements of RCC and other pipes are discussed in Chapter 6.

5.1.2.1 *Precast concrete*

Plain cement concrete pipes are used in sewerage systems on a limited scale only and generally reinforced concrete pipes are used. Non-pressure pipes are used for gravity flow and pressure pipes are used for force mains, submerged outfalls, inverted siphons and for gravity sewers where absolute watertight joints are required. Nonpressure pipes used for the construction of sewers and culverts shall conform to IS : 458. Certain heavy duty pipes which are not specified in IS : 458 should conform to other approved standards.

5.1.2.2 *Cast-in-situ reinforced concerete*

Cast-in-situ reinforced concrete sewers are constructed where it is more economical, or when non-standard sections are required, or when a special shape is required or when the headroom and working space are limited. The sewer shape should be of an economic design, easy to construct and maintain and should have good hydraulic characteristics. Rectangular sewers having their widths in excess of one and a half times their heights become uneconomical and have poor hydraulic characteristics. Wide flat culvert bottoms should be provided with a "Vee", of atleast 15cm depth in the centre.

All form work for concerte sewers should be unyielding and tight and should produce a smooth sewer interior. Collapsible steel forms will produce the desirable sewer surface and may be used when the sewer size and length justify the expense.

Reinforcement steel, concrete aggregates, cement and sand should conform to Indian Standard specifications. It is desirable to specify a minimum clear cover of 50 mm over reinforcement steel and a minimum slump consistent with workability should be used for obtaining a dense concrete structure free of voids. The distance for chuting concrete should be kept to a minimum to avoid aggregation and the vibrating of concrete done by approved mechanical vibrators. Air entraining cement or plasticizing agents may be used to improve workability and ensure a denser concrete. Concrete should conform to IS : 456.

5.1.3 Stoneware or Vitrified Clay

Salt glazed stoneware pipes are manufactured in sizes 80 mm to 600 mm in dia but sizes greater than 380 mm dia are not generally used because of economic considerations. Specifications for the AA and A classes are identical except that in the case of Grade AA pipes, 100% hydraulic testing has to be carried out at the manufacturing stage while in the case of Class A only 5% of the pipes are tested hydraulically (IS : 651). The lengths of vitrified clay pipes are 60 cm., 75 cm. and 90 cm. the preference being for the longer pipes for obvious reasons.

Standard pipe fittings of vitrified clay are available to meet most requirements.

When specifying vitrified clay pipes, the pipe diameter, class or strength, the method of jointing and the type of protective coating or lining, if any, should be stipulated.

The resistance of vitrified clay pipes to corrosion from most acids and to erosion due to grit and high velocities gives it an advantage over other pipe materials in handling those wastes which contain high acid concentrations. Though a minimum crushing strength of 1600 kg/m is usually adopted for all sizes manufactured presently, vitrified clay pipes of crushing strength 2800 kg/m and over are manufactured in sizes upto 750 mm dia in most of the advanced countries. The strength of vitrified clay pipes often necessitates special bedding or concrete cradling to improve field supporting strength.

5.1.4 Asbestos Cement

Asbestos cement pipes are usually manufactured in sizes ranging from 500 mm to 1000 mm in dia (IS : 1592).

Some of the advantages of A. C. pipes are : noncorrosiveness to most natural soil conditions; freedom from electrolytic corrosion; good flow characteristics; light weight; ease in cutting, drilling, threading and fitting with G. I. specials; allowance of greater deflection upto about 12° with mechanical joints; ease of handling; tight joints; and quick laying and backfilling.

A. C. pipes cannot, however, stand high superimposed loads and may be broken easily. They are subject to corrosion by acids, highly septic sewage and by highly acidic or high sulphate soils. Protective measures as in 5.3.1 should be provided in such cases. While using A.C. pipes, strict enforcement of approved bedding practices will reduce possibility of flexure failure. Where grit is present, high velocities such as those encountered on steep grades may cause erosion.

5.1.5 Iron and Steel

5.1.5.1 *Cast iron*

Cast iron pipes in sizes ranging from 150 to 750 mm in dia with a variety of jointing methods are used for pressure sewers, sewers above ground surface, submerged outfalls, piping in sewage treatment plants and occasionally on gravity sewers where absolutely water tight joints are essential or where special considerations require their use. IS: 1536 and IS : 1537 give the specifications for spun and vertically cast pipes respectively.

The advantages of cast iron pipes are long laying lengths with tight joints, ability when properly designed to withstand relatively high internal pressures and external loads and corrosion resistance in most natural soils. They are however subject to corrosion by acids or highly septic sewage and acid soils.

Whenever it is necessary to deflect pipes from a straight line either in the horizontal or in the vertical plane, the amount of deflection allowed should not normally exceed 2·5° for lead caulked joints and not more than 10° for mechanical joints.

When specifying cast iron pipe, it is necessary to give the pipe class, the type of joint, the type of lining and the type of exterior coating.

5.1.5.2 *Steel*

Aqueducts, pressure sewer mains, underwater river crossings, bridge crossings, necessary connections for pumping stations, self-supporting spans, railway crossing and penstocks are some of the situations where steel pipes are preferred.

Steel pipes can withstand internal pressure, impact load and vibrations much better than C.I. pipes. They are more ductile and withstand water hammer better. They are generally preferred for diameters above 750 mm.

The disadvantage of steel pipe is that it cannot withstand high external load. Further the main is likely to collapse when it is subjected to negative pressure.

Steel pipes are susceptible to various types of corrosion. A thorough soil survey is necessary all along the alignment where steel pipes are proposed. Steel pipes should be protected from external corrosion by cathodic protection. Protective measures as discussed in 8·8 of the companion volume : Manual on water supply and Treatment (Second Edition) may be adopted.

Steel pipes should conform to IS : 3589-Electrically Welded Steel Pipes (200 mm to 2000 mm) for gas, water and sewage and laying should conform to IS : 5822.

5.1.6 ***Plastic***

The use of plastic, polyethylene or unplasticised PVC for sewer pipes carrying domestic sewage is not common. But in special cases where industrial wastes with corrosion problems are to be handled, these pipes may be conveniently used.

Some of the advantages of plastic pipes are resistance to corrosion, excellent flow characteristics resulting in flatter ruling gradients and economy in excavation, light weight, longer lengths, faster laying, cold negotiation of bends, flattening out effect in water hammer and greater shock resistance.

Among some of the disadvantages of plastic pipes are reduction of strength with increasing temperature, stress cracking and ductile failure in vacuum.

5.2 JOINTING IN SEWER PIPES

From the consideration of structural requirements, joints may be classified as rigid and flexible joints. Joints such as cement mortar, lead, flanged and welded joints are under the category of rigid joints as they do not withstand any angular rotation. All types of mechanical joints such as rubber gasket joints are flexible as they take rotation to the extent of a few degrees and thus reduce the undue settlement stress. Flexible joints are preferable to rigid joints, particularly with granular bedding.

Chapter 5 of the companion volume : Manual on Water Supply and Treatment (Second Edition) gives the types of joints used for C.I., steel, AC, concrete and plastic pipes. The socket and spigot type of joint is the most widely used joint for vitrified clay pipes. Internal flush joints have also been occasionally used.

5.3 CORROSION PREVENTION IN SEWERS

The main cause of corrosion of sewers is chemical reaction between the constituents of sewage and materials of sewers that come in intimate contact with each other and exposure to the gases particularly hydrogen sulphide coming out from the decomposition in the sewers. Some concrete structures which have been immune to attack over a period of years suddenly begin to corrode due to a change in the flow characteristics and in the biological balance in the sewage resulting in rapid generation of hydrogen sulphide which is oxidised to sulphuric acid. Corrosion control methods can either be the treatment of the sewage or the treatment of the conveyance system.

In order to choose the type of protective measure, it is necessary to know the characteristics of the sewage carried and the manner in which the corrosion takes place. The more important of the contributing factors are high temperature of sewage, high BOD, low velocity of flow, detention period in force mains and wet wells, degree of turbulence in partially filled conduits and lack of ventilation.

5.3.1 **Protective Barriers**

Commonly used protective barriers for steel, concrete and stoneware pipes are: cement plasters; epoxy resin; PVC sheets; bitumen and coaltar products; fibre glass; and paints.

All these linings should be provided under strict supervision and control conforming to IS specifications and the directions of the manufacturers as applicable.

Protecting concrete and asbestos-cement pipe against acid attack by means of a barrier is difficult. Coatings and linings of bituminous or coaltar products, vinyl and epoxy resins and paints have been used with varied success for the protection of pipes and structures. If any acid seeps or diffuses through the lining at any point, even through a pinhole, reaction with the cement ensues and the effectiveness of the lining is destroyed. To be effective, the lining, including joints, must be sealed completely to protect the sewer system throughout its expected life.

A lining used on large-diameter concrete pipe that has proved reasonably satisfactory is a plasticised polyvinyl chloride sheet, having T-shaped projections on the back which key into the pipe wall at the time of manufacture. All pining seams and the joints between the pipes must be sealed completely.

5.3.2 **Modification of Material**

Normally, the materials that are most suitable under circumstances likely to be encountered should be used commensurate with the economy. The pipe at joint should be as impervious as possible to prevent ground water from permeating through the walls and forcing the lining away. The remedial measures discussed below mainly apply to concrete sewers and appurtenances.

(i) To prevent the sulphate corrosion, the use of special type of cement, such as slag cement for concrete sewers is recommended. Sulphate attack is however, not common in concrete sewers, in all areas. The type of cement used affects the resistance to acid attack.

(ii) On concrete pipes extra wall thickness sometimes is specified to serve as sacrificial concrete and increase the life of the pipe. However, the use of sacrificial concrete is to be considered in relation to the other available methods as it involves substantial increase in capital expenditure and also the thickness needed cannot be predicted.

(iii) Some authorities recommend the use of limestone or dolomite aggregate in concrete as a retardant to corrosion. The use of such aggregates increases the amount of acid soluble material in the concrete that is available to react with the acid which prolongs the life of the pipe. The rate of acid attack of limestone or dolomite aggregate pipe may be only one fifth as great as when granite or trap aggregate is used. However, this method also needs to be used with caution because the compressive strength, specific gravity, abrasion resistance, soundness, absorption and other qualities of limestone can vary over a wide range even in the same quarry.

5.3.3 **Other Measures**

The following measures are also of interest in minimising the sulphide generation and consequent corrosion :

(i) minimising points of high turbulence within the system;

(ii) designing wet wells to preclude surcharge of tributary lines; and

(iii) provision of forced ventilation at a point where air may be depleted seriously of its oxygen.

CHAPTER 6

STRUCTURAL DESIGN OF BURIED SEWERS

The essential steps in the design and construction of buried conduits to provide safe installations are:

(i) determination of the maximum load that will be applied to the conduit based on the trench and backfill conditions and the live load to be encountered;

(ii) computation of the safe load carrying capacity of the conduit when installed and bedded in the manner to be specified using a suitable factor of safety and making certain the design supporting strength thus obtained is greater than the maximum load to be applied;

(iii) specifying the maximum trench widths to be permitted, the type of pipe bedding to be obtained and the manner in which the backfill is to be made in accordance with the conditions used for the design;

(iv) checking each pipe for structural defects before installation and making sure that only sound pipes are installed; and

(v) ensuring, by adequate inspection and engineering supervision that all trench widths, subgrade work, bedding, pipe laying and backfiling are in accordance with design assumptions as set forth in the project specifications.

Proper design and adequate specification is alone are not enough to insure protection from dangerous or destructive overloading of pipe. Effective value of these depends on the degree to which the design assumptions are realised in actual construction. For this reason, thorough and competent inspection is necessary to insure that the installation conforms to the design requirements.

The structural design of a sewer is based on the relationship: the supporting strength of the conduit as installed divided by a suitable factor of safety must equal or exceed the loads imposed on it by the weight of earth and any superimposed loads.

6.1 TYPES OF LOADS

In a buried sewer, stresses are induced by external loads and internal pressure in case of a pressure main. The stress due to external loads is of utmost importance and may be the only one considered in the design. Besides, if the sewer is exposed to sunlight, temperature stresses induced may be considerable and these will have to be taken into consideration particularly in case of metallic pipes. The external loads are of two categories, *viz.*, load due to backfill material known as backfill load and superimposed load which again is of two types *viz.*, concentrated load and distributed load. Moving loads may be considered as equivalent uniformly distributed load. Sewer lines are mostly constructed of stoneware concrete or cast iron which are considered as rigid pipes (while steel pipes, if used, are not considered as rigid pipes). The flexibility of the pipe affects the load imposed on the pipe and the stresses induced.

6.2 LOADS ON CONDUITS DUE TO BACKFILL

Methods for determining the vertical load on buried conduits due to gravity earth forces in all commonly encountered conditions as developed by A. Marston are generally accepted as the most suitable and reliable for computation. Theoretically stated, the load on a buried conduit is equal to the weight of the prism of earth directly over the conduit, called the interior prism of earth plus or minus the frictional shearing forces transferred to the prism by the adjacent prism of earth.

The considerations are :

(a) the calculated load due to the backfill is the load which will develop when ultimate settlement has taken place;

(b) the magnitude of the lateral pressure causing the shearing force is computed by Rankine's theory; and

(c) there is negligible cohesion except for tunnel conditions.

The general form of Marston's formula is

$W=C.w.B^2$..............(6—1)

where W is the vertical load per unit length acting on the conduit due to gravity earth loads;

w is the unit weight of earth;

B is the width of trench or conduit depending upon the type of installation conditions; and

C is a dimensionless co-efficient that measures the effect of :

(a) ratio of height of fill to width of trench or conduit ;

(b) shearing forces between interior and adjacent earth prisms ; and

(c) direction and amount of relative settlement between interior and adjacent earth prisms for embankment conditions.

6.2.1 Types of Installation or Construction Conditions

The accepted types of installation or construction conditions are shown in figure 6.1. There are three classifications for the construction conditions *viz.*, (1) Trench Condition (2) Embankment Condition and (3) Tunnel Condition.

Trench condition exists when the conduit is installed in a relatively narrow trench cut in

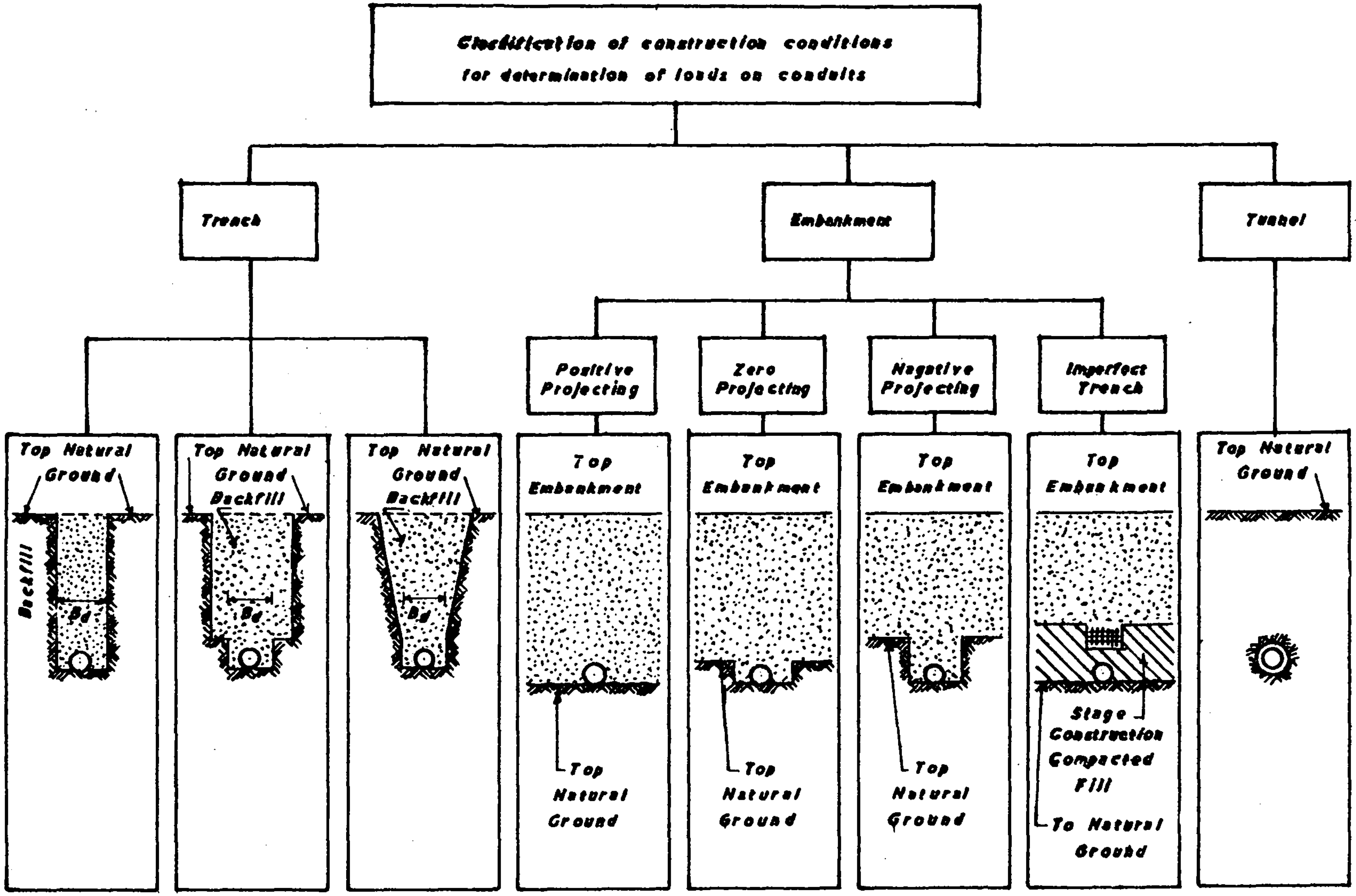


FIG. 6.1 CLASSIFICATION OF CONSTRUCTION CONDITIONS

undisturbed soil and then covered with earth backfill upto the original ground surface.

Embankment condition pervails when the conduit is covered with fill above the original ground surface or when a trench in undisturbed ground is so wide that trench wall friction does not affect the load on the pipe. The embankment condition is further classified, depending upon the position of the top of conduit in relation to the original ground surface, as

(i) positive projecting condition;

(ii) zero projecting condition;

(iii) negative projecting condition ; and

(iv) imperfect trench condition.

Tunnel condition exists when the sewer is placed by means of jacking or tunnelling.

6.2.2 **Loads for Different Conditions**

6.2.2.1 *Trench condition*

Generally sewers are laid in ditches or trenches by excavation in natural or undisturbed soil and then covered by refilling the trench to the original ground level.

(a) LOAD PRODUCING FORCES

The vertical load to which a conduit is subjected under trench conditions is the resultant of two major forces. The first component is the weight of the prism of soil within the trench and above the top of the pipe and the second is due to the friction or shearing forces generated between the prism of soil in the trench and the sides of the trench produced by settlement of backfill. The resultant load on the horizontal plane at the top of the pipe within the trench is equal to the weight of the backfill minus these upward shearing forces as shown in figure 6.2.

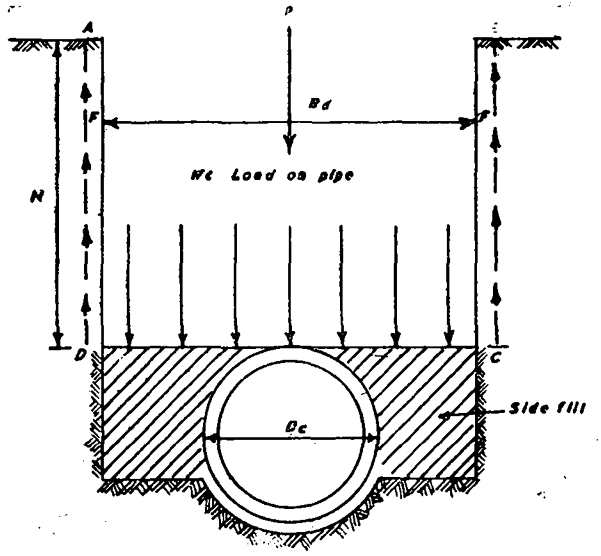


FIG. 6.2 LOAD PRODUCING FORCES

P = WEIGHT OF BACKFILL ABCD

F = UPWARD SHEARING FORCES ON AD AND BC AND $W_c = P - 2F$

(b) COMPUTATION OF LOADS

The load on rigid conduits in trench condition is given by the Marston's formula in the form

$$W_c = C_d w B_d^2 \quad \ldots\ldots\ldots\ldots\ldots (6\text{—}2)$$

in which

W_c is the load on the pipe in kg per linear metre,

w is the unit weight of backfill soil in kg/m³,

B_d is the width of trench at the top of the pipe in m, and

C_d is the load coefficient which is a function of the ratio of height of fill to width of trench (H/B_d) and of the friction coefficient between the backfill and the sides of the trench.

Weights of common filling materials (w) and values of C_d for common soil conditions encountered are given in Tables 6.1 and 6.2 respectively.

Table 6·1

Weights of Common Filling Material

Material	*Weight kg/m³*
Dry sand	1 600
Ordinary (damp) sand	1 840
Wet sand	1 920
Damp clay	1 920
Saturated clay	2 080
Saturated topsoil	1 840
Sand and damp soil	1 600

Equation (6—2) gives the total vertical load due to backfill in the horizontal plane at the top of the conduit as shown in Figure 6.2 if the pipe is rigid. For flexible conduits, the formula may be modified as

$$W_c = C_d w B_c B_d \ldots\ldots\ldots \quad (6\text{—}3)$$

where B_c is the outside width of the conduit in m.

(c) INFLUENCE OF WIDTH OF TRENCH

It has been experimentally seen that when the width of trench excavated is not more than twice the external width of the conduit, the assumption made in the trench condition of loading holds good. If the width of the trench goes beyond three times the outside dimension of the conduit, it is necessary to apply the embankment condition of loading. In the transition width from $B_d = 2B_c$ to $B_d = 3B_c$ computation of load by both the procedures will give the same results.

Table 6·2

Values of Cd. for calculating loads on pipes in Trenches ($W_c = C_d.\ W\ B^2_d$)

Ratio H/B_d*	Safe Working Values of C_d				
	Minimum possible without Cohesion**	Maximum for Ordinary Sand ***	Completely saturated Top soil	Ordinary Maximum for Clay ****	Extreme Maximum for Clay *****
0·5 .	0·455	0·461	0·464	0·469	0·474
1·0 .	0·830	0·852	0·864	0·881	0·898
1·5 .	1·140	1·183	1·208	1·242	1·278
2·0 .	1·395	1·464	1·504	1·560	1·618
2·5 .	1·606	1·702	1·764	1·838	1·923
3·0 .	1·780	1·904	1·978	2·083	2·196
3·5 .	1·923	2·075	2·167	2·298	2·441
4·0 .	2·041	2·221	2·329	2·487	2·660
4·5 .	2·136	2·344	2·469	2·650	2·856
5·0 .	2·219	2·448	2·590	2·798	3·032
5·5 .	2·286	2·537	2·693	2·926	3·190
6·0 .	2·340	2·612	2·782	3·038	3·331
6·5 .	2·386	2·675	2·859	3·137	3·458
7·0 .	2·423	2·729	2·925	3·223	3·571
7·5 .	2·454	2·775	2·982	3·299	3·673
8·0 .	2·479	2·814	3·031	3·366	3·764
8·5 .	2·500	2·847	3·073	3·424	3·845
9·0 .	2·518	2·875	3·109	3·476	3·918
9·5 .	2·532	2·898	3·141	3·521	3·983
10·0 .	2·543	2·918	3·167	3·560	4·042
11·0 .	2·561	2·950	3·210	3·626	4·141
12·0 .	2·573	2·972	3·242	3·676	4·221
13·0 .	2·581	2·989	3·266	3·715	4·285
14·0 .	2·587	3·000	3·283	3·745	4·336
15·0 .	2·591	3·009	3·296	3·768	4·378
Very Great .	2·599	3·030	3·333	3·846	4·548

W_c=load on pipe in kg per linear metre.

C_d=Coefficient.

w=Weight of trench filling material in kg/m³.

B_d=Width of trench a little below the top of the pipe in metres.

* Height of Fill above top of pipe to width of trench a little below the top of the pipe.

** These values give the loads generally imposed by granular filling materials before tamping or settling.

*** Use these values as safe for all ordinary cases of sand filling.

**** Thoroughly wet. Use these values as safe for all ordinary cases of clay filling.

***** Completely saturated. Use these values only for extremely unfavourable conditions.

In case of excavations with sloping sides (possible in undeveloped areas), the provision of a sub-trench (Figure 6·3) minimises the load on the pipe by reducing the value of B_d.

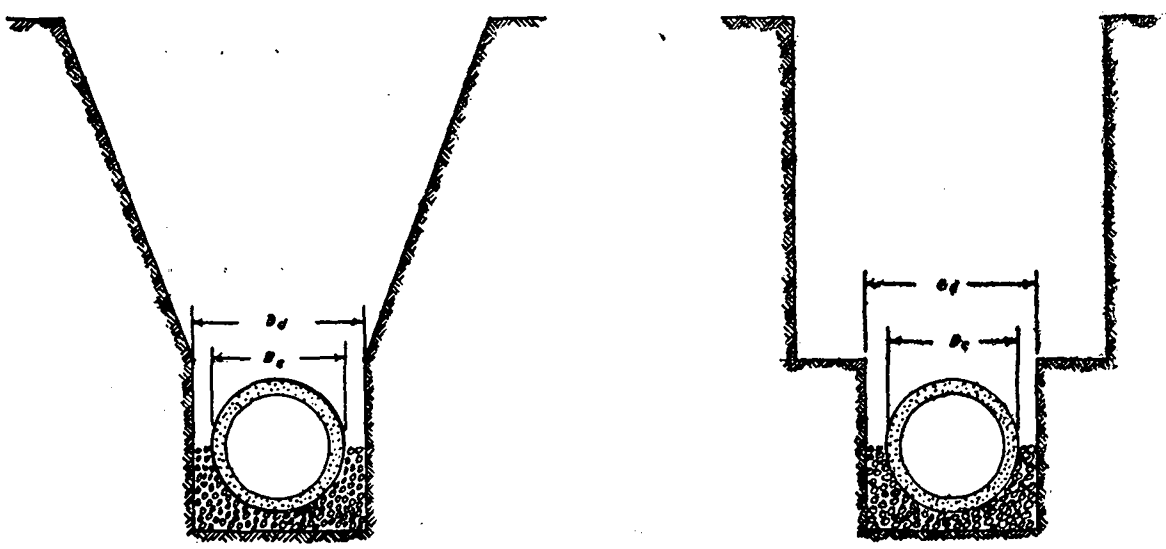


FIG 6.3 EXAMPLES OF SUBTRENCH

6.2.2.2 *Embankment or projecting conduit condition*

(1) POSITIVE PROJECTING CONDUIT

A conduit is said to be laid as a positive projecting conduit when the top of the conduit is projecting above the natural ground into the overlying embankment (Figure 6·4).

(a) Load producing forces

The load on the positive projecting conduit isequal to the weight of the prism of soil directly above the structure plus or minus vertical shearing forces which act in a vertical plane extending upward into the embankment from the sides of the conduit. These vertical shearing forces ordinarily do not extend to the top of the embankment but terminate in a horizontal plane at some elevation above the top of the conduit known as the plane of equal settlement as shown in figure 6·4 which also shows the elements of settlement ratios.

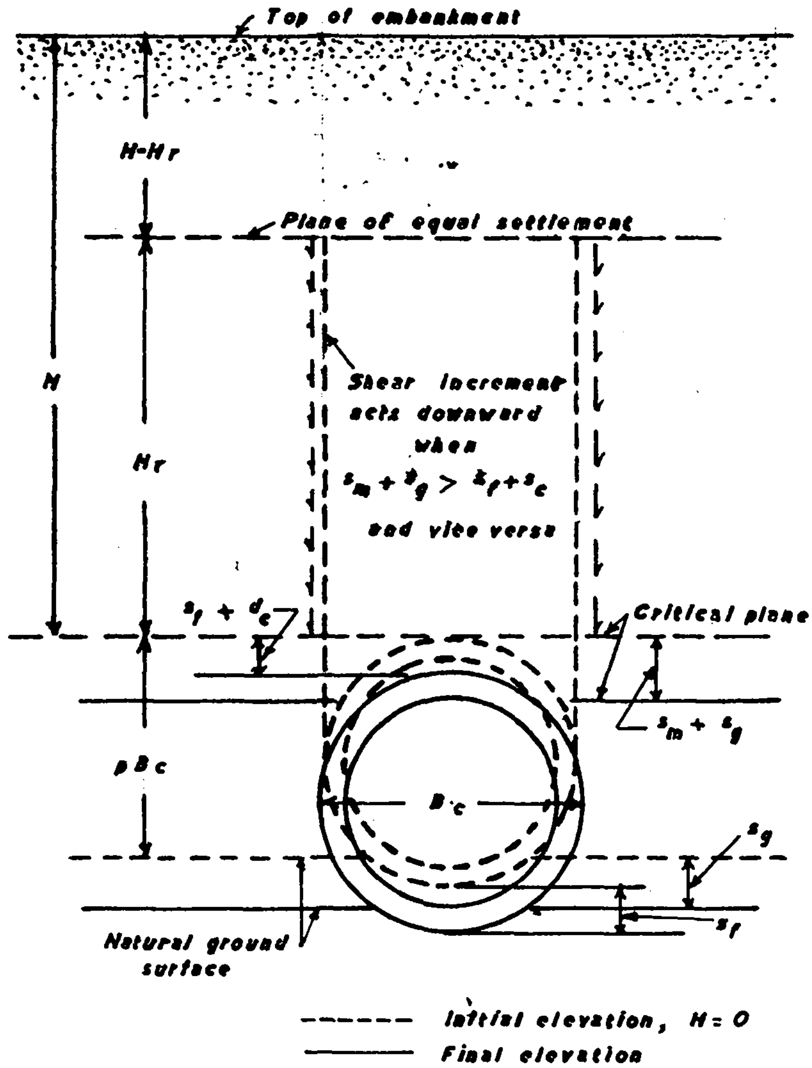


FIG. 6.4 SETTLEMENTS THAT INFLUENCE LOADS ON POSITIVE PROJECTING CONDUITS.
s_g = SETTLEMENT OF NATURAL GROUND ADJACENT TO CONDUIT, s_m = COMPRESSION OF COLUMNS OF SOIL OF HEIGHT pBc, d_c = DEFLECTION OF THE CONDUIT, AND s_f = SETTLEMENT OF BOTTOM OF CONDUIT.

Settlement ratio r_{sd}

$$= \frac{\text{Settlement of critical plane—settlement of top of conduit}}{\text{Compression of height of column H of embankment}}$$

$$= \frac{(S_m + S_g) - (S_f + d_c)}{S_m} \quad \ldots\ldots \quad (6\text{—}4)$$

where, H = height of top of conduit above adjacent natural ground surface (initial) or the bottom of a wide trench;

= p. B_c where p is the projection ratio;

S_m = compression column of height H of embankment;

S_g = settlement of natural ground adjacent to the conduit;

S_f = settlement of the bottom of conduit; and

d_c = deflection of conduit or shortening of its vertical height.

When $(S_m+S_g) > (S_f + d_c)$, r_{sd} is positive *i.e.* the shearing forces act downwards. Therefore the load on conduit is equal to weight of critical prism plus shear force.

When $(S_m+S_g) < (S_f + d_c)$, r_{sd} is negative and the shear force acts in the upward direction.

The settlement ratio r_{sd}, therefore, indicates the direction and magnitude of the relative settlement of the prism of earth directly above and adjoining the conduit.

The product $r_{sd} \times p$ gives the relative height of plane of equal settlement and hence of the magnitude of the shear component of the load.

When $r_{sd} \times p = 0$, the plane of equal settlement coincides with the critical plane and there are no shearing forces and the load is equal to the weight of the central prism. It is not practicable to predetermine this r_{sd} value. However, recommended design values based on actual experience are given below :—

Type of conduit	Type of soil	Settlement ratio (r_{sd})
1. Rigid .	Rock or unyielding foundation.	+1·0
2. Rigid .	Ordinary foundation	+0·5 to +0·8
3. Rigid .	Yielding foundation	0 to +0·5
4. Rigid .	Negative projecting installation.	—0·3 to —0·5
5. Flexible .	Poorly compacted sidefill.	—0·4 to 0
6. Flexible .	Well compacted sidefill.	0.

(b) Computation of loads

Marston's formula for positive projecting conduits (both rigid and flexible) is as follows :

$$W_c = C_c \, w \, B^2_c \quad \ldots\ldots\ldots\ldots (6—5)$$

where, W_c=load on conduit in kg/m;

w=unit weight of backfill material in kg/m³;

B_c=outside width of conduit in m; and

C_c=load coefficient, which is a function of the product of the projection ratio and the settlement ratio and of the height of fill above the top of the conduit to the outside width of the conduit (H/B_c). It is also influenced by the coefficient of internal friction of the backfill material and the Rankine's ratio of lateral pressure to vertical pressure $K\mu$. Suggested values for $K\mu$ for positive and negative settlement ratios are 0·19 and 0·13 respectively.

The value of C_c can be obtained from Figure 6·5.

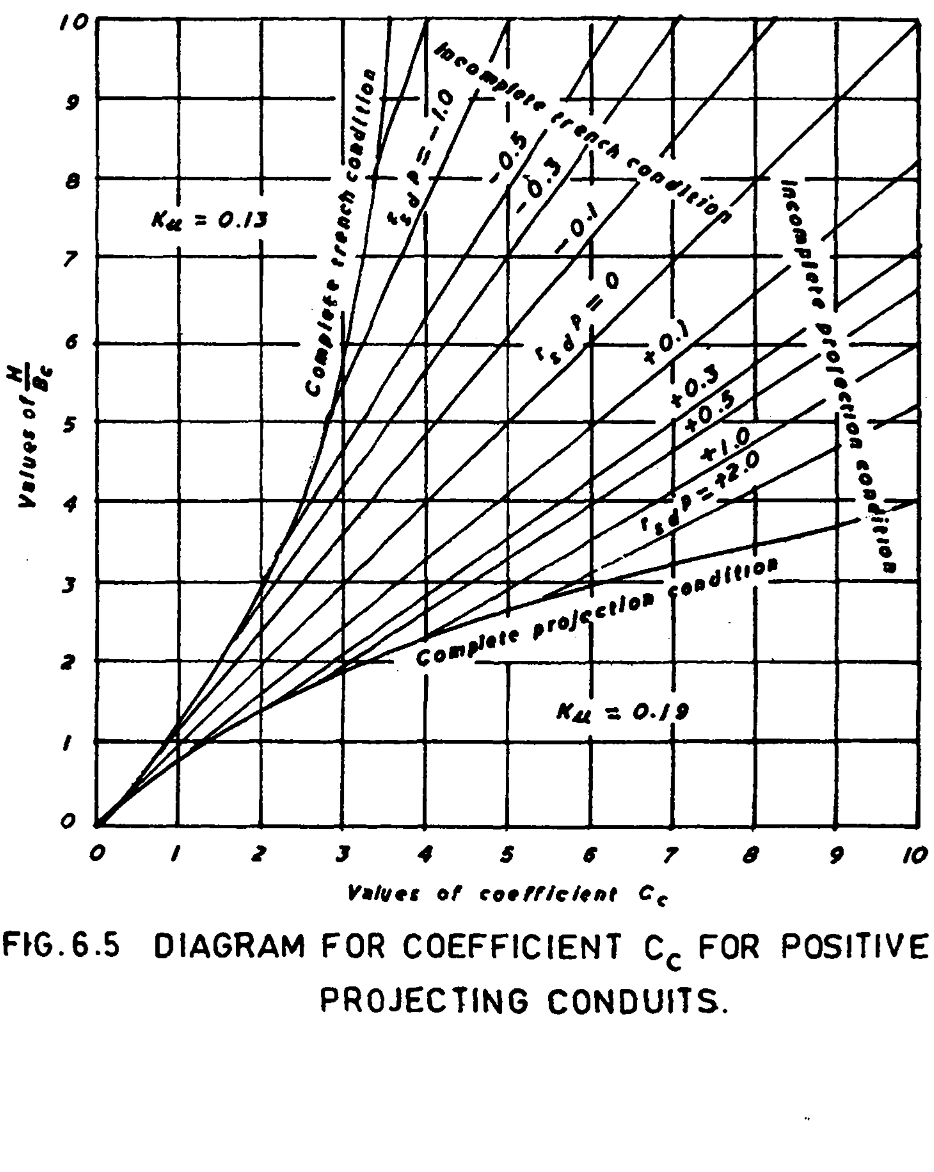


FIG. 6.5 DIAGRAM FOR COEFFICIENT C_c FOR POSITIVE PROJECTING CONDUITS.

(ii) NEGATIVE PROJECTING CONDUIT

A conduit is said to be laid in a negative projecting condition when it is laid in a trench which is narrow with respect to the size of pipe and shallow with respect to depth of cover and the native material of the trench is of sufficient strength that the trench shape can be maintained dependably during the placing of the embankment; the top of the conduit being below the natural ground surface and the trench refilled with loose material and the embankment constructed above (figure 6·6). The prism of soil above conduit, being loose and great in depth compared to the adjoining embankment, will settle more than the prism over the adjoining areas thus generating upward shear forces which relieve or reduce the load on the conduit.

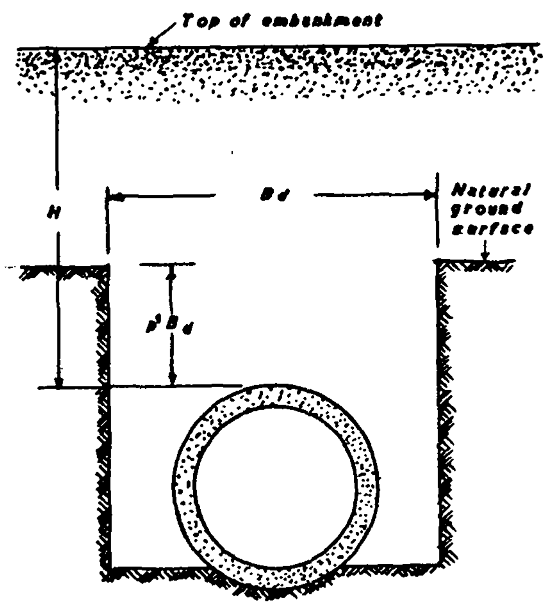


FIG. 6.6 NEGATIVE PROJECTING CONDUIT

(a) Computation of loads

Marston's formula for negative projecting conduits is given by

$$W_c = C_n \, w \, B_d^2 \qquad (6\text{—}6)$$

where—

W_c=load on the conduit in kg/m;

B_d= the width of trench in m; and

w = the unit weight of soil in kg/m³

C_n =load coefficient, which is a function of the ratio (H/B_d) of the height of fill and the width of trench equal to the projection ratio p' (Vertical distance from the firm ground surface down to the top of the conduit/width of the trench) and the settlement ratio r_{sd} given by the expression.

$$r_{sd} = \frac{\text{Settlement of natural ground—Settlement of critical plane}}{\text{compression of the backfill within the height } p' \, B_d} = \frac{S_g - (S_d + S_f + d_c)}{S_d} \quad \ldots\ldots \quad (6\text{—}7)$$

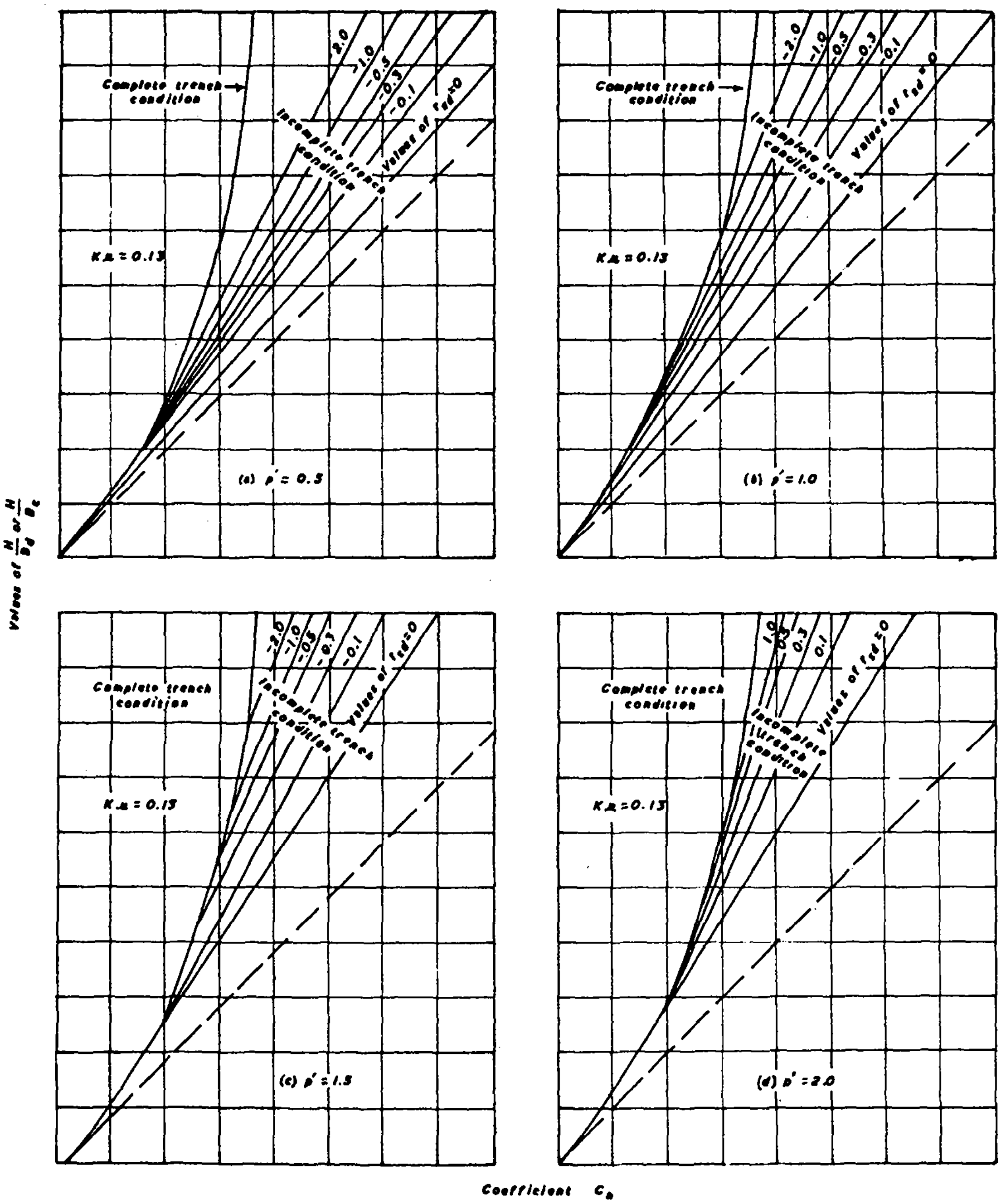


FIG. 6.7 COEFFICIENT C_n FOR NEGATIVE PROJECTING CONDUITS AND IMPERFECT TRENCH CONDUITS.

Values of C_n for various values of H/B_d, r_{sd} and p' are given in Figure 6·7.

Exact determination of the settlement ratio is very difficult.

Recommended value of r_{sd} is—0·3 for design purposes. Elements of settlement ratio are shown in Figure 6·8.

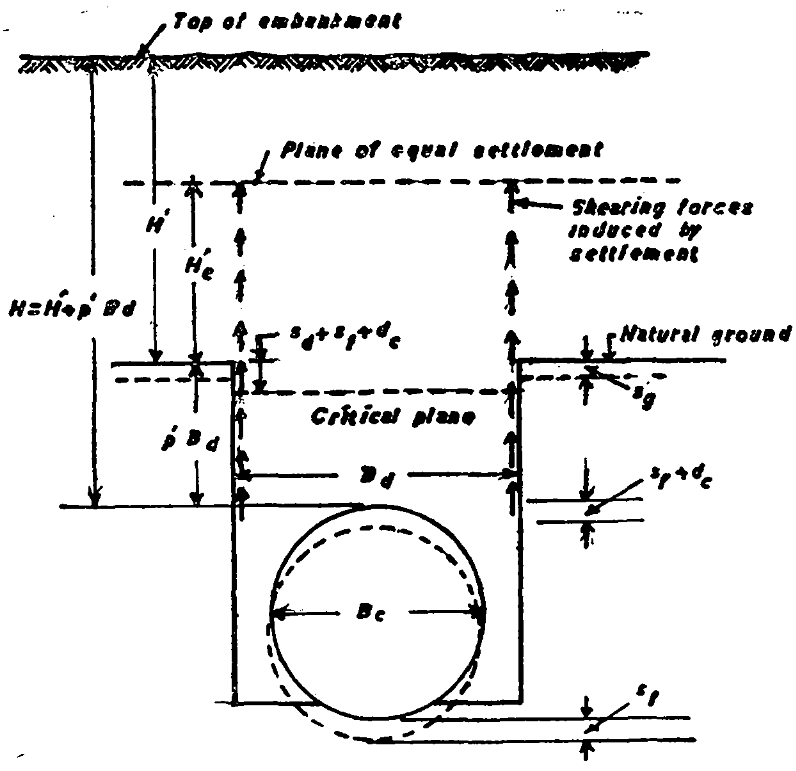


FIG. 6.8 SETTLEMENTS THAT INFLUENCE LOADS ON NEGATIVE PROJECTING CONDUITS.

(iii) IMPERFECT TRENCH CONDUITS

An imperfect trench conduit is employed to minimise the load on a conduit under embankments of unusual heights. The conduit is first installed as a positive projecting conduit. The embankment is then built up to some height above the top and thoroughly compacted as it is placed. A trench of the same width as the conduit is excavated directly over it down to or near its top. This trench is refilled with loose compressible material and the balance of the embankment completed in a normal manner (figure 6·9)

The Marston's Formula for this installation condition is given by $W_c = C_n \; W \; B_c^2 \ldots (6\text{—}8)$

The values of C_n in this case also may be obtained from Figure 6·7 for negative projecting conduits taking $B_c = B_d$ on the assumption that the trench in fill is no wider than the pipe.

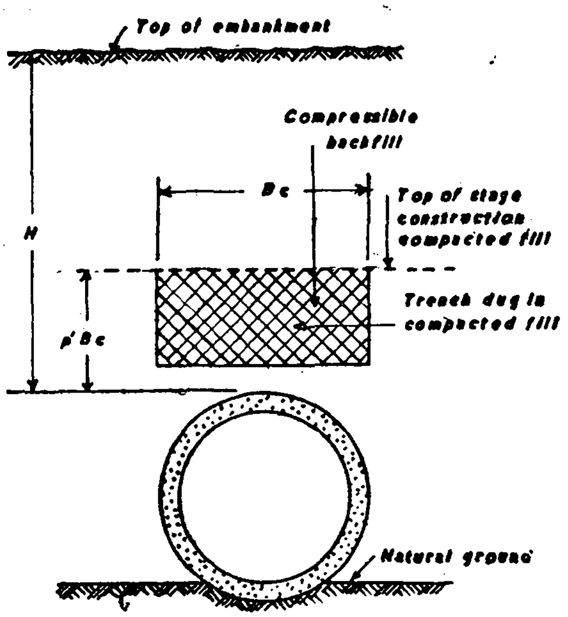


FIG. 6.9 IMPERFECT TRENCH CONDUIT

6.2.2.3 *Tunnel condition*

When the conduit is laid more than 9 to 12 m deep or when the surface obstructions are such that it is difficult to construct the pipeline by the conventional procedure of excavation and backfilling, it may be more economical to place the conduit by means of tunnelling. The general method in this case is to excavate the tunnel, to support the earth by suitable means and then to lay the conduit. The space between the conduit and the tunnel is finally filled up with compacted earth or concrete grout as indicated in figure 6-10.

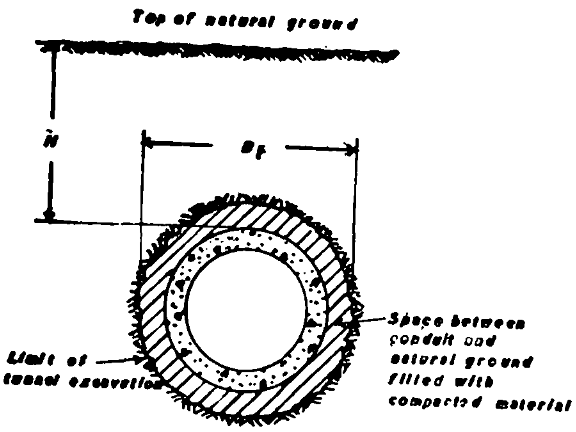


FIG. 6.10 CONDUIT IN TUNNEL

(a) LOAD PRODUCING FORCES

The vertical load acting on the tunnel supports and eventually the pipe in the tunnel is the resultant of two major forces viz., the weight of the overhead prism of soil within the width of the tunnel excavation and the shearing forces generated between the interior prisms and the adjacent material due to friction and cohesion of the soil.

(b) LOAD COMPUTATIONS

Marston's Formula to be used in this case of installation of conduit is given by :

$W_t = C_t\ B_t\ (wB_t—2C)$ .... (6—9)

where

- W_t is the load on the pipe or tunnel support in kg/m;
- w is the unit weight of soil above the tunnel in kg/m³;
- B_t is the maximum width of the tunnel excavation in m;
- C is the coefficient of cohesion in kg/m²; and
- C_t is a load coefficient which is a function of the ratio (H/B_t) of the distance from the ground surface to the top of the tunnel to the maximum width of tunnel excavation and of the coefficient of internal friction of the material of the tunnel.

When the coefficient of cohesion is zero, the formula reduces to the same form as in trench condition (eqn 6—2)

Value of C_t for various values of H/B_t and different soil conditions are to be obtained from Figure 6.11.

Recommended values of coefficient of cohesion for different types of soils are as under :

Type of soil	kg/m²
Soft clay	200
Medium clay	1200
Hard clay	4700
Loose dry sand	0
Silty sand	500
Dense sand	1400
Saturated Top soil	500

6.2.2.4 *Effect of submergence*

Sewers may be laid in trenches or under embankment in areas which may be temporarily or permanently submerged in water. The fill load in such cases will be reduced and will correspond to the buoyant weight of the fill material. However, effect of submergence could be ignored which produces an additional factor of safety, but it may be necessary to check whether a pipe is subject to floatation. Under submergence, the minimum height of the fill material that will be required to prevent floatation ignoring the frictional forces in the fill can be determined from the equation :

$$H_{min}.\ B_c\ (w_s—w_o)+w_e=\pi\ B_c^2.w_s \ldots (6—10)$$

where

- H_{min} = minimum height of fill material in m;
- w_s = the saturated density of the soil in kg/m³;
- w_o = the density of water in kg/m³;
- w_e = the unit weight of the empty pipe in kg/m³; and
- B_c = the outside width of the conduit in m.

6.3 LOAD ON CONDUIT DUE TO SUPERIMPOSED LOADS

The types of superimposed loads which are generally encountered in buried conduits may be categorised as (a) concentrated load and (b) distributed load. These are explained diagrammatically in Figure 6·12.

6.3.1 **Concentrated Load**

The formula for load due to superimposed concentrated load such as a truck wheel

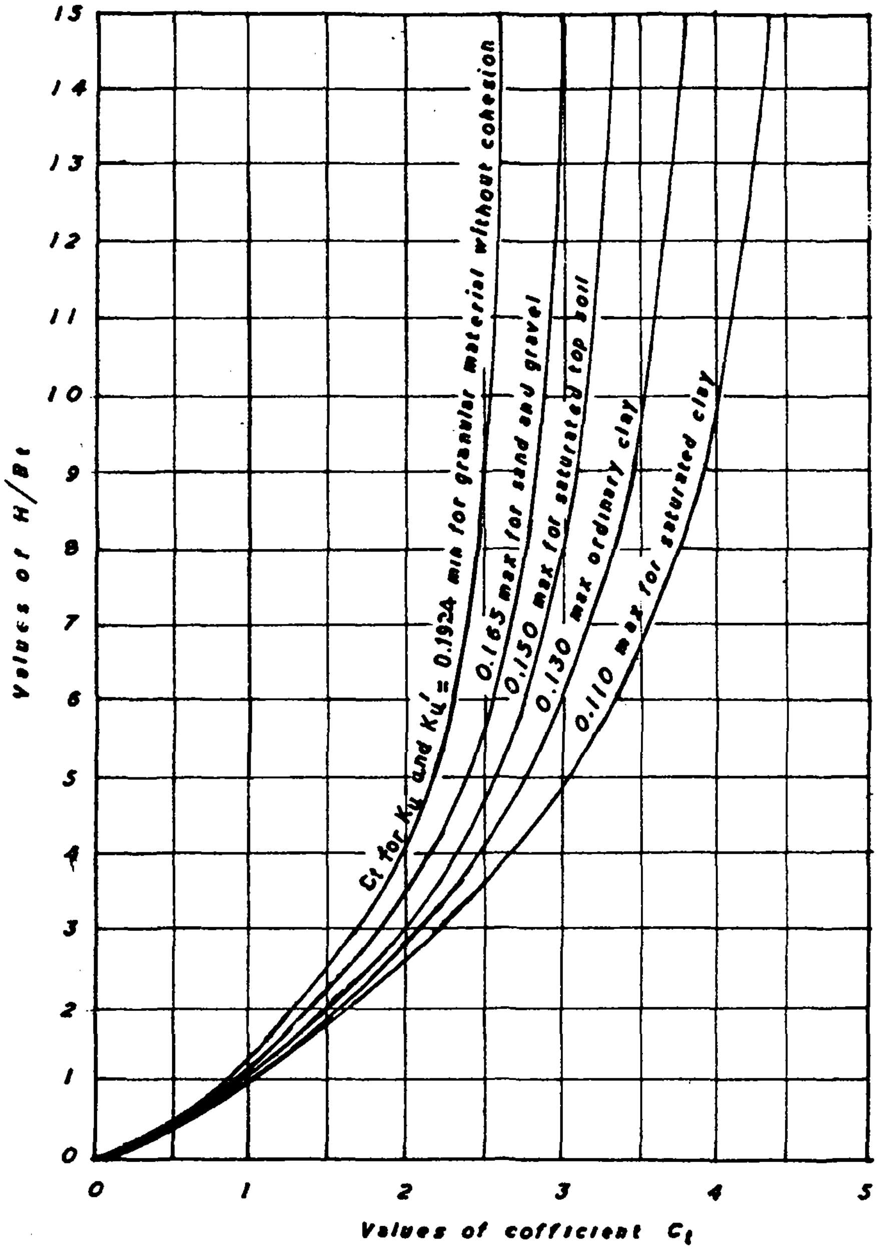


FIG. 6.11 DIAGRAM FOR COEFFICIENT C_t FOR TUNNELS IN UNDISTURBED SOIL.

(figure 6·12) is given by the following form by Holl's integration of Boussinesq's formula

$$W_{sc}=C_s\frac{PF}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6\text{—}11)$$

where

W_{sc} = the load on the conduit in kg/m;

P = the concentrated load in kg acting on the surface;

F = the impact factor (1·0 for air field runways, 1.5 for highway traffic and air field taxi ways, 1·75 for railway traffic); and

C_s = the load coefficient which is a function of :

$$\frac{B_c}{2H} \text{ and } \frac{L}{2H}$$

where

H = the height of the top of the conduit to ground surface in m;

B_c = the outside width of conduit in m; and

L = the effective length of the conduit to which the load is transmitted in m;

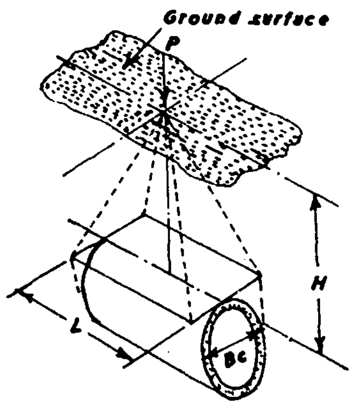


FIG. 6.12 A CONCENTRATED SUPERIMPOSED LOAD VERTICALLY CENTRED OVER CONDUIT.

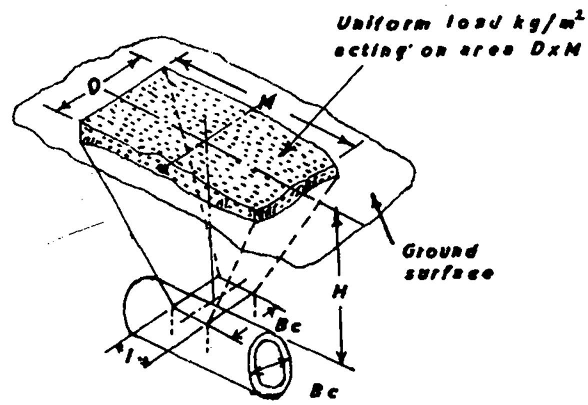


FIG. 6.12 B DISTRIBUTED SUPERIMPOSED LOAD VERTICALLY CENTRED OVER CONDUIT.

Values of C_s for various values of $\frac{B_c}{2H}$ and $\frac{L}{2H}$ are obtained from Table 6·3.

Table 6·3

Values of Load Coefficients, C_s, for Concentrated and Distributed Superimposed Loads Vertically Centred Over Conduits.

$\frac{D}{2H}$ or $\frac{Bc}{2H}$	$\frac{M}{2H}$ or $\frac{L}{2H}$: 0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9	1·0	1·2	1·5	2·0	5·0
0·1 .	0·019	0·037	0·053	0·067	0·079	0·089	0·097	0·103	0·108	0·112	0·117	0·121	0·124	0·128
0·2 .	0·037	0·072	0·103	0·131	0·155	0·174	0·189	0·202	0·211	0·219	0·229	0·238	0·244	0·248
0·3 .	0·053	0·103	0·149	0·190	0·224	0·252	0·274	0·292	0·306	0·318	0·333	0·345	0·355	0·360
0·4 .	0·067	0·131	0·190	0·241	0·284	0·320	0·349	0·373	0·391	0·405	0·425	0·440	0·454	0·460
0·5 .	0·079	0·155	0·224	0·284	0·336	0·379	0·414	0·441	0·463	0·481	0·505	0·525	0·540	0·548
0·6 .	0·089	0·174	0·252	0·320	0·379	0·428	0·467	0·499	0·524	0·544	0·572	0·596	0·613	0·624
0·7 .	0·097	0·189	0·274	0·349	0·414	0·467	0·511	0·546	0·584	0·597	0·628	0·650	0·674	0·688
0·8 .	0·103	0·202	0·292	0·373	0·441	0·499	0·546	0·584	0·615	0·639	0·674	0·703	0·725	0·740
0·9 .	0·108	0·211	0·306	0·391	0·463	0·524	0·574	0·615	0·647	0·673	0·711	0·742	0·766	0·784
1·0 .	0·112	0·219	0·318	0·405	0·481	0·544	0·597	0·639	0·673	0·701	0·740	0·774	0·800	0·816
1·2 .	0·117	0·229	0·333	0·425	0·505	0·572	0·628	0·674	0·711	0·740	0·783	0·820	0·849	0·868
1·5 .	0·121	0·238	0·345	0·440	0·525	0·596	0·650	0·703	0·742	0·774	0·820	0·861	0·894	0·916
2·0 .	0·124	0·244	0·355	0·454	0·540	0·613	0·674	0·725	0·766	0·800	0·849	0·894	0·930	0·956

The effective length of the conduit is defined as the length over which the average load due to surface traffic units produces the same stress in the conduit wall as does the actual load which varies in intensity from point to point. This is generally taken as 1 m or the actual length of the conduit if it is less than 1 m.

6.3.2 Distributed Load

For the case of distributed superimposed loads, the formula for load on conduit is given by—

$$W_{sd} = C_s p F B_c \quad \ldots\ldots\ldots\ldots\ldots \quad (6\text{—}12)$$

in which

W_{sd} = the load on the conduit in kg/m.

p = the intensity of the distributed load in kg/m²;

F = the impact factor;

B_c = The width of the conduit in m.

C_s = the load coefficient, a function of D/2H; and M/2H from table 6·2;

H = the height of the top of conduit to the ground surface in m; and

D and M are width and length in m respectively of the area over which the distributed load acts.

For class AA IRC loading, in the critical case of wheel load of 6·25 tonnes, the intensity of distributed load with wheel area 300 mm × 150 mm is given by

$$p = \frac{6 \cdot 25}{0 \cdot 3 \times 0 \cdot 15} \text{ in } T/m^2.$$

6.3.3 Conduits under Railway Track

The load on conduits under railway track is given by

$$W = 4C_s \; UB_c \qquad \ldots\ldots\ldots\ldots\ldots\ldots (6\text{—}13)$$

where

U is the uniformly distributed load in tonnes/m^2 from the surface directly over the conduit and equal to

$$U = \frac{PF + 2W_tB}{4AB} = \frac{PF}{4AB} + \frac{W_t}{2A} \ldots\ldots\ldots (6\text{—}14)$$

where

- P is the axle load in tonnes (22.5 tonnes for Broad gauge);
- F is the impact factor for railroad = 1.75;
- 2A is the length of the sleeper in m (2.7m for Broad gauge);
- 2B is the distance between the two axles (1.84 m for broad gauge);
- Wt is the weight of the track structure in tonnes/m (0.3 tonnes/m for broad gauge);
- C_s is the load coefficient which depends on the weight of the top of sleeper from the top of the conduit; and
- B_c is the width of the conduit in m. For broad gauge track the formula will reduce to :

$$W = 32.14 . C_s \; B_c \ldots\ldots\ldots\ldots\ldots \qquad (6.15)$$

6·4 SUPPORTING STRENGTH OF RIGID CONDUIT

The ability of a conduit to resist safely the calculated earth load depends not only on its inherent strength but also on the distribution of the vertical load and bedding reaction and on the lateral pressure acting against the sides of the conduit. The inherent strength of a rigid conduit is usually expressed in terms of the three-edge bearing test results, the conditions of which are, however, different from the field load conditions. The magnitude of the supporting strength of a pipe as installed in the field is dependent upon the distribution of the vertical load and the reaction against the bottom of the pipe. It also depends on the magnitude and distribution of the lateral pressure action on the sides of the pipe.

6.4.1 Laboratory Test Strength

All rigid pipes may be tested for strength in the Laboratory by the three-edge bearing test (ultimate load). Methods of test and minimum strength for concrete (unreinforced and reinforced), stoneware and AC pipes and other details are given in Appendix 7.

6.4.2 Field Supporting Strength

The field supporting strength of a rigid conduit is the maximum load per unit length which the pipe will support while retaining complete serviceability when installed under specified conditions of bedding and backfilling. The field supporting strength, however, does not include any factor of safety. The ratio of the strength of a pipe under any stated condition of loading and bedding to its strength measured by the three-edge bearing test is called the load factor.

The load factor does not contain a factor of safety. Load factors have been determined experimentally and analytically for the commonly used construction conditions for both trench and embankment conduits.

6.4.3 Supporting Strength in Trench Conditions

(a) CLASSES OF BEDDING

Four classes, A, B, C and D of bedding most often used for pipes in trenches are illustrated in Figure 6.13. Class A bedding may be either concrete cradle or concrete arch. Class B is a bedding having a shaped bottom or compacted granular bedding with a carefully compacted backfill. Class C is an ordinary bedding having a shaped bottom or compacted granular bedding but with a lightly compacted backfill. Class D is one with flat bottom trench with no care being taken to secure compaction of backfill at the sides and immediately over the pipe and hence is not recommended. Class B or C bedding with a compacted granular bedding is generally recommended.

(b) LOAD FACTORS

The load factors for the different classes are as follows :

Class		Load factor
A a	Concrete cradle—plain concrete and lightly tamped backfill	2·2
A b	Concrete cradle—plain concrete with carefully tamped backfill	2·8
A c	Concrete cradle—R.C.C. with p=0·4% (p is the ratio of the area of steel to the area of concrete at the invert)	upto 3·4
A d	Arch type—plain concrete	2·8
	R.C.C. (p=0·4%)	upto 3·4
	R.C.C. (p=1·0%) (p is the ratio of the area of steel to the area of concrete at the crown)	upto 4·8
B	Shaped bottom or compacted granular bedding with carefully compacted backfill	1·9
C	Shaped bottom or compacted granular bedding with lightly compacted backfill	1·5
D	Flat bottom trench	1·1

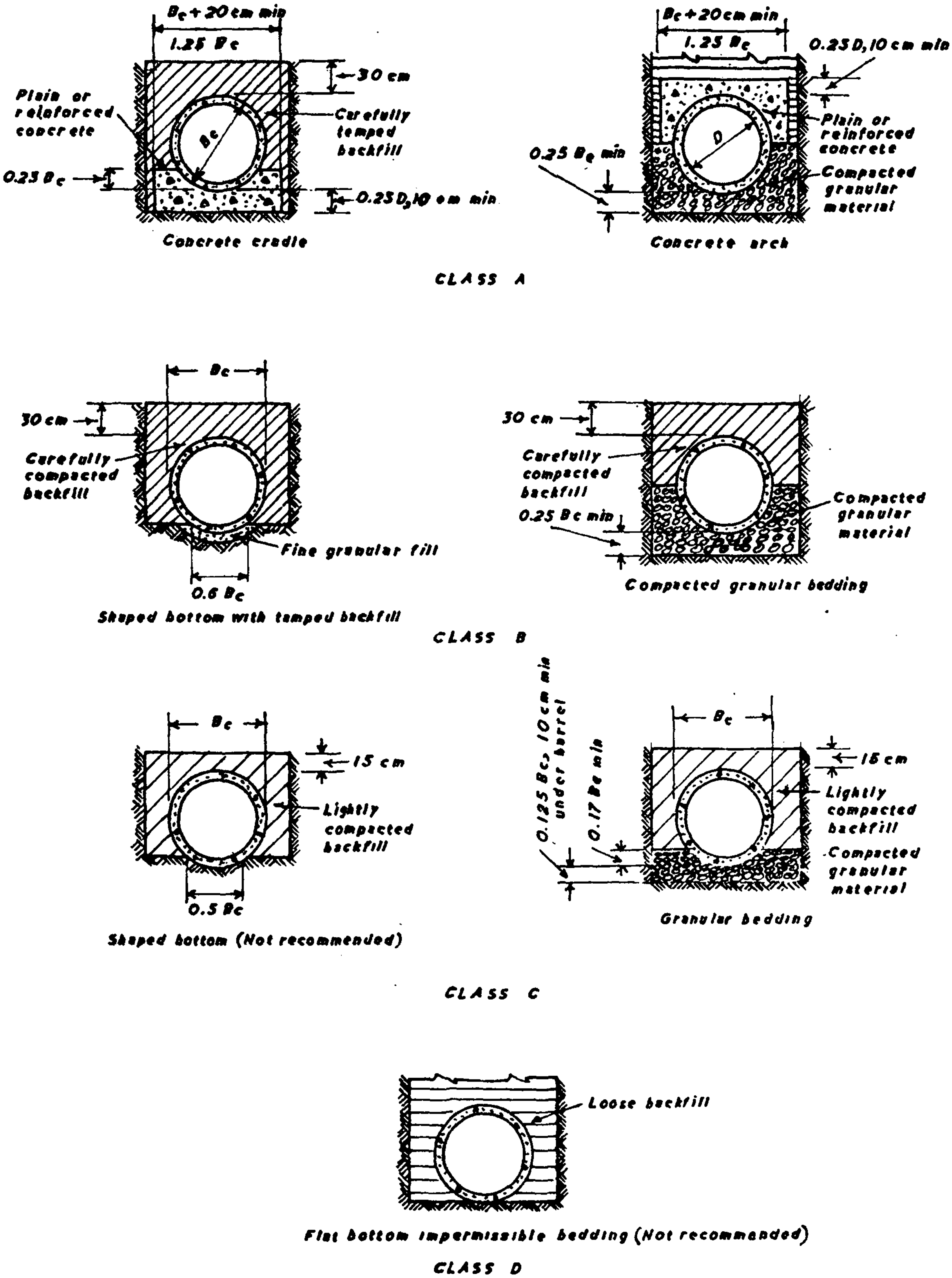


FIG. 6.13 CLASSES OF BEDDING FOR CONDUITS IN TRENCH

NOTE :- IN ROCK TRENCH EXCAVATED AT LEAST 15 cm BELOW THE BELL OF THE PIPE EXCEPT WHERE CONCRETE CRADLE IS USED

The granular material used must stabilize the trench bottom in addition to providing a firm and uniform support for the pipe. Well graded crushed rock or gravel with the maximum size not exceeding 25 mm is recommended for the purpose.

Where rock or other unyielding foundation material is encountered, bedding may be according to one of the classes A, B, or C but with the following additional requirements :

Class A : The hard unyielding material should be excavated down to the bottom of the concrete cradle.

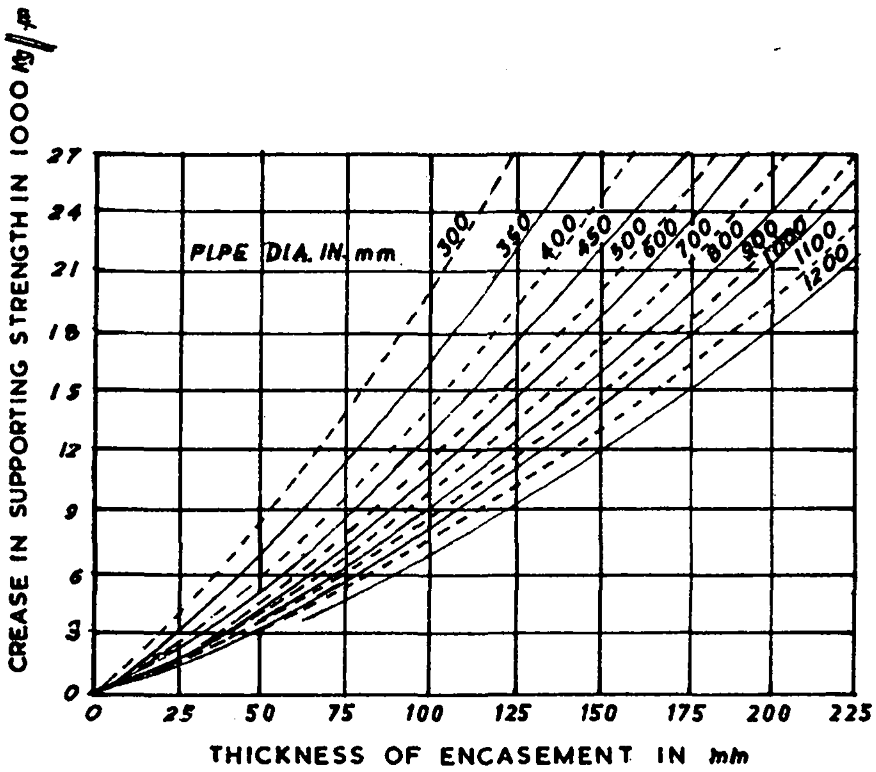
Class B or C : The hard unyielding material should be excavated below the bottom of the pipe and pipe bell to a depth of at least 15 cm.

The width of the excavation should be at least 1.25 times the outside dia of the pipe and it should be refilled with granular material.

Total encasement of non-reinforced rigid pipe in concrete may be necessary where the required safe supporting strength cannot be obtained by other bedding methods. The load factor for concrete encasement varies with the thickness of concrete. The effect of M-200 concrete encasement of various thicknesses on supporting strength of pipe under trench conditions is given in Figure 6.14.

6.4.4 Supporting Strength in Embankment Conditions

The soil pressure against the sides of a pipe placed in an embankment may be significant in resisting the vertical load on the structure.



EFFECT OF M-200 CONCRETE ENCASEMENT OF VARIOUS THICKNESS ON SUPPORTING STRENGTH OF PIPE UNDER TRENCH CONDITIONS.

FIGURE: 6.14

(a) CLASSES OF BEDDING

The beddings which are generally adopted for projecting conduits laid under the embankment conditions of installation are illustrated in Figure 6.15. The classification of the beddings are as under :

Class A : In this case the conduit is laid on a mat of concrete.

Class B : The conduit is laid on accurately shaped earth to fit the bottom of the pipe and the sides are filled with thoroughly tamped earth.

Class C : In this type of bedding the conduit is laid on accurately shaped earth to fit the bottom surface of the conduit. For rock foundations the conduit is laid on a layer of granular cushion and the sides of the conduit are filled up.

Class D : The conduit is laid on earth not shaped to fit the bottom of the

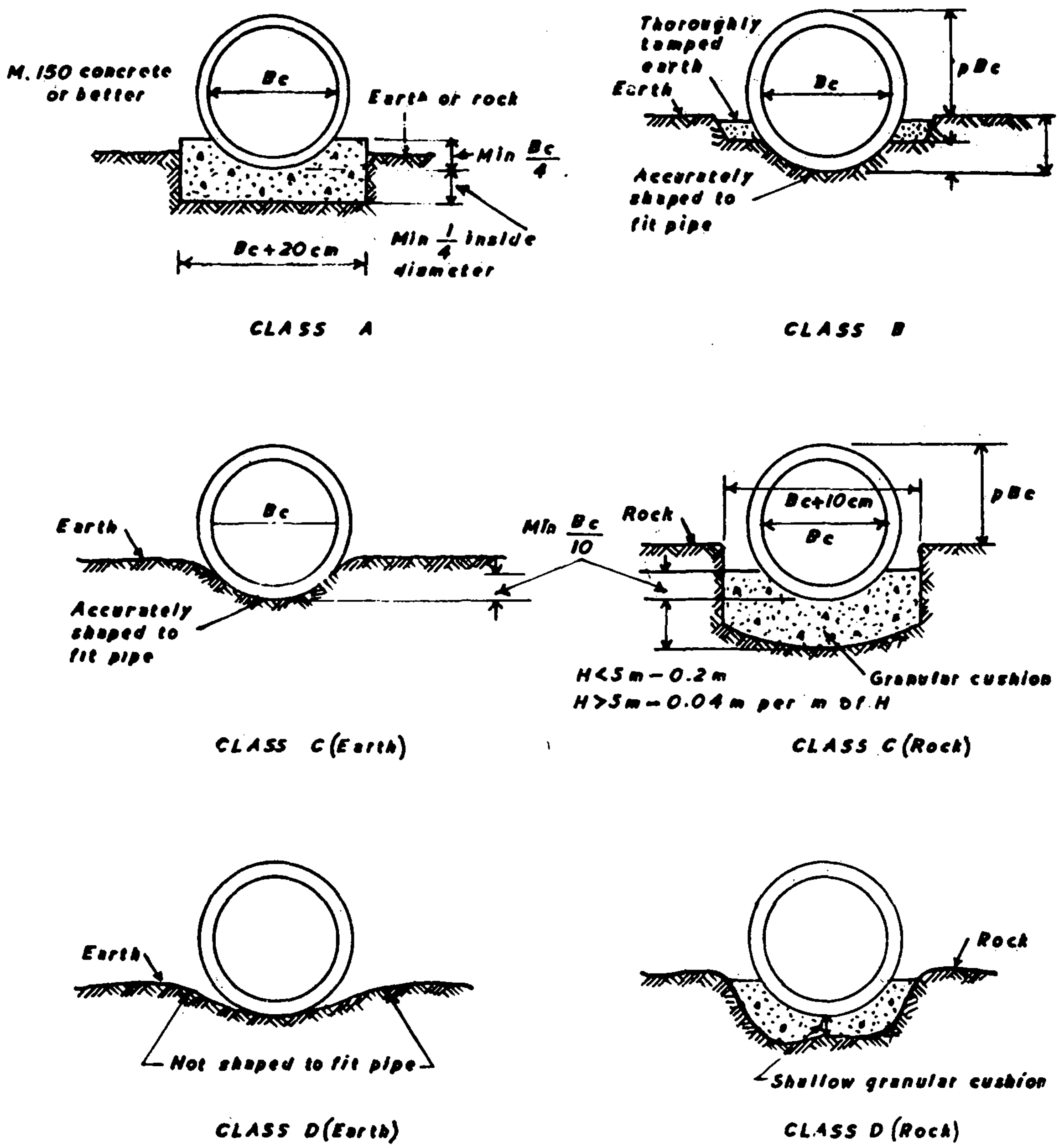


FIG. 6.15 CLASSES OF BEDDING FOR PROJECTING CONDUITS.

conduit. In case of rocky soil the conduit is laid on a shallow granular cushion.

(b) LOAD FACTORS

The load factor for rigid pipes installed as projecting conduits under embankments or in wide trenches is dependent on the type of bedding, the magnitude of the active lateral soil pressure and on the area of the pipe over which the active lateral pressure acts.

The load factor for projecting circular conduits may be calculated by the formula.

$$L_f = \frac{1.431}{N x q} \quad \text{(6—16)}$$

where

L_f is the load factor;

N is a parameter dependent on the type of bedding;

x is a parameter dependent upon the area over which the lateral pressure acts effectively; and

q is the ratio of total lateral pressure to total vertical load on pipe.

(i) Positive projecting conduits

The ratio 'q' for positive projecting conduits may be estimated by the formula

$$q=\frac{mk}{C_c}\left\{\frac{H}{B_o}+\frac{m}{2}\right\} \qquad (6-17)$$

where

k is the Rankine's ratio which may be taken as 0.33. The value of N for different types of beddings for circular pipes are given below :

Type of bedding	*Value of "N"*
'A' Reinforced concrete cradle	0.42 to 0.51
'A' Plain concrete cradle	0.51 to 0.64
'B'	0.71
'C'	0.84
'D'	1.31

The value of "x" in case of circular pipes is given below :

Fraction of conduit on which lateral pressure acts 'm'	*Value of 'x' for*	
	'A' class bedding	*Other 'beddings*
0	0·150	0
0·3 . . .	0·743	0·217
0·5 . . .	0·856	0·423
0·7 . . .	0·811	0·594
0·9 . . .	0·678	0·655
1·0 . . .	0·638	0·638

(ii) Negative projecting conduits

The load factor for negative projecting conduits may also be determined by the equations (6—16) and (6—17) with a value of k of 0.15, provided the side fills are well compacted.

(iii) Imperfect trench conditions

The equations for positive projecting conditions will hold good for these conditions as well.

6.4.5 Conduits Under Simultaneous Internal Pressure and External Loading

Simultaneous action of internal pressure and external load gives a lower supporting strength of a pipe than what it would be if the external load acted alone.

If the bursting strength and the three-edge strength of a pipe are known, the relation between the internal pressure and external loads which will cause failure may be computed by means of the formula.

$$t=T\left(1-\frac{s^2}{S}\right) \qquad (6\text{-}18)$$

where

t = internal pressure in kg/cm^2 at failure when external load is simultaneously acting;

T = bursting strength of a pipe in kg/cm^2 when no external load is simultaneously acting;

s = three-edge bearing load at failure in kg/linear metre when there is simultaneous action of internal pressure and

S = Three-edge bearing load at failure in kg/linear metre when there is no internal pressure simultaneously acting.

6.5 RELATIONSHIP BETWEEN THE DIFFERENT ELEMENTS IN STRUCTURAL DESIGN

The basic design relationships between the different design elements are :

Safe Supporting Strength (Maximum allowable field load) or w $=\dfrac{\text{Field Supporting Strength}}{\text{Factor of safety}}$

$=\dfrac{\text{Load factor x three edge bearing strength}}{\text{Factor of Safety}}$

∴ Required three-edge bearing strength

$=\dfrac{\text{Maximum allowable field load x Factor of Satety}}{\text{Load factor}}$

A factor of safety of atleast 1.5 should be applied to the specified minimum three-edge bearing strength to determine the working strength for all rigid conduits.

6.6 SAMPLE CALCULATIONS

The general assumptions relating to the characteristics of soil and other factors are given below :

(i) Saturated density of fill (w)=2000 kg/m^3

(ii) $k.\mu=k.\mu'=0.130$, ordinary maximum for clay (thoroughly wet);

(iii) r_{sd} for rigid conduit on ordinary bedding=0.7 for positive projection and —0.3 for negative projection;

(iv) projection ratio=1;

(v) Concentrated surcharge corresponding to wheel load for class AA wheeled loading=6.25 T;

(vi) Impact factor=1.5;

(vii) Factor of safety for safe supporting strength=1.1;

(viii) The design also provides for accidental surcharge of drains and accounts for a water load of 75% as per standard practice, based on the assumption that the sewage flow is 3/4 full.

Example I

Determine the fill load on a 1200 mm dia NP_2 class concrete pipe installed in a trench of width 2.3 m and depth of 4.00 m.

Pipe thickness 't'=65 mm for D of 1200 mm.

$B_c=D+2t=1200+130=1330$ mm=1.33 m

$B_d=2.3$m

H=4.00—1.33=2.67 m

$\therefore H/B_d=\frac{2.67}{2.3}=1.16$

B_d is $<$ $2B_c$. Hence trench formula is applicable. C_d.=0.9965 or 1.00 (from table 6.2) for ordinary maximum for clay.

∴from equation (6—2)

$W_c = C_d w B_d^2 = 1.00 \times 2000 \times 2.3^2 = 10{,}580$ kg/m.

Example II

Determine the fill load on a 900 mm dia NP_2 class concrete pipe installed in a trench of width 2.1 m and depth 6.0 m.

Pipethickness 't'=50 mm for D for 900 mm.

$B_c = D + 2t = 900 + 100 = 1000 = 1$ m.

$w = 2000$ kg/m^3

$H = 6.0 - 1.0 = 5.0$ m

$B_d = 2.1$

$$\frac{H}{B_d} = \frac{5.0}{2.1} = 2.38$$

$2B_c < B_d < 3B_c$. Hence either the trench or embankment formula can be used.

From table 6.2

$C_d = 1.77188$ or say 1.8

From equation (6—2)

$W_c = C_d . w . B_d^2 = 1.8 \times 2000 \times 2.1^2 = 15{,}876$ kg/m or say, 16000 kg/m.

Example III

Determine the fill load on a 1200 mm dia NP_3 class concrete pipe installed as a positive projecting conduit under a fill of 7 m height above the top of pipe. The pipe wall thickness is 65 mm and the fill weight 2000 kg/m^3.

Assume $r_{sd} = 0.7$ and $p = 1.0$

$H = 7$m

$B_c = 1200 + 130 = 1330$ m$= 1.33$m

$H/B_c = 7/1.33 = 5.26$

$r_{sd} \times p = 0.7 \times 1 = 0.7$

$C_c = 9$ (from figure 6.5)

Using equation (6—5)

$W_c = C_c\, w\, B_c^2 = 9 \times 2000 \times 1.33^2 = 31.850$ kg/m

Example IV

Determine the fill load on a 1200 mm dia NP_3 Class pipe installed as a negative projection conduit in a trench the depth of which is such that the top of the pipe is 2 m below the surface of natural ground in which the trench is dug. The height of the fill over the top of the pipe is 10 m.

Assume the width of the trench as 2 m and $w = 2000$ kg/m

$r_{sd} = -0.3$ and $p' = 1.0$

$H = 10$ m, $B_d = 2.00$ m, $H/B_d = 10/2 = 5.00$

For values of $p' = 1.0$, $r_{sd} = -0.3$ and $H/B_d = 5.00$

$C_n = 3.2$ (from figure 6.7)

Using eqn (6—6)

$W_c = C_n\; w\; B_d^2 = 3.2 \times 2000 \times 2.0^2 = 25.600$ kg/m

Example V

Determine the load on 1500 mm dia conduit in tunnel condition 15 m deep in a soil of silty sand.

The maximum width of excavation (B_t) may be assumed as 1950 mm; and the cohesion coefficient (C) of the soil as 500 kg/m^2,

$k\mu = 0.150$ and $w = 1800$ kg/m^3.

$H = 15$ m; $B_t = 1.95$ m

$H/Bt = 15/1.95 = 7.7$

$C_t = 3.00$ (from figure 6.11)

Using eqn (9—9)

$W_t = C_t\, B_t (w B_t - 2C) = 3.00 \times 1.95\ (1800 \times 1.95 - 2 \times 500)$

$= 3.00 \times 1.95 \times 2510 = 14{,}680$ kg/m

Example VI

Determine the load on a 600 mm dia NP_2 Class pipe (t=40 mm) under 1 m cover caused by 6.25 T wheel load applied directly above the centre of pipe.

L =1 m (since standard length of conduit >1 m);

$H = 1$ m

$B_c = 600 + 80 = 680$mm$= 0.68$ m;

$$\frac{L}{2H} = \frac{1.0}{2 \times 1} = 0.50 ; \frac{B_c}{2H} = \frac{0.68}{2 \times 1} = 0.34$$

From table 6.3 for values of $\frac{L}{2H} = 0.50$

and $\frac{B_c}{2H} = 0.34$

$C_s = 0.248$

Using equation (6—11)

$$W_{sc} = \frac{C_s PF}{L} = \frac{0.248 \times 6250 \times 1.5}{1.0} = 2325 \text{kg/m.}$$

Example VII

Determine the load on a 1200 mm dia concrete pipe under 2 m of cover resulting from a broadgauge railway track loading.

Assumed thickness of pipe =100 mm

Axle load P =22·5 tonnes

Impact factor F =1·75

Length of sleeper 2A= D= 2·7 m

Assume 4 axles spaced 1·84 m on the locomotive ;

$M = 4 \times 2B = 4 \times 1{\cdot}84 = 7{\cdot}36$ m;

$H = 2$ m.

Wt of track structure$= w_t = 0{\cdot}3$ T/m

Using equation (6—14)

$$U = \frac{PF+2W_t, B}{4\,AB} = \frac{PF}{4AB} + \frac{W_t}{2A}$$

$$= \frac{22\cdot5\times1\cdot75}{2\cdot7\times1\cdot84} + \frac{0\cdot3}{2\cdot7} \ T/m^2$$

$= 7\cdot925+0\cdot111=8\cdot036$ tonnes/m²

$B_c=1200+200 = 1400$ mm $= 1\cdot4$ m

$$\frac{D}{2H} = \frac{2\cdot7}{2\times2} = 0\cdot675$$

$$\frac{M}{2H} = \frac{4\times1\cdot84}{2\times2} = 1\cdot84$$

From table 6·3,

Influence Coefficient $C_s = 0.652$

Using equation (6—13)

$W = 4\,C_s.\,U.\,B_c$

$= 4\times0\cdot652\times8\cdot036\times1\cdot4$

$= 29\cdot34$ tonnes/m

$= 29340$ kg/m

(Since it has been given that it is a broad gauge track, the formula $W = 32\cdot14\ C_s.\ B_c$ could be used direct without calculating the value of U.).

Example VIII

Design the structural requirement for a 900 mm dia NP_2 class sewer pipe which is to be laid in 6 m deep trench of 2·0 m width assuming that the total vertical load will account for concentrated surcharge of 6·25 T applied at the centre of the pipe. The water load should also be considered.

The type of bedding for the purpose of this example may be assumed as Ab class with load factor of 2·8.

$B_c = 900+2\times50 = 1000$ mm $=1\cdot00$ m;

$H = 6—1 = 5$ m

$B_d = 2\cdot0$

$H/B_d = 5/2\cdot0 = 2\cdot50$

$C_d = 1\cdot764$ (from table 6·2 for saturated top soil).

Using equation (6—2)

$W_c = 1\cdot764\times2000\times2^2 = 14110$ kg/m

$L = 1$ m; $H = 5$ m;

$$\frac{L}{2H} = \frac{1}{10} = 0\cdot1 \text{ and } \frac{B_c}{2H} = \frac{1}{10} = 0\cdot1$$

From table 6·3, $C_s = 0\cdot019$

Using equation (6—11)

$$W_{sc} = C_s \frac{PF}{L}$$

$$= \frac{0\cdot019\times6250\times1\cdot5}{1} = 178 \text{ kg/m}$$

$$W_w = \frac{22}{7}\times\frac{9}{10}\times\frac{9}{10}\times\frac{1}{4}\times1000\times\frac{75}{100} = 471 \text{ kg/m}$$

$W_t = W_c + W_{sc} + W_w = 14{,}110+178+471 = 14{,}759$ or say $= 14{,}800$

Safe supporting strength of 900 mm NP_2 pipe with class Ab bedding = 3750 × 2·8 = 10,500 kg/m (Appendix 7)·

Hence, the load exceeds the safe supporting strength of the pipe by 14800—10,500 = 4,300 kg/m. From figure 6·14, 60 mm encasement in M 200 mix will give the additional strength of 4,300 kg/m.

6·7 RECOMMENDATIONS

The factor of safety recommended for concrete pipes for sewers is considerably less (max. of 1·5 for unreinforced pipes, 1·2 for reinforced pipes laid in trench and 1·5 for all pipes laid under embankment) as compared to that for most engineering structures which have a factor of safety of at least 2·5. As the margin of safety against the ultimate failure is low, it becomes imperative to guarantee that the load imposed on sewer pipes are not greater than the design loads for the given installation conditions. In order to achieve this objective, the following procedures are recommended :

1. The width of trench specified for a particular job should be minimum in consonance with the requirements of adequate working space to allow access to all parts and joints of pipes.
2. Specification should lay proper emphasis on the limit of the width of trench to be adopted in the field which should not exceed that adopted in the design calculations. Any deviations from this requirement during the construction should be investigated for their possible effect on the load coming on the pipe and steps should be taken to improve the safe supporting strength of pipe for this condition of loading by adopting suitable bedding or such other methods when necessary.
3. The Field Engineer should keep in touch with the Design Engineer throughout the duration of the project and any deviation from the design assumptions due to the exigencies of work, should be immediately investigated and corrective measures taken in time.
4. All pipes used on the work should be tested as per the IS specifications and test certificates of the manufacturers should be furnished for every consignment brought to the site.
5. Whenever shoring is used, the pulling out of planks on completion of work, should be carried out in stages and this work should be properly supervised to ensure that the space occupied by the planks is properly backfilled.
6. Proper backfilling methods both as regards to selection of materials, methods of placing and proper compaction should be in general agreement with the design assumptions.

CHAPTER 7

CONSTRUCTION OF SEWERS

7·1 CONSTRUCTION METHODS

The planning and the construction of sewers are so interdependent, the knowledge of one is an essential prerequisite to the competent performance of the other. The ingenuity of the planner, the supervising engineer and the contractor is continually called for, to reduce the construction cost and to achieve a quality workmanship. Barring unforeseen conditions it shall be the responsibility of the supervising engineer and the contractor to complete the work as shown on the plans at minimum cost and with minimum disturbance of adjacent facilities and structures.

7·1·1 Trench

7.1.1.1 *Dimensions*

The width of trench at and below the top of a sewer should be the minimum necessary for its proper installation with due consideration to its bedding. The width of a sewer trench depends on the type of shoring, (single stage or two stage), working space required in the lower part of the trench and the type of ground below the surface. The width of the trench from the top of the sewer to the ground surface is primarily related to its effect upon the adjoining services and nearby structures. In undeveloped areas or open country, excavation with side slope shall be permissible from the top of the sewer to the ground surface instead of vertical excavations with proper shoring. In developed areas, however, it is essential to restrict the trench width so as to protect the existing facilities and properties and to reduce the cost of restoring the surface. Increase in width over the minimum required would unduly increase the load on the pipe.

7.1.1.2 *Excavation*

Excavation for sewer trenches for laying sewers shall be in straight lines and to the correct depths and gradients required for the pipes as specified in the drawings. The material excavated from the trench shall not be deposited very close to the trench to prevent the weight of the materials from causing the sides of the trench to slip or fall. The sides of the trench shall, however, be supported by shoring where necessary to ensure proper and speedy excavation. In case, the width of the road or lane where the work of excavation is to be carried out is so narrow as to warrant the stacking of materials near the trench, the same shall be taken away to a place to be decided by the Engineer incharge. This excavated material shall be brought back to the site of work for filling the trench.

7.1.1.3 *Shoring*

The shoring shall be adequate to prevent caving in of the trench walls or subsidence of areas adjacent to the trench. In narrow trenches of limited depth, a simple form of shoring shall consist of a pair of 40 to 50 mm thick and 30 cm wide planks set vertically at intervals and firmly strutted. For wider and deeper trenches a system of wall plates (wales) and struts of heavy timber section is commonly used. Continuous sheeting shall be provided outside the wall plates to maintain the stability of the trench walls. The number and the size of the wall plates shall be fixed considering the depth of trench and the type of soil. The cross struts shall be fixed in a manner to maintain pressure against the wall plates which in turn shall be kept pressed against the timber sheeting by means of timber wedges or dog spikes.

In noncohesive soils combined with considerable ground water, it may be necessary to use continuous interlocking sheet steel piling to prevent excessive soil movements due to ground water percolation. Such sheet piling shall extend at least 1.5 m below the bottom of the trench unless the lower part of the trench is in firm material. In case of deep trenches, if conditions demand, excavation and shoring may be done in stages.

In case the presence of water is likely to create unstable soil conditions, a wellpoint system shall be employed to drain the immediate area of the sewer trench prior to excavation operation. A wellpoint system consists of a series of perforated pipes driven or jetted into the water bearing strata on either side of a sewer trench and connected with a header pipe leading to a pump.

In the event of excavations being made deeper than necessary, the same shall be filled in with cement concrete of appropriate grade.

7.1.1.4 *Underground services*

All pipes, ducts, cables, mains and other services exposed due to the excavation shall be effectively supported.

7.1.1.5 *Dewatering*

Trenches for sewer construction shall be dewatered for the placement of concrete and the laying of pipe sewer or construction of concrete or brick sewer and kept dewatered until the concrete foundations, pipe joints or brick work and concrete have cured. The pumped out water from the trenches shall be disposed of in the existing storm water drainage arrangement nearby. In the absence of any such arrangement the pumped water may be drained through the completed portion of sewer to a permissible place of disposal. Where a trench is to be maintained dry for a sufficient period of time to permit the placement of forms for sewer construction, an underdrain shall be laid of granular material leading to a sump for further disposal.

7.1.1.6 *Foundation*

Where a sewer has to be laid in a soft under ground strata or in a reclaimed land, the trench shall be excavated deeper than what is ordinarily required. The trench bottom shall be stabilised by the addition of coarse gravel or rock; in case of very bad soil the trench bottom shall be filled in with cement concrete of appropriate grade.

In areas subject to subsidence, pipe sewer shall be laid on a timber platform or concrete cradle supported on piles.

In the case of cast-in-situ sewers, an R. C. C. section with both transverse and longitudinal steel reinforcement shall be provided when intermittent variations in soil bearing capacity are encountered. In case of long stretches of very soft trench bottom, soil stabilization shall be done either by rubble, concrete or wooden crib.

7.1.2 **Tunnelling**

Tunnels are employed in sewer systems when it becomes economical, considering the nature of soil to be excavated and surface conditions with reference to the depth at which the sewer is to be laid. Generally in soft soils the minimum depth is about 10m; in rocks, however, tunnels may be adopted at lesser depths. Crowded conditions on the surface, expensive constructions or presence of other service facilities near the surface sometimes makes it advantageous to tunnel at shallower depths. Each situation has to be analysed in detail before any decision to tunnel is taken.

7.1.2.1. *Shafts*

Shafts are essential in tunnelling to gain access to the depth at which tunnelling is to be done and to remove the excavated material. The size of shaft depends on the type and size of machinery employed for tunnelling irrespective of the size of the sewer. Care should be taken to provide proper timbering or sheeting during construction.

Shafts are not normally placed at less than 150 m depending on the depth and size of tunnel, surface conditions and type of material excavated. Shallower cover on the tunnel demands closer shafts. Provision of numerous shafts, which allows for material to be removed more economically, should be balanced against the extra cost of the shafts.

7.1.2.2 *Ventilation*

All tunnels more than 15 m in length should be provided with ventilation arrangement. Under normal conditions, the requirement of air per person working in the tunnel is about 2.2 m^3/min. If explosive gases are met or if hot conditions develop, then the requirement increases to 5 or 6 times. Careful consideration should be given for providing the necessary facilities for proper ventilation to meet the requirements of the workers.

7.1.2.3 *Construction*

The line and grade of sewer can be transferred to the tunnel work, from the completed position of sewer, if the tunnel is to continue in the same alignment. When tunnelling begins from an isolated shaft, lot of care is to be taken to transfer the line and grade from the surface. If the tunnel work extends for long lengths, especially in a curvilinear alignment, it is necessary to drive vertical line pipes to the elevation of the tunnel at intervals of 180 m or less, so that plumb lines could be dropped through them when excavation reaches these points for checking the alignment and grade.

Alignment of the sewer or tunnel can also be transferred to the bottom of the shaft using wire plumb lines which are heavily weighted with cylindrical steel plumb weights kept freely immersed in a bucket of oil to prevent rotation. The alignment can then be transferred to the spads, set in the roof of the tunnel, by transit and can be extended as the work of tunnelling proceeds. Alignment can be checked with reference to markers on finished sections and from line pipes as encountered.

Tunnel grade can be carried through the excavated portions on bench marks set in the tunnel wall or in case of small boxes, carried down from the bench marks set in the roof in conjunction with alignment spads.

7.1.2.4 *Methods*

The tunnelling methods adopted for sewer construction can be classified generally as auger or boring; jacking of preformed steel or concrete lining; and mining methods.

(a) AUGER OR BORING

In this method, rigid steel or concrete pipes are pushed into ground to reasonable distances and the earth removed by mechanical means from the shaft or pit location. Presence of boulders is a serious deterrent for adoption of this method, in which case it may be more economical to first install an oversize lining by conventional tunnelling or jacking and then place the smaller pipe within the liner pipe and fill the space between the pipe and lining with sand, cement or concrete.

(b) JACKING

In this procedure, the leading pipe is provided with a cutter or edge to protect the pipe while jacking. Soil is gradually excavated by hand and removed through the pipe as successive lengths of pipes are added between the leading pipe and the jacks and pushed forward taking care to limit the jacking upto the point of excavation. This method usually results in minimum disturbance of the natural soils adjacent to the pipe Jacking operation should continue without interruption as otherwise soil friction may increase, making the operation more difficult.

Jacking of permanent tunnel lining is generally adopted for sewers of sizes varying from 750 to 2750 mm, depending upon the conditions of soil and the location of the line. The pipes selected should be able to withstand the loads exerted by the jacking procedure. The most common pipes used for this are reinforced concrete or steel.

(c) MINING

Tunnels larger than 1.5 m are normally built with the use of tunnel shields, boring machines or by open face mining depending on the type of material met with. Rock tunnels normally are excavated open-face with conventional mining methods or with boring tools.

Tunnel shields are used as a safety precaution in mining operations in very soft clay or in running sand especially in built up areas. In this method, a primary lining of adequate strength to support the surrounding earth is installed to provide a progressive backstop for the jacks which advance the shield. As the excavation continues the lining may be installed either against the earth, filling the annular space by grouting with pea gravel or the lining may be expanded against the earth as the shield advances; the latter eliminating need for any grouting.

Boring machines of different types have been developed for tunnel excavation in clay and rock. They are usually provided with cutters mounted on a rotating head which is moved forward as boring operations continue. Earth excavated is usually carried by a conveyor system. Some machines are also equipped with shields. Though the machines are useful in fairly long runs through similar material, difficulties are encountered when the material to be excavated varies.

Open face mining without shields are adopted in particular instances where the conditions permit such operation as in rock. Segmental support of timber or steel is used for the sides and the top of the tunnel.

7.1.3 Laying of Pipe Sewers

In laying sewers, the centre of each manhole shall be marked by a peg. Two wooden posts 100mm × 100mm × 1800mm high shall be fixed on either side at nearly equal distance from the peg and sufficiently clear of all intended excavation. The sight rail when fixed on these posts shall cross the centre of manhole. The sight rails made from 25 cm wide × 40 mm thick wooden planks and screwed with the top edge against the level marks shall be fixed at distances more than 30 m apart along the sewer alignment. The centre line of the sewer shall be marked on the sight rail. These vertical posts and the sight rails shall be perfectly square and planed smooth on all sides and edges. The sight rails shall be painted half white and half black alternately on both the sides and the tee heads and cross pieces of the boning rods shall be painted black. When the sewers converging to a manhole come in at various levels there shall be a rail fixed for every different level.

The boning rods with cross section 75×50 mm of various lengths shall be prepared from wood. Each length shall be a certain number of metres and shall have a fixed tee head and fixed intermedite cross pieces, each about 300 mm long. The top edge of the cross piece shall be fixed at a distance below the top edge equal to, the outside dia of the pipe, the thickness of the concrete bedding or the bottom of excavation, as the case may be. The boning staff shall be marked on both sides to indicate its full length.

The posts and the sight rails shall in no case be removed until the trench is excavated, the pipes are laid, jointed and the filling is started.

Where large sewer lines are to be laid or where sloped trench walls result in top-of-trench widths too great for practical use of sight rails or where soils are unstable, stakes set in the trench bottom itself on the sewer line, as rough grade for the sewer is completed, would serve the purpose.

7.1.3.1 *Stoneware pipes*

The stoneware pipes shall be laid with sockets facing up the gradient, on desired bedding. Special bedding, haunching or encasing may be provided where conditions so demand (as discussed in Chapter 6). All the pipes shall be laid perfectly true, both to line and gradient. (IS : 4127). At the close of each day's work or at such other times when pipe is not being laid, the end of the pipe should be protected by a close fitting stopper.

7.1.3.2 *R. C. C. pipes*

The R. C. C. pipes shall be laid in position over proper bedding, the type of which may be determined in advance, the abutting faces of the pipes being coated by means of a brush with bitumen in liquid condition. The wedge shaped groove in the end of the pipe shall be filled with sufficient quantity of either special bituminous compound or sufficient quantity of cement mortar in grade M-100 or M-250. The collar shall then be slipped over the end of the pipe and the next pipe butted well against the plastic ring by appliances so as to compress roughly the plastic ring or cement mortar into the grooves, care being taken to see that concentricity of the pipes and the levels are not disturbed during the operation. Spigot and socket R. C. C. pipes shall be laid in a manner similar to stoneware spigot and socket pipes. The structural requirements as discussed in Chapter 6 and IS : 783 should be followed.

7.1.3.3 *Cast-in-situ concrete sections*

For sewer sizes beyond 2 m internal dia, cast-in-situ concrete sections shall generally be used, the choice depending upon the relative costs worked out for the specific project. The concrete shall be cast in suitable number of lifts usually two or three. The lifts are generally designated as the invert, the side wall and the arch.

7.1.3.4 *Construction of brick sewers*

Sewers larger than 2 m are generally constructed in brick work. The brick work shall be in CM 1 : 3 and plastered smooth with cement plaster in 1 : 2, 20 mm thick both from inside and outside. A change in the alignment of brick sewer shall be on a suitable curve conforming to the surface alignment of the road. Construction shall conform to IS : 2212 in general.

7.1.3.5 *Cast iron pipes*

The pipes shall be laid in position with the socket ends of all pipes facing up gradient. Any deviations either in plan or elevation of less than $11\frac{1}{4}^{\circ}$ shall be effected by laying the straight pipes round the flat curve of such radius that the minimum thickness of lead at the face of the socket shall not be reduced below 6 mm. The spigot shall be carefully pursued into the socket with one or more laps of spun yarn wound round it. Each joint shall be tested before running the lead, by passing completely round it, a wooden gauge notched out to the correct depth of lead and the notch being held close up against the face of the socket. IS : 3114 should be followed in setting out the sewers.

7.1.4 **Jointing of Sewers**

7.1.4.1 *Stoneware pipes*

All the pipe joints shall be caulked with tarred gasket in one length for each joint and sufficiently long to entirely surround the spigot end of the pipe. The gasket shall be caulked lightly home but not so as to occupy more than a quarter of the socket depth. The socket shall then be filled with a mixture of one part of cement and one part of clean fine sand mixed with just sufficient quantity of water to have a consistency of semi-dry condition and a fillet shall be formed round the joint with a trowel forming an angle 45° with the barrel of the pipe (IS : 4127). Rubber gaskets may also be used for jointing.

7.1.4.2. *Concrete pipes*

The collars shall be placed symmetrically over the end of two pipes and the annular space between the inside of the collar and the outside of the pipe shall be filled with hemp yarn soaked in tar or cement slurry tamped with just sufficient quantity of water to have a consistency of semi-dry condition, well packed and thoroughly rammed with caulking tools and then filled with cement mortar 1 : 2. The joints shall be finished off with a fillet sloping at 45° to the surface of the pipe. The finished joints shall be protected and cured for atleast 24 hours. Any plastic solution or cement mortar that may have squeezed in shall be removed to leave the inside of the pipe perfectly clean. (IS : 783).

7.1.4.3 *C. I. pipes*

The C.I. pipes shall be examined for line and level and the space left in the socket shall be filled in by pouring molten pig lead. This shall be done by using proper leading ring. One or two air vents shall be provided around the lower end of the joint. The lead used shall be soft and of best quality conforming to IS:782. The quantity and depth of lead to be used per joint as well as general procedure for jointing shall be in accordance with IS:3114.

Rubber rings and plastic joints may also be used in special cases.

7.1.5 **Hydraulic Testing of Pipe Sewers**

7.1.5.1 *Water test*

Each section of sewer shall be tested for water tightness preferably between manhole. To prevent change in alignment and disturbance after the pipes have been laid, it is desirable to backfill the pipes upto the top, keeping *atleast 90 cm length of the pipe open at the joints.* However, this may not be feasible in the case of pipes of shorter length, such as stoneware and RCC pipes. With concrete encasement or concrete cradle, partial covering of the pipe is not necessary.

In case of concrete and stoneware pipes with cement mortar joints, pipes shall be tested three days after the cement mortar joints have been made. It is necessary that the pipelines are filled with water for about a week before commencing the application of pressure to allow for the absorption by pipe wall.

The sewers are tested by plugging the upper end with a provision for an air outlet pipe with stop cock. The water is filled through a funnel connected at the lower end provided with a plug. After the air has been expelled through the air outlet, the stop cock is closed and water level in the funnel is raised to 2 m above the invert at the upper end. Water level in the funnel is noted after 30 minutes and the quantity of water required to restore the original water level in the funnel is determined. The pipe line under pressure is then inspected while the funnel is still in position. There shall not be any leaks in the pipe or the joints (small sweating on the pipe surface is permitted). Leakage in 30 minutes determined by measuring the replenished water in the funnel shall not exceed 15 ml in the smaller dia and 60 ml. in the larger dia per cm dia of pipe for 100 m length. Any sewer or part thereof that does not meet the test shall be emptied and repaired or relaid as required and tested again.

For concrete, R. C. C. and Asbestos cement pipes of more than 600 mm dia, the quantity of water inflow can be increased by 10% for each additional 100 mm of pipe dia.

For brick sewers, regardless of their dia, the permissible leakage of water shall not exceed $10 m^3/24$ hrs per km length of sewer.

7.1.5.2. *Air testing*

Air testing becomes necessary particularly in large dia pipes when the required quantity of water is not available for testing.

It is done by subjecting the stretch of pipe to an air pressure of 100 mm of water by means of a hand pump. If the pressure is maintained at 75 mm, the joints shall be assumed to be water tight. In case the drop is more than 25 mm the leaking joints shall be traced and suitably treated to ensure water tightness. The exact point of leakage can be detected by applying soap solution to all the joints in the line and looking for air bubbles.

7.1.6 **Check for Obstruction**

As soon as a stretch of sewer is laid and tested, a double disc or solid or closed cylinder, 75 mm less in dimension than the internal dimension of the sewer shall be run through the stretch of the sewer to ensure that it is free from any obstruction.

7.1.7 **Construction of Manholes**

The manholes shall be constructed simultaneously with the sewers. The manholes shall normally be of brick-work in cement morta rl : 3 and plastered both inside and outside with 20 mm thick cement plaster in CM :1:2. The foundation of manholes shall be 15 cm thick cement concrete of appropriate grade and thickness may be increased to 30 cm when subsoil water is encountered, the projection of concrete being 10 cm on all sides of the external face of brick work. The floor of the manholes shall be in cement concrete of appropriate grade. Salt glazed or concrete half channel pipes of the required size and curve shall be laid and embedded in cement on the concrete base to the same line and fall as the sewer. Both sides of the channel pipes shall be benched up in concrete and rendered smooth in 20 mm thick cement mortar and formed to a slope 1 in 10 to the channel. Bricks on edge shall be cut to a proper form and laid around the upper half of all the pipes entering or leaving the manhole, to form an arch. All around the pipe there shall be a joint of cement mortar 12 mm thick between the pipe and the bricks. The ends of the pipes shall be built in and neatly finished off with cement mortar. The masonry shaft or the manhole shall be provided on the top with a heavy air tight cast iron frame and cover conforming to IS : 1726. Where the depth of the manhole exceeds 90 cm below the surface of the ground, cast iron steps shall be built into the brickwork. The distance between the two consecutive steps shall not be more than 40 cm. The top of manhole shall be flush with the finished road level (IS : 4111 Part I—Manholes).

The entire height of the manhole shall be tested for watertightness by closing both the incoming and outgoing ends of the sewer and filling the manhole with water. A drop in water level not more than 50 mm per 24 hours shall be permitted. In case of high subsoil water it should be ensured that there is no leakage of ground water into the manhole by observing the manhole for 30 minutes after emptying it.

7.1.8 **Sewer Connections**

These shall be laid in the same manner as the sewer. In case the connection is at a level higher than 60 cm, a vertical drop arrangement comprising of 90° bend or a double tee junction encased in 1/2 brick-thick brick work shall be provided. The drop arrangement shall be in brick work in CM 1:3 plastered with 20 mm thick cement plaster from outside in CM 1 : 2. The owest bend may preferably be of cast iron and the entire vertical pipeline encased in concrete. The top end of the drop arrangement in the manhole, when a tee is used, shall be plugged with brick work with a conspicuous mark thereon so that in case a serious sewer choke occurs in the incoming line, this end can be made use of for rodding purposes.

7.1.9 **Backfilling of the Trenches**

Backfilling of the sewer trench is a very important consideration in sewer construction. The method of backfilling to be used varies with the width of the trench, the character of the material excavated, the method of excavation and the degree of compaction required. In developed streets, a high degree of compaction is required to minimise the load while in less important streets, a more moderate specification for back fill may be justified. In open country it may be sufficient to mound the trench and after natural settlement return to regrade the areas.

No trench shall be filled in unless the sewer stretches have been tested and approved for watertightness of joints. However partial filling may be done keeping the joints open to avoid disturbance. The refilling shall proceed around and above the pipes. Soft material screened free from stones or hard substances shall first be used and hand pressed under and around the pipes to half their height. Similar soft material shall then be put upto a height of 30 cm above the top of the pipe and this will be moistened with water and well rammed. The remainder of the trench can be filled with hard material, in stages, each not exceeding 60 cm. At each stage the filling shall be well rammed, consolidated and completely saturated with water and then only further filling-in shall be continued. Before and during the backfilling of a trench, precautions shall be taken against the floatation of the pipe line due to the entry of large quantities of water into the trench causing an uplift of the empty or the partly filled pipe line. Upon completion of the backfill, the surface shall be restored fully to the level that existed prior to the construction of the sewer.

7.1.10 **Removal of sheeting**

Sheeting driven below the spring line of a sewer shall be withdrawn a little at a time as the back-filling progresses. Some of the backfilled earth is forced into the void created by withdrawing the sheeting by means of a water jet. To avoid any damage to buildings, cables, gas mains, water mains, sewers etc., near the excavation or to avoid disturbance to the sewer already laid portions of the sheeting may be left in the trenches.

CHAPTER 8

MAINTENANCE OF SEWERAGE SYSTEMS

Maintenance of sewers in general relates to the work of keeping any installed sewerage facility in a working condition for the benefit of the people for whom it is intended. It may be preventive or routine maintenance which constitutes works executed and precaution taken to prevent any breakdown of sewerage facilities or corrective maintenance which constitutes work of repairs after a breakdown has occurred. Preventive maintenance is more economical and provides for reliability in operation of the sewerage facilities; nevertheless corrective maintenance will also have to be provided for, as breakdowns are possible inspite of the preventive maintenance.

Sewer maintenance functions are too often treated as a necessary evil, to be given attention only as emergency arises. Adequate budgets are seldom provided for supervision, manpower and equipment, unlike the case for maintenance of other utilities like electric cables, telephone cables, gas and water mains. This casual attitude towards sewer maintenance is found even in large cities. In other words, it has not become a "felt need" for the authorities to care for.

All efforts should be made to see that there is no failure in the internal drainage system of a premises. A serious health hazard results when sewage backs up through the plumbing fixtures or into the basements. The householder is confronted with the unpleasant task of cleaning the premises after the sewer line has been cleaned. Extensive property damage may also occur, particularly where expensive appliances are located in the basements.

Maintenance helps to protect the capital investment and ensures an effective and economical expenditure in operating the sewerage facilities. It also helps to build up and maintain cordial relations with the public, whose understanding and support are essential for the success of the facility.

The organisation responsible for the maintenance of the sewerage system will vary with the size and type of the sewerage system and the relative age of the system. The larger the Municipality, the larger and more complex will be its maintenance organization. The size of the organization will vary from a couple of employees to several hundred regular employees. The primary effort of the staff is to maintain sewers free flowing and unobstructed.

The sewer system with its components properly designed and installed is handed over to the person in charge of maintenance who assumes the responsibility to make it function satisfactorily for the benefit of the community. He should have sufficient experience in the design and construction of the system to enable him to perform his task efficiently with an understanding and appreciation of the problems that may arise during maintenance. He has not only to be a technical man but has also to deal with human relations in order to be successful in his work. Inservice training shall be imparted to the maintenance personnel to improve upon the methods adopted based on the latest trends. Failure to develop a better understanding of human relations and also lack of development of the concept of "service to the community" generally results in the maintenance part becoming unpopular.

8.1 PROVISIONS IN DESIGN

Maintenance really begins with the design and construction of the sewerage system. Hence due consideration shall be given to maintenance requirements at the time of designing sewerage systems.

Since sewer maintenance has to commence from manholes which are located in the streets. the size of the manholes must be designed to permit safe access and sufficient working space. The sewers shall be laid at a sufficient grade to provide self cleansing velocity. Inverted siphons should be avoided wherever possible.

In a pumping station for protection against physical injury and bacterial infection, the following are to be observed :

1. Staircases, tanks and other deep openings should be enclosed by suitable guards or railings.
2. Staircases with non-skid treads and hand rails shall be provided in preference to ladders.
3. Machinery should be conveniently located with adequate space for access and operation.
4. All moving parts shall be provided with guards.
5. Proper insulation and earthing of all equipment and wiring should be done.
6. Adequate ventilation, illumination and fire protection measures should be provided.
7. Provision of a continuous safe water supply for personal cleanliness should be ensured.
8. Cross connection between the water supply and sewer lines should be avoided.

9. Where sewage or sludge or gas is collected or where chlorine is stored and handled, due consideration for protection against possible gas hazards should be given while designing the structures.
10. The number and capacity of each pumping unit shall be selected to prevent surcharged conditions in pumping stations, undue detention of sewage in the wet well causing settlement in the well and surcharge conditions of the sewers.

8.2 SEWER MAINTENANCE

It is most essential to protect the sewers and to preserve their capacity by prohibiting discharge into the system of wastes that will damage the system or cause odours or bring about explosive conditions and carry out a regular sewer cleaning programme to prevent and remove obstructions that occur in the course of normal and proper use of the system. The carrying capacity of a sewer is dependent upon its internal diameter. This capacity may be reduced by accumulations or obstructions resulting from the discharge of grease and viscous materials into sewers.

8.2.1. Problems

8.2.1.1 *Clogging of sewers*

The factors responsible for the clogging of sewers may be :

(a) deposition of grit or other detritus which creates stagnation resulting in the putrefaction of organic matter giving rise to odours and poisonous gases; deposition of grease from hot liquid wastes from kitchens finding entry into the sewers, getting cooled and deposited on the sides which in course of time may lead to clogging;

(b) penetration of roots from nearby trees through the joints or cracks in the sewers which eventually choke up the sewers;

(c) growth of fungi which forms a network of tendrils and starts floating, offering an obstruction to the free flow inside the sewers;

(d) deposition of tarry materials assisting in binding the detritus and leading to their growth;

(e) stagnation of sewage in the sewer due to improper working of pumping units leading to settlement of grit and other materials and dumping of solid wastes in the manholes indiscriminately.

8.2.1.2. *Hazards*

Personnel engaged in operation and maintenance of sewerage systems including sewage pumping stations are exposed to different types of occupational hazards like physical injuries, injuries caused by chemicals and radioactive wastes, infections caused by pathogenic organisms in sewage and dangers inherent with explosive or noxious vapours and oxygen deficiency.

The health and safety of personnel can be safeguarded to a great extent by taking the likely hazards into consideration at the time of designing the sewers, sewer appurtenances and pumping stations. Hazards which are still possible inspite of due consideration being given at the design stages, can be reduced by use of safety equipment and precautions appropriate for each hazardous condition. Finally to guard against human error and carelessness, proper job instructions and adequate effective supervision by competent personnel are most essential.

"Sewer gas" is a mixture of gases in sewers and manholes containing abnormally high percentage of carbon dioxide, varying amounts of methane, hydrogen, hydrogen sulphide and low percentages of oxygen caused by septic action through the accumulation of organic matter inside the sewer. The actual hazard is due to the presence of high levels of methane forming an explosive mixture or the oxygen deficiency or hydrogen sulphide in excess of permissible levels. Some times trade wastes may also contribute to other gases like chlorine, ammonia, sulphur-dioxide etc.

The characteristics of common gases encountered in sewers, sewage pumping stations and sewage treatment plants, their physiological effects and safe exposure limits are detailed in Appendix—8.

A noxious gas or vapour is any gas or vapour that is directly or indirectly injurious or destructive to the health or life of human beings. It can be a simple asphyxiant, chemical asphyxiant, irritant, volatile solvent or a combustible gas.

Simple asphyxiants are the physiologically inert gases like nitrogen, methane and hydrogen which when breathed in high concentrations act mechanically by excluding oxygen.

Chemical asphyxiants are substances like carbon monoxide which by combining with the haemoglobin of the blood or with some constituents of the tissues either prevent oxygen from reaching the tissues or prevent the tissues from using it.

Irritants are substances like chlorine which injure the air passage and lungs and induce inflammation in the surface of the respiratory tract.

Volatile solvents and drug-like substances exert little or no effect on the lungs but affect the nervous system inducing anaesthesia. Inorganic and organic metallic poisonous substances in a volatile form prove toxic after their absorption into the body.

Combustible vapours will burn as long as they are in contact with a flame, spark or a heated material having a temperature equal to or greater than the ignition temperature of the gas or vapour and provided there is enough oxygen present for combustion. The reaction of the gas or vapour with the oxygen of the air, resulting in progressive combustion, or propagation of flame, occurs only when the concentration of the gas or vapour is within certain limits expressed as percentage of gas or vapour in air by volume. These limits are called "explosive limits". Outside these limits, local combustion may occur at the source of ignition but there will be no propagation of flame and hence the combustion ceases on removal of the source of ignition. Within the limits, along with the propagation of the flame through the mixture, there will be development of pressure leading to violent explosion.

8.2.2 **Precautions**

8.2.2.1 *Precautions against gas hazards*

When a sewer or a manhole has to be entered for cleaning or clearing an obstruction, it will be advisable to go in for efficient ventilation either natural or forced and get assured that the atmosphere inside the manhole or sewer is free from oxygen deficiency and noxious gases or vapours. Where such clearance of the noxious atmosphere by ventilation is not possible, or time consuming, the following precautions must be taken before entering into the manhole or sewer :

(i) traffic warning signs should be erected;

(ii) no smoking or open flames should be allowed and sparks guarded against;

(iii) only safety, explosion-proof electric lighting equipment or mirrors for reflection of light should be used;

(iv) the atmosphere should be tested for noxious gases and oxygen deficiency;

(v) if the atmoshpere is normal, the worker may enter in the manhole or sewer with safety harness attached with two men available at the top;

(vi) if any noxious gas or oxygen deficiency is found, forced ventilation should be resorted to using a portable blower;

(vii) frequent tests as in item (iv) should be conducted even if the initial tests are satisfactory, as conditions may change during the period the workers are inside the manhole or sewers;

(viii) if forced ventilation is not possible or is not satisfactory and men have to enter urgently, such as in saving a life, a gas mask should be worn and extreme care taken to avoid all source of ignition if inflammable gas is present. Only permissible safety lights (not ordinary flash lights), rubbers or non-sparking shoes and non-sparking tools should be used.

(ix) Only personnel experienced in the line and familiar with the dangers involved in working in a noxious gas environment and fully equipped with the proper protective safety equipment should be allowed to enter.

8.2.2.2. *Precautions against infections*

The personnel working in sewerage maintenance systems are prone to infections and hence the following precautions should be taken.

(i) emergency first aid treatment kits shall be provided to take care of all minor injuries like cuts and burns;

(ii) a physician's services should be available for emergencies;

(iii) the workers should be educated about the hazards of waterborne diseases such as typhoid and cholera through sewage and tetanus through cuts and wounds;

(iv) the importance of personal hygiene should be emphasized and the workers should be instructed to keep finger nails short and well trimmed; not to brush fingers when they are sore; wash hands with soap and hot water before taking food: and to keep fingers out of nose, mouth and eyes, because the hands carry most infections;

(v) use of rubber gloves should be insisted so that sewage or sludge does not come in direct contact with the hands;

(vi) the workers should be provided with a complete change of work clothes to be worn during working hours. Gum boots should also be provided for the workers;

(vii) in laboratory work, only pippettes with rubber teats should be used to prevent contamination of the mouth. Laboratory glassware should not be used for drinking purposes. In no event food should be prepared in the laboratory.

8.2.3. **Safety Equipment**

The various safety equipments that are normally required in sewer maintenance work are gas masks, oxygen breathing apparatus, portable lighting equipment, nonsparking tools, portable air blowers, safety belts and inhalators.

The use of the particular safety equipment is governed by the detection of various gases and oxygen deficiency.

A knowledge of the type of gases in the atmosphere of the working location becomes essential for the selection of the right type of safety equipment. Equipment and simple tests for detection of various gases and oxygen deficiency are furnished in Appendix—9.

8.2.3.1 *Gasmasks*

General purpose gas masks are used for respiratory protection from low and moderately high concentrations of all types of toxic gases and vapours present in the atmosphere in which there is sufficient oxygen to support life. Masks afford necessary respiratory protection under many circumstances but it is most important to know the limitations of the various types available and to be familiar with their use. Even when masks are used properly, other precautions such as never using open flames or creating sparks in the presence of inflammable gases must be taken. The general purpose gas masks affords protection against organic vapours, acid gases, carbon monoxide upto 2 per cent concentration, toxic dusts, fumes and smoke.

The gas mask consists of a face piece, a canister containing purifying chemicals, a timer for showing duration of service and a harness for support. Protection against specific contaminants can be achieved by the selection of appropriate canisters.

Persons using gas masks should practise regularly with them in order to become proficient in putting them on quickly and breathing through them.

Gas masks cannot be used in oxygen deficient atmospheres or in unventilated locations or areas where large concentrations of poisonous gases exist.

8.2.3.2. *Oxygen breathing apparatus*

This is designed for respiratory protection from atmosphere that contains very high concentrations of toxic gases and vapours or that are deficient in oxygen. It fully protects a worker against all gases, vapours, dusts, fumes, smokes and oxygen deficiencies and can be safely used in petroleum vapours and is the most dependable device for work in atmospheres normally encountered in sewage works.

This can be either an air hose respirator or a pure oxygen respirator.

(a) AIR HOSE RESPIRATOR

This is used where a source of fresh air is available within a distance of 50 m from the working location. It is essential that the supply of air is obtained from an uncontaminated source.

It consists of a mask which is a tight fitting face piece attached to a large dia flexible hose, breathing tubes and a harness. Fresh air is blown to the mask through either a power operated or a hand operated blower. An inhalation check valve in the breathing tube assembly and exhalation valve in the face piece permit air flow only in one direction, from the source to the mask, when the blower is in operation. Exhaled air is released into the surrounding atmosphere through the exhalation valve. The valve arrangement permits the wearer to breathe directly through the hose in the event of blower failure. The maximum length of hose will be about 50 m. The hose, being of large diameter, permits breathing without excessive resistance in the event of blower failure. A special pressure release valve on the blower permits regulation of air delivery and a fresh air bypass valve functions automatically in the event of blower failure permitting the wearer to breathe directly through the hose. When a hand operated blower is used the operator will be available to attend to any emergent situation also.

This apparatus does not depend on chemicals and may be used over extended periods at low costs.

(b) PURE OXYGEN RESPIRATOR

This is used where a source of fresh air is not available within 50 m to permit the use of an air hose respirator or in situations where an air hose would encumber the worker.

It consists of a mask, similar to that of the air hose respirator, attached to a steel cylinder or bottle containing oxygen under high pressure, a pressure reducing valve to supply oxygen to the wearer as needed, at slightly higher than normal pressure, a canister that holds chemicals to absorb carbon dioxide from exhaled air and a breathing bag.

8.2.3.3. *Portable lighting equipment*

The equipment normally used are of portable electric hand lamps of permissible types, electric cap lamps and explosion proof flash lights.

8.2.3.4 *Nonsparking tools*

These are made of an alloy (containing at least 80 per cent of copper) that will not spark when struck against other objects and metals and yet retains the necessary strength and resistance to wear.

8.2.3.5. *Portable air blowers*

Forced ventilation of manholes, pits and tanks can be provided by portable air blowers powered by enclosed explosion proof motors. Special precautions should be taken to ensure that the blowers do not serve as a source of ignition for inflammable gases. Such precautions shall include placing of the blower upwind from the manhole or at right angles to the wind direction and at least 2 m away from the opening. The use of such equipment requires a consideration of the depth of the manhole, size of enclosure and the number of openings to uncontaminated atmosphere. Trailer mounted blower having a capacity of 210 m^3/min can ventilate easily many metres of medium sized sewers.

8.2.3.6. *Safety belt*

This consists of a body belt with a buckle and a shoulder harness. The life line is of high grade spliced manila rope or a steel cable anchored with rings, on each side of the belt and provided with safety straps for anchoring or securing to a stable support. The life line should be about 15 m in length and the overall assembly should be capable of withstanding a tensile load of 2000 kg. The safety belt and life line should be tested by lifting the wearer clear of ground before each day's use.

8.2.3.7 *Inhalators*

Approved inhalators employing a mixture of oxygen and carbon dioxide are used for resuscitating victims of gas collapse, drowning or electric shock. Artificial respiration should be started at once on the patient and an inhalator face piece attached to the victim's mouth as soon as the equipment can be made ready. The carbon dioxide used in small percentages stimulates deep breathing so that more oxygen may be inhaled. Pure oxygen should be used only when irritant gases such as hydrogen sulphide or chlorine have caused the victim's collapse.

8.2.4 Sewer Clearing Equipment and Devices

Sewer cleaning work calls for the following equipments and devices like a portable pump set running on either diesel or petrol engine, sectional sewer rods, flexible sewer rods with thick manila rope for manual cleaning, a ferret used in conjunction with a firehose, a sewer cleaning bucket machine, a dredger, a rodding machine with flexible sewer rods and cleaning tool attachments such as augers, corkscrews, hedgehogs and sang cups, scraper, automatic flushing tanks and hydraulically propelled devices such as flush bags, sewer balls, wooden ball and sewer scooters.

8.2.4.1 *Portable pump set*

In cases where sewers are blocked completely and sewage has accumulated in manholes, the collected sewage has to be pumped out to tackle the sewer blockage. Such pumps should be of non-clogging type, preferably on four wheel trailors for tne larger sizes and should be provided with a self priming unit to save time and effort. Small pneumatic pumps can be used where high lifts are required and the volume of liquid to be pumped is not large, such as when pumping out flooded basements and dewatering deep trench excavations.

8.2.4.2 *Sectional sewer rods*

These rods are used for cleaning small sewers. The sewer rods may be of bamboo or teakwood or light metal usually about 1 m long at the end of which is a coupling which remains intact in the sewer but can be easily disjointed in the manhole. Sections of the rods are pushed down the sewer until the obstruction is reached and dislodged. The front or the advancing end of the sewer rod is generally fitted with a cutting edge to cut and dislodge the obstructions. These rods are also useful to locate the obstruction from either manhole in case that particular portion of the sewer has to be exposed for repairing the damages.

8.2.4.3 *Flexible sewer rod*

The flexible rod used in manual cleaning is usually made by sandwiching a manila rope between bamboo strips and tying at short intervals. The flexible rod is introduced first from one manhole to the other, its end being connected to a thicker rope which, when dragged down the sewer, draws out sand and detritus into the downstream manhole. This method is adopted in routine cleaning of sewers.

8.2.4.4 *Ferret used in conjunction with a fire hose*

This is used for breaking and removing sand stoppages. It uses a fire hose connection and produces a small but high velocity stream of water forward (upstream) from a central nozzle and several lower velocity jets to the rear (downstream). The forward stream loosens the accumulated debris ahead of the tool and the rear jets of the ferrets admit water to wash the sand back downstream where it can be removed from the manhole manually. The ferret must be attached to a fire hose of sufficient length to reach at least the next manhole and must be kept in motion to prevent sand from locking the fire hose in the line.

8.2.4.5 *Sewer cleaning bucket machine*

The bucket machine consists of two powered winches with cables in between. In cleaning a section of sewer, the winches are centred over two adjacent manholes. To get the cable from one winch to the other, it is necessary to thread the cable through the sewer line by means of sewer rods. The cable from the drum of each winch is fastened to the barrel on each end of an expansion sewer bucket fitted with closing device, so that the bucket can be pulled in either direction by the machine on the appropriate end. The bucket is pulled into the loosened material in the sewer until the operator feels that it is loaded with debris. The motor is then thrown out of gear and the opposing winch is put into action. When the reverse pull is started, the bucket automatically closes and the dirt is deposited in a truck or a trailer. This operation is repeated until the line is clear. Various bucket sizes are available for sewers of 150 to 900 mm in size. The machine is also used along with other scraping instruments for loosening sludge banks of detritus or cutting roots and dislodging obstructions.

8.2.4.6 *Dredger*

A dredger can be used to clean larger manholes. It consists of a crane and a pulley with the help of which a grab bucket is lowered. This scrapes the bottom deposits and brings it to the ground where the bucket opens and the silt is automatically dropped into a truck or a trailer. The disadvantage in this system is that it cannot clean the corners of the catchpits of manholes. Sometimes the deposits at the corners may become so hard that the same may be required to be chiselled out.

8.2.4.7 *Rodding machine with flexible sewer rods*

This consists of a machine which rotates a flexible rod to which is attached the cleaning tool such as auger, corkscrew or hedgehog and sand cups. The flexible rod consists of a series of steel rods with screw couplings. The flexible rod is guided through the manhole by a bent pipe. The machine rotates the rod with the tool attached to one end, the other being fixed to the machine. The rotating rod is thrust into the bent pipe manually with clamps with long handles holding the rod near the couplings. As the rod is thrust inside, the machine also is drawn towards the manhole. The rod is pulled in and out in quick succession when the tool is engaging the obstruction, so as to dislodge or loosen it. When the obstruction is cleared, the rod is pulled out by means of clamps keeping the rod rotating to facilitate quick and easy removal.

8.2.4.8 *Scraper*

This method is used for sewers of dia larger than 750 mm. The scraper is an assembly of wooden planks of slightly smaller size than the sewer to be cleaned. Where the scrapers cannot be lowered through the opening of a manhole, the scraper has to be assembled inside the manhole. The scraper chains, being attached to a control chain in the manhole where it is lowered, is then connected to a winch on the next downstream manhole by means of chains. The winch is then revolved to push the debris ahead of the scraper. The heading up of the flow behind the scraper will also assist in pushing it in the forward direction. This ensures that the bottom and the sides of the sewer are cleaned thoroughly. The scraped debris is removed manually.

8.2.4.9 *Automatic flushing tanks*

The automatic flushing tanks employ an outside water source with a controlled flow so that the required quantity of water is released at predetermined intervals as discussed in 4.1.2.3.

8.2.4.10 *Hydraulically propelled devices*

The hydraulically propelled devices take advantage of the force of impounded water to effectively clear sewers. Efficiency depends on the hydraulic principle that an increase in velocity in a moving stream is accompanied by a greatly increased ability to move entrained material. The transporting capacity of water varies as the sixth power of its velocity.

(a) FLUSH BAGS

A most effective tool for cleaning portions of sewers where rods cannot be used is the sewer flusher or flush bag. The flusher is a canvas bag or rubber bag equipped with a fire hose coupler at one end and a reducer at the other end. The flusher is connected to the firehose and placed in the downstream end from the point where a choke is located. The bag is allowed to fill up until it expands and seals the sewer. The upstream pressure built up due to this damming effect breaks loose the obstructions. Caution must be exercised in using these types of devices as there is a likelihood of sewage flowing back into the hose connections or breaking of the pipes or joints due to high pressures that may develop.

(b) SEWER BALLS

These are simple elastic pneumatic type rubber balls which can be blown up to varying degrees of inflation. They are manufactured in sizes from 150 to 750 mm dia when fully inflated. When used in cleaning a sewer, the ball is first inflated and then wrapped in a canvas cloth, the edges of which are sewed together. A trial line, little longer than the distance between the manholes, is attached securely to the covering. The size of the ball and the covering shall be such as to fit fairly snugly into the sewer. Immediately the ball is thrust into the sewer, sewage commences to back up in the manhole and continues to rise until such time as its pressure is great enough to force sewage under the ball and moving it downstream through the pipe. Acting as a compressible floating plug, it affords enough obstruction, so that a continuous high velocity jet spurts under and to some extent around the ball, thereby sluicing all the movable material ahead to the next manhole. If the ball encounters an obstruction which is immovable, the ball merely indents to the necessary degree and moves forward. The only fixed obstructions which will stop the forward progress of the ball is a root mass or some similar obstruction tightly wedged into the pipe. Bricks, stones, bottle, loose metal parts, broken pieces of pipes, sand, gravel and settled sludge are easily moved ahead. If the ball stops momentarily, a pull on the trial line is usually sufficient to set it in motion again. If the pipe is very dirty, the trial line can be tied to a step in the upper manhole and the ball's progress can be retarded to the required degree as the lower manhole is reached, thus giving time for complete removal of accumulated silt and debris which has piled up ahead of the ball.

A wooden ball, also called a sewer pill, can also be used for this purpose, particularly for cleaning large outfall sewers. It is dropped into the sewer and owing to its buoyant action rolls along the invert of the sewer. The obstructions caused by it to the flow produces a vigorous scouring action along the invert and the sides which has the effect of removing the growths and the deposits from the sewers. This method is cheap and hence can be used at frequent intervals.

(c) SEWER SCOOTER

This arrangement is an improved version of the scraper and consists of the two jacks, a controlling rope and the scooter with a tight fitting shield. In contrast to the scraper, the scooter completely stops any flow of sewage. The scooter, attached to the control rope, is lowered into the manhole and then into the downstream sewer line. The downstream manhole jack is lowered into place from the road and the upper manhole jack set across the top of the manhole.

When the scooter is introduced into the line, it stops the flow of sewage thus building up a head behind the shield. The resulting pressure causes the scooter to move through the sewer until it accumulates enough debris to stop its movement. The head is then allowed to build up approximately 1m before the control rope is pulled, causing the shield to fold back, thus allowing the accumulated sewage to gush into the sewer downstream, flushing the debris ahead to the next manhole from where it is removed. The control rope is released, clearing the shield against the sewage and causing the scooter to advance again until the debris stops its movement. This process is repeated till the scooter reaches the down stream manhole where it may be removed or allowed to continue through the next section.

Some of the sewer cleaning equipments have been shown in Appendix-10.

8.2.5. **Procedures**

The objective of sewer maintenance is to keep the system operating satisfactorily without breakdown. The system is susceptible to corrosion, erosion, clogging or other deterioration. Maintenance procedures can be divided into preventive or routine maintenance and corrective maintenance or maintenance by necessity.

Preventive or routine maintenance is represented by a systematic programme of cleaning, inspection and repairs that reduces breakdowns and complaints to an absolute minimum. The corrective maintenance refers to the practice of undertaking remedial measures only after the occurrence of breakdowns. Often times, the preventive

Maintenance is neglected and only fire brigade type of measures are adopted. Preventive maintenance would not only reduce the need for corrective maintenance to the minimum but also prolong the life of the system and would fully justify the capital investment. Hence it should invariably be made an integral part of the maintenance programme.

8.2.5.1. *Preventive maintenace*

Many of the causes leading to the clogging of sewers can be prevented by periodic cleaning and removal of silt accumulations in sewer lines including manholes while the system is functioning.

Each of the sewer maintenance gangs shall be under the supervision of a competent person well trained in the use of sewer cleaning and safety equipments and qualified to render first aid service in case of emergencies.

The covers of all manholes immediately upstream and downstream of the particular manholes into which men are to enter should be removed and kept open for at least half an hour, for natural ventilation. Sufficient protective barricades shall be erected on the street to regulate flow or prevent entry of the traffic in the area.

After the lapse of this period, there should be no noxious gas or vapour or oxygen deficiency as evidenced by the tests decribed in Appendix-9. Additional time may be allowed if the foul atmosphere has not cleared up. Forced ventilation shall be resorted to for purging out the foul atmosphere inside using portable blowers. If the forced ventilation is already provided for in the sewerage system, the evacuation of air will be carried out from one end with the manhole covers kept open permitting the entry of outside air into the sewer system. It has to be ensured that the atmosphere inside the manhole is safe before the workers are directed to enter the manholes.

The sewer cleaning equipment used ordinarily is the flexible sewer rod with manila rope. The composite flexible rod composed of rope tied together with bamboo strips is lowered inside the manhole by a person on top, while another man inside the manhole thrusts the same into the sewer in the direction of flow. The man inside the next downstream manhole receives the rod and pushes it out of the manhole assisted by a man on the top. As soon as the end of the rod which may be about 60 m long is thrust into the sewer, it is connected to a thick manila rope which is dragged and kept coiled on the surface near the downstream manhole. As the rope is dragged through the sewer, the silt is drawn out into the downstream manhole where it is collected in buckets and lifted out. This downstream manhole becomes the first upstream manhole for the next stretch of sewer length to be cleaned.

With the cleaning of the sewer other repairs, if any, to the inside of the manhole, the foot steps the cover, etc. should be carried out. In the case of larger conduits through which a man can walk the work will also include the examination of the inner surface of the conduit and carrying out necessary repairs. If a sewer is damaged, it has to be tackled separately and not by routine maintenance personnel.

Based on local needs, a schedule of cleaning of all the sewers in the system has to be prepared and followed. The frequency of cleaning depends on the nature of sewage flowing in a particular section and the velocities obtained in the sewers during the peak flow.

Flushing of sewers from either static or mobile flush tanks may be carried out periodically to clear laterals and sewers laid within sufficient slope for maintaining a velocity so as to dislodge the already settled material at the invert of the sewer and keep it in suspension.

The temporary dam methods of flushing like the flush bags should be adopted when settleable solids in sewage is low so as to keep the deposits to a minimum. This method of flushing and flooding will also help in getting rid of vermin and other pests in manholes and sewers.

8.2.5.2. *Corrective maintenance*

Corrective maintenance becomes necessary for removal of obstructions in sewers caused by excessive silt accumulation or damage leading to the break down of the system with flows much lower than the normal.

The sewer gang for this type of work should consist of specially trained men who are aware of the hazards and capable of coping with situations calling for prompt action. The supervision in this case should be entrusted to a responsible person well-versed in the use of the special sewer cleaning equipment, safety equipment and in first aid.

For locating the exact position of blockages, it is necessary to commence observation from the overflowing manholes down the line until the first manhole with little or no flow is reached. The section between this manhole and the one immediately upstream is the one which should be cleaned first, after taking the necessary precautions for the safety of the workers. The accumulated sewage should be pumped out into the manhole further downstream and the mouth of the sewer exposed. After allowing for natural ventilation for one hour, by keeping all manholes within a radius of 200 m open, tests shall be conducted to ensure that the conditions are satisfactory for the workmen to enter the manhole, resorting to forced ventilation, if necessary. Air from the bottom of the manhole should be sucked out by an exhaust pump which will enable fresh air being drawn into the manhole. This is helpful to evacuate gases heavier than air such as carbon dioxide, petrol vapour and hydrogen sulphide. Sometimes air may be blown inside the manhole which pushes out the foul gases from the manhole. This is very effective for gases which are lighter than air, such as carbon monoxide, methane, nitrogen etc. If the sewage flow into the manhole is heavy and pumping has to be carried out continuously to enable men to get in and work, it is preferable to block the mouth of the sewer on the upstream side and start pumping from the second next upstream manhole while the work of clearing the obstruction in the sewer down below is being carried out.

In the case of simple blockages, the flexible sewer rod of the bamboo and manila rope type will be sufficient. A ferret with a fire hose can also be used for breaking and removing sand stoppages. Where a rodding machine with flexible sewer rods is available, it can be used with suitable tool attachments to break the blockage.

When the above methods are not successful or damage to the sewer is suspected, the location of the blockage can be found by the use of sectional rods from either end of the blocked sewer. Once this is located, the sewer length near the blocks can be exposed by open excavation to examine and set right the sewer line. If the damage to the sewer is extensive and is caused by poor foundation then the stretch between the two manholes may have to be relaid on a proper foundation.

In case it is not possible to wait for proper evacuation of the foul atmosphere inside a manhole or a tank and men have to enter urgently to save a life, then only men trained to work with safety belts, gas masks and other safety equipment may be allowed to enter the manhole, observing the precautions against gas hazards and under the guidance and supervision of a competent supervisor. In an emergency the supervisor should promptly get in touch with the nearest police station, a fire station and a hospital for help.

Where the material of the sewer, especially concrete, is damaged, the cause must be investigated and due rectification carried out. If flows much lower than the designed ones lead to low velocities and consequent septicity resulting in the production of hydrogen sulphide, routine chlorination of the sewage may have to be considered.

If the damage is caused by the indiscriminate discharge of industrial wastes of high acidic or alkaline nature or with high organic matter or solids, steps should be taken to treat the wastes to the standards specified in IS : 3306.

Problems created due to poor design and construction methods which lead to frequent complaints of chokes in the sewers have to be set right by redesign and construction.

8.2.6. **Chemical Treatment**

Control of growth of roots and slimes in sewers can be achieved by application of chemicals. This method may have to be confined to small lines because of the cost of chemicals involved. The use of copper sulphate is successful in the combating of root troubles in sewers and can be fed through water closets or manholes in the sewerage system. A dose of about 10mg/l is generally found to be effective. As copper sulphate is corrosive to metals, care should be taken in its use. It can be placed in bowls of water closets and flushed into the sewer with sufficient water to carry the crystals of copper sulphate into the sewer system.

CHAPTER 9

SEWAGE AND STORM WATER PUMPING STATIONS

9.1 GENERAL CONSIDERATIONS

The availability of land, scope of expansion, arrangement, type of equipment and structure, external appearance and general aesthetics are basic considerations in the design of pumping stations.

9.1.1. Location

Proper location of the pumping station requires a comprehensive study of the area to be served to ensure that the entire area can be adequately drained. Special consideration has to be given in undeveloped or developing areas to probable future growth as location of the pumping station will, in many cases, be determined by the future overall development of the area. The site should also be aesthetically satisfactory. The pumping station shall be located and constructed in such a manner that it will not be flooded at any time. The storm water pumping stations shall be located such that water may be impounded without creating an undue amount of flood damage if the inflow exceeds the pumping station capacity. The station should be easily accessible under all weather conditions.

9.1.2 Capacity

The capacity of the station will be determined by the present and future sewage flows based on a design period of 15 years. However, adequate attention should be given for future expansion, such as provision of additional space for replacing the smaller pumping units by larger ones thus increasing the capacity of the wet well and constructing new pumping stations to cope with the increased flows. The initial flows are generally too small and the effect of the minimum flow should be studied before selecting the size of the pumps for the project to be commissioned, in order to avoid too infrequent pumping operations and long retentions of sewage in wet wells.

9.1.3 Type of Pumping Stations

While the term pumping station covers all types of pumping of sewage or storm water, the term lift station is restricted to lifting of sewage across a very short distance and discharging it to another gravity sewer. Pumping stations are provided with two separate wells; wet wells for receiving the incoming sewage and dry wells for housing the pumps. The wet and dry wells may be of any of the following types :—

(i) rectangular, with dry and wet wells adjacent to each other;

(ii) circular, with central dry well and peripheral wet well; and

(iii) circular with a dividing wall to separate the dry and wet well.

The rectangular type may require thicker walls to withstand the pressures of the lateral soil, subsoil water and sewage and are not recommended except for smaller installations or where the availability of space is the deciding factor.

9.2 PUMP HOUSE STRUCTURE

The structure must be designed to withstand floatation forces. Isolated pumping stations, particularly unmanned, should be protected against vandalism. The site should be adequately protected from flooding.

Dry wells shall have a separate entrance. For easy access to the substructure of the wet pumps, stairs shall be provided in preference to steps. Both the dry and wet wells should be of RCC construction and IS:3370 and IS:4111 (pt. IV) shall be followed for their design and construction. In case of high subsoil water table, sufficient cover, usually 25 mm over the normal requirement, should be provided to safeguard the reinforcement against corrosion.

In many parts of the country, especially in the arid western regions, the ground water contains very high proportions of sulphates leached from the soils which may cause corrosion. Under these conditions, sulphate resistant cement shall be used in concrete. The civil structure of both dry sump and wet wells shall be designed for a flow 30 years hence.

Provision shall be made to facilitate easy removal of pumps and motors for periodic repairs and replacements. This shall be done by providing a gantry of suitable capacity and with suitable travelling type chain and pulley blocks. A dewatering pump of nonclog type shall be fitted at a level about 2 m above the floor of the dry well. Suitable stairways preferably not less than 90 cm in width, for convenient access to dry wells of pumping stations, shall be provided along with railings 90 cm high wherever required.

9.3 WET WELL DESIGN

Storage capacity is generally required for all sewage and storm water pumping stations where automatic controls and variable speed drives are not provided to match pumping rates exactly with inflow rates to the station. The selection of proper storage capacity is critical because it affects :

(a) the time for which the liquid will be retained in the pumping station, and

(b) the frequency of operation of pumping equipment.

The shape of wet well and the detention time provided for sewage stations shall be such that deposition of solids is avoided and sewage does not turn septic. The capacity of the wet well is reckoned between the level at which air affects the suction line of the pump of minimum duty installed in the pumphouse and the designed sewage level in the incoming sewer i.e. the portion of the well below the uppermost starting point and the lowermost stopping point. It is governed by the pumpsets installed to deal with the varying flows. The principle recommended is that any pump should work for at least 5 minutes before it is stopped. The size of the wet well is to be kept such that with any combination of inflow and pumping, the cycle of operation for each pump will not be less than 5 minutes and the maximum detention time in the wet well will not exceed 30 minutes of average flows. In the wet well, baffles should be provided at required places to ensure uniform flow at each pump suction. The wet well flooring shall have a minimum slope of 1:1 to avoid deposition of solids. Dividing walls between the pump suctions may be provided for uniform distribution of sewage flow. Provision for removal of accumulated sludge should be made. Suitable overflow arrangements for the wet well should be provided, where feasible, as a protection against flooding due to breakdown of plant or failure of the power supply.

Wherever possible, grit removal ahead of pumping may be adopted as it would increase the life of pumps.

Coarse screens shall be provided before the wet well with a clear opening of 40 to 50 mm between the bars for manually cleaned type and 25 mm for the mechanical type. The screening units shall always be provided in duplicate. The screens shall conform to IS:6280.

9.4 PUMPS

9.4.1 Capacity

The capacity of the pumps shall be adequate to meet the peak rate of flow with 50% standby. Average flow conditions are of interest in that they indicate the conditions under which the station will usually operate. To obtain the least operating cost, pumping equipment shall be selected to perform efficiently at all flows including peak flow. Two or more pumps are always desirable at sewage pumping stations. The size and the number of units for larger pumping stations shall be so selected that the variations of inflow can be handled by throttling of pumps without starting and stopping of pumps too frequently or necessitating excessive storage.

9.4.2 Protection Against Clogging

Suction and delivery openings of the pumps shall not be less than 100 mm and pump shall be capable of passing a ball of at least 80 mm dia.

9.4.3 Types

Both centrifugal type and pneumatic ejectors are used in sewage or storm water stations, the latter being used only in small installations where centrifugal pumps will be too large for the purpose.

9.4.3.1 *Centrifugal pumps*

The centrifugal pumps can be classified as :

(a) axial flow pumps;

(b) mixed flow pumps; and

(c) radial flow pumps (commonly referred to as centrifugal pumps).

The classification is usually based on its specific speed (Ns) at the point of maximum efficiency. The specific speed of an impeller is defined as the speed, in revolutions per minute at which a geometrically similar impeller would run, if it were of such size to deliver 1 m^3/min against 1 m head. The specific speed Ns, is given by the expression :

$$N_s = \frac{3 \cdot 65\ n}{QH^{0 \cdot 75}}$$

where,

Q=flow in m^3/min

H=head in m and

n=speed in rpm

(a) AXIAL FLOW PUMPS

Axial flow pumps develop most of their head by the propelling action of the impeller vanes on the liquid. They are characterised by a single inlet impeller with the flow entering axially and generally used for large installations with capacities greater than 2,000 m^3/hr and heads less than 9 m. The pumps are generally of vertical type. The axial flow pumps have relatively high specific speeds ranging between 8,000 to 16,000. The vertical units shall have positive submergence of the impeller for proper operation.

(b) MIXED FLOW PUMPS

The head developed by mixed flow pumps is partly by centrifugal action and partly by the lift of the impeller vanes on the liquid. This pump has a single inlet impeller with the flow entering axially and discharging in an axial and radial direction, usually into a volute type casing. They are used for medium heads of 8 to 15 m and for medium to large capacities. The specific speeds of these pumps range between 4200 and 9000. They generally require positive submergence but may be used for limited suction lift.

(c) RADIAL FLOW OR CENTRIFUGAL PUMPS

The head developed in these types of pumps is principally by the action of centrifugal force. Pumps of this type can be obtained with either single suction or double suction inlet impellers, the flow leaving the impeller radially and normal to the shaft axis. As almost any range of head and capacity can be obtained, majority of pumps are of this type. These pumps are characterised by relatively low specific speeds, with single inlet impeller having specific speeds upto 4200 and double suction units having specific speeds less than 6000. Single suction pumps are generally used for sewage and storm water pumping as they are less susceptible to clogging.

The characteristics of the three type of pumps are summarised in the following table:

Characteristic	Axial flow	Mixed flow	Radial flow
Usual capacity range . . .	Greater than 2000 m^3/hr	Greater than 100 m^3/hr	All flows
Head Range	0—9 m	8—15 m	All heads
Shut off head above rated head at max. efficiency point.	About 200%	165%	120 to 140%
Kilowatt characteristics . .	Decreases with capacity	Flat	Increases with capacity
Suction lift	Usually requires submergence.	Usually requires submergence. Short suction lift is permitted.	Usually not over 4·5
Specific speed	8000 to 16000	4200 to 9000	Below 4200 for single suction. Below 6000 for double suction.
Service	Used where space and cost are considerations and load factor is low.	Used where load factor is high and where trash or other solid matter is encountered.	Used where load factor is high and high efficiency & ease of maintenance are desired.

The impellers for centrifugal type pumps may be classified into three categories :

(a) Enclosed (Front and back shroud); and

(b) Open (No shroud); and

(c) Semi-enclosed (back shroud).

Enclosed impellers with shrouds are generally specified for pumps to handle sanitary sewage because they are less subject to clogging and also pass stringy materials better than other types. Open or semi-enclosed impellers are used for storm water pumping installations especially on equipment for handling large capacities and for intermittent service. The impellers for sewage pumps should be self cleaning, able to pass through solids of 100 mm size and capable of dealing with sewage with a specific gravity of 1.1 and containing grit, rags, fibrous material etc.

Pump casing may be either of the volute type, or turbine or diffusion type. The main difference in the two types is that with the volute type casing, the velocity of the liquid leaving the impeller is transformed to head by gradually increasing the area of the liquid passage in the spiral shaped scroll, whereas in the turbine or diffusion type vane casing, the velocity of liquid leaving the impeller is transformed to head by means of curved vanes. Dry sump pumps are generally provided with volute type casing. Vertical wet sump pumps, generally used for storm water pumping installations are usually of the turbine or diffusion vane type. The casing of either type may be split axially or radially to obtain access to impeller for cleaning. Both the pump casing and the impeller of pumps used for sanitary sewage shall be made of close grained cast iron. The pumps shall be erected on common C.I. base plate fixed on adequate size cement concrete foundation.

9.4.3.2 *Pneumatic ejectors*

The pneumatic ejectors are mostly used for lifting sewage from basements of buildings and for small lift stations where their use is advantageous inspite of their low efficiency of about 15%.

The ejectors have the following advantages :

(a) no sewer gases can escape except through the vent shafts as sewage is completely enclosed;

(b) operation is fully automatic and the ejector comes into operation only when needed;

(c) only a few parts are in contact with sewage thus necessitating little attention or lubrication;

(d) ejectors are less susceptible to clogging; and

(e) screening is not required as check valves and connecting lines will pass all the solids that enter the ejector compartment.

A pneumatic ejector consists essentially of a closed chamber into which sewage flows by gravity until it reaches a certain level. Then sufficient air under pressure is admitted into the chamber by float or other type of air vessel to eject the sewage. The nonreturn valve on the inlet pipe prevents sewage from leaving the vessel except through the nonreturn valve on the outlet. This valve on the outlet prevents back flow into the tank. Thus air under pressure displaces the sewage volumetrically until the low water level fixed by the limiting float travel or other control cuts off the air supply. Ejectors shall be installed in duplicate to assure that service is not interrupted if there is a mechanical failure of one unit and during periods when it is necessary to remove equipment for servicing or cleaning.

The quantity of air required to operate the ejector is given by the following formula :

$$V = \frac{Q\ (H+10{\cdot}3)}{12{\cdot}2}$$

where,

V=Volume of free air required in m^3/min;

H=Total head in m; and

Q=Rate of sewage discharge in m^3/min.

The volume of air storage tank and the characteristics of the compressor shall be adequate to provide the necessary volume of air at a pressure at least 40% higher than that required to raise all the sewage to the maximum computed lift.

9.4.4 Head of Pumping

9.4.4.1 *Standard terms*

The various standard terms used to describe the mechanics of pumping equipment include the following :

(a) DATUM

All measurements for suction lift, suction head, total discharge head and net positive suction head are recorded with reference to the datum which is the elevation of the pump centre line in case of horizontal pumps and is the elevation of the entrance eye of the suction impeller in case of vertical pumps.

(b) SUCTION LIFT

Suction lift exists where the total suction head is below atmospheric pressure. Total suction lift is the reading of a liquid manometer at the suction of the pump converted into metres of water referred to datum minus the velocity head at the point of measurement.

(c) SUCTION HEAD

Suction head exists when the total suction head is above atomospheric pressure. Total suction head is the reading of the pressure gauge at the suction of the pump converted into metres of water with references to datum plus the velocity head at the point of measurement.

(d) TOTAL DISCHARGE HEAD

Total discharge head is the reading of the pressure gauge at the discharge of the pump, expressed as metres of water with reference to datum plus the total suction head at the point of measurement.

(e) TOTAL HEAD

Total head is the measure of the energy increase per kg of the liquid imparted to it by the pump and is the algebraic difference between the total discharge head and the total suction head in metres.

(f) NET POSITIVE SUCTION HEAD (NPSH)

The net positive suction head is the total suction head of water in metre absolute, determined at the suction end referred to datum less the vapour pressure of the water in metre absolute.

9.4.4.2 *Oomputation of total head and selection of head*

Pumps having head capacity characteristic which corresponds as nearly as possible to the overall station requirements should be selected. This can best be achieved by preparing a system of head capacity curves showing all conditions of head and capacity under which the pumps shall be required to operate. Friction losses in straight lengths of pipes shall be calculated using Hazen Williams formula and friction losses in valves and fittings as in 5.2 of the companion volume "Manual on Water Supply and Treatment".

Where long pipe lines are involved, it is not possible to predict accurately the total friction loss over an extended period of time. When the line is new, friction losses will be minimum and will increase with use. The friction losses will materially affect the capacity of the pumping units and also their successful operation. System curves shall be prepared to indicate the possible maximum and minimum friction losses expected in the pipeline during the life of the pumping units. Pumps shall be selected having their maxiumm efficiency at average operating conditions. The maximum speed at which a pump shall run is determined by the net positive suction head available at the pump, the quantity of liquid being pumped and the total head.

9.4.5 Miscellaneous Considerations

The proper sizing of piping is a matter of economics. The suction and delivery pipes are designed to have maximum velocities of 1.5 and 2.5 mps respectively. Piping less than 100 mm should not be used for conveying sewage. Valves shall be provided both on suction and delivery side of each pumping unit to allow proper maintenance of pumps. Each pumping unit shall have a separate suction line from the wet well. C.I. Pipes used inside the dry well shall necessarily be of the flanged type. C.I. pipes should never be encased in concrete because failure in flange will be difficult to repair.

9.5 PRIME MOVERS

In deciding the type of prime mover to be used, the following points shall be considered :

(a) low cost,

(b) suitable performance characteristics and

(c) simplicity and ruggedness of construction.

Considering the above, electrical drive is best both for sewage and storm water pumping stations. The constant speed squirrel cage induction motor is generally used.

Although it is a good practice from maintenance point of view to restrict the speed of pump to 720 rpm or 500 rpm for smaller units, the initial cost and replacement facility has to be considered before deviating from the normal speed of 1440 rpm.

A diesel standby unit should be provided in all the pumping stations, except the very small and less important ones, to meet the requirements in case of power failure.

9.6. ELECTRICAL EQUIPMENT

The electrical equipment selected shall be adequate, reliable and safe. The adequacy o the equipment is determined by the continuous current requirements of the station loads and available short circuit capability of the power supply. The reliability of the equipment depends on the capacity of the electrical system to deliver power, when and where it is required, under normal as well as abnormal conditions. Safety involves the protection of the plant personnel as well as the safeguarding of equipment under all conditions of operations and maintenance. None of these three requirements shall be sacrificed for the sake of initial economy. The electrical system shall be designed with enough flexibility to permit one or more components to be taken out of service at any time without interrupting the continuous operation of the station. A proper selection of voltages in the electrical system of pumping station is one of the most important decisions that will affect the overall system characteristics and plant performance. The station bus bar voltage shall be at the level that is most suitable for the pump motors which constitute the major part of the load. The preferred voltage is 440 V from an economy point of view unless other considerations dictate the use of lower voltages.

9.6.1 Switch Gears

The functions of a switch gear in a distribution system include normal and fault switching operations and equipment protection. Motor starting function may sometimes be vested in switchgear but only when the required frequency of starting and stopping is low or in applications where the motors are of such magnitude that no other equipment is suitable.

9.6.2 Motor Starting Equipment

The full voltage type of starting equipment is always preferred from the stand point of cost, floor space and reliability. Where motors are started more than three times per day, the use of power circuit breakers as motor starting equipment may not prove economical in the long run.

9.6.3 Cables

Cables shall be heavy duty, PVC sheathed conforming to IS : 1554 and be of adequate capacity. Cables inside the pump house shall be laid in specially made trenches covered with lean concrete.

9.7 CONTROLS

Controls should be simple, direct and reliable. Large pumping stations may have centralised control systems that automatically start and stop the pump units and associated valves and auxiliaries. A proper hand operated selector switch may also be provided to avoid overworking of any one pumping unit. Liquid level controls generally employ floats, ceramic floats being preferred to metal floats as the latter are affected by chemical action of sewage. All floats are subject to accumulation of grease and scum.

9.8 FLOW MEASURING DEVICES

At all pumping stations, devices such as venturimeter for measuring sewage flow shall be provided. The throat of venturimeter shall be of a metal harder than grit or sand as otherwise the throat gets abraded and gives incorrect readings.

9.9 FUNCTIONAL REQUIREMENTS

9.9.1 Ventilation

The pumping station design shall provide for adequate ventilation throughout. The dry well, if deeper than 4 m below ground level shall be provided with positive ventilation equipment. When the ventilation equipment is of continuous operation type, the minimum capacity shall be 6 turnover per hour. Ventilation design should provide for dissipation of the heat generated from electric motors, especially during hot weather. Wet wells and screen chambers with mechanical equipment shall be provided with positive ventilation equipment to provide 12 turnovers per hour, as this equipment is operated intermittently.

9.9.2 Safety Measures

Railing shall be provided around all manholes and openings where covers may be left open during operation and at other places where there are differences in levels or where there is a danger of the operators falling. Guards shall be provided on and around all mechanical equipments, where the operator may come in contact with belt drives, gears, rotating shafts or other moving parts of the equipment. Staircases with landings shall be provided in preference to ladders particularly for dry well access. Straight staircases shall be provided as against spiral or circular staircases or steps. The steps to be provided in the staircase shall be of a non-slippery type. Telephone is an essential feature in a pump house as it will enable the operator to maintain a regular contact with the main office. In case of injury, fire or equipment difficulty, telephones will provide facility to obtain proper assistance as rapidly as possible.

Fire extinguishers, first aid boxes and other safety devices shall be provided at all the pumping stations.

A system of colours for pipes shall minimise the possibility of cross connections.

To prevent explosive gas leakages, wet well should not be directly connected by any opening to the dry well or superstructure.

All electrical equipment and wiring should be properly insulated and grounded and switches and controls should be of nonsparking type. All wiring and devices in hazardous areas should be explosion proof.

9.9.3 **Other Facilities**

All pumping stations should have potable water supply, wash room and toilet facilities and precautions taken to prevent cross connections.

Hoisting equipment shall be provided at all pumping stations for handling of equipment and materials which cannot be readily lifted or removed from the station by manual labour. In large pumping stations, gantries of adequate capacities shall be provided to lift the pumps, motors and large piping.

Fencing shall be provided around the pumping stations to prevent any trespassing.

The station site shall be landscaped to make it blend with the surrounding areas and add to the aesthetic effect particularly in residential areas.

Adequate lighting is essential at the plinth and all working levels of the pumping stations. Glares and shadows shall be avoided in the vicinity of machinery and floor openings.

PART II

SEWAGE TREATMENT

CHAPTER 10

BASIC DESIGN CONSIDERATIONS

The object of sewage treatment is to stabilize decomposable organic matter present in sewage so as to produce an effluent and sludge which can be disposed of in the environment without causing health hazards or nuisance. Before proceeding with the design of the treatment plant, it is essential to know the variations in quantity and characteristics of the raw sewage and the quality of the final effluent desired.

10.1 DEGREE OF TREATMENT

The degree of treatment will mostly be decided by the regulatory agencies and the extent to which the final products of treatment are to be utilised. These regulatory bodies might have laid down standards for the effluent or might specify the conditions under which the effluent could be discharged into a natural stream, sea or disposed of on land. These regulatory bodies may be the local body or a State Water Pollution Prevention and Control Board. The method of treatment adopted should not only meet the requirements of these regulatory agencies but also result in the maximum use of end products consistent with economy.

10·2 PERIOD OF DESIGN

The treatment plant, like the sewerage system is normally designed to meet the requirements over a 30 year period after its completion. The time lag between the design and the completion should not ordinarily exceed 2 to 3 years and even in exceptional circumstances 5 years. The 30 year period may be modified in regard to specific components of the project depending on their useful life or the facility for carrying out expansions when required so that expenditure far ahead of utility is avoided. Care should be taken to see that the plant is not considerably underloaded in the initial stages, particulary the sedimentation tanks. The comparative merits to cover the full 30 year period versus the first 15 years should be examined to decide on the most economic initial arrangement satisfactory to cover the first 15 years. Even though some mechanical units may not be constructed in the beginning, enough provision should be made in the civil structures for their installation at a later date.

The treatment plant should be considered as a part of the main sewerage project and the area to be served is to be decided based on the needs of the main project itself. The main project may not be executed at one stretch but may be done in stages as the development of the area takes place. But in any case, the ultimate period of design of the project should be 30 years and to that extent sufficient accommodation should be provided for all the units necessary to cater to the needs of this ultimate population. In some cases, it may be necessary to combine a number of sewerage systems with a common sewage treatment plant. This should naturally be considered in the main sewerage project itself and if the treatment plant is to cater to the needs of such additional areas at a later date, enough provision should be made for accommodating the expansion in the beginning itself. Zoning regulations particularly as they affect the use of undeveloped land including the possible changes in the use of lands already developed may be important. It may be planned to combine treatment of the waste from a nearby industry along with the community wastes from the urban area. In such cases, it should be carefully investigated whether liquid wastes from the industry do not adversely affect either the sewerage system or the treatment process and could be accepted, if necessary with some preliminary treatment, so that it could be combined with the sewage from the community for a unified and economical treatment.

10·3 POPULATION SERVED

Estimates for present and future population of areas involved in the project are made to determine the quantity of sewage to be treated. These estimates would have formed a part of the main sewerage project itself as in 3·1·2.

10·4 SEWAGE FLOWS

The quantity of sewage and its characteristics show a marked range of hourly variation and hence peak, average and minimum flows are important considerations. The process loadings in the sewage treatment plant are based on the daily average flows and the average characteristics as determined from a 24 hour weighted composite sample. In the absence of any data, an average flow of 150 lpcd may be adopted. The hydraulic design load varies from component to component of the treatment plant with all appurtenances, conduits, channels, etc. being designed for the maximum flow which may vary from 2·0 to 3·5 times the average flow. Sedimentation tanks are designed on the basis of average flow, while consideration of both maximum and minimum flow is important in the design of screens and grit chambers.

10.4.1 Population Equivalent

The population equivalent is a parameter useful in the conversion of the contribution of wastes from industrial establishments for accepting into the sanitary sewer systems by the authorities concerned and serves as a basis for levying an equitable charge for the same. The average daily per capita contribution of suspended solids and BOD_5 are 90 gms and 45 gms respectively which is used for estimating population equivalents.

10.5 SEWAGE CHARACTERISTICS

Characterisation of wastes is essential for an effective and economical waste management programme. It helps in the choice of treatment methods, deciding the extent of treatment, assessing the beneficial uses of wastes and utilizing the waste purification capacity of natural bodies of water in a planned and controlled manner. While analysis of waste in each particular case is advisable, data from other cities may be utilised during initial stages of planning.

Domestic sewage comprises spent water from kitchen, bathroom, lavatory, etc. The factors which contribute to variations in characteristics of the domestic sewage are daily per capita water use, quality of water supply and the type, condition and extent of sewerage system and habits of the people. Municipal sewage which contains both domestic and industrial waste water may differ from place to place depending upon the type of industries and number of industrial establishments. The important characteristics are discussed below.

10.5.1 Temperature

Observation of temperature of sewage is useful in indicating the solubility of oxygen which affects oxygen transfer capacity of aeration equipment and rate of biological activity. Extremely low temperature affects adversely the efficiency of sedimentation. Normally the temperature of domestic and municipal sewage is slightly higher than that of the water supply.

10.5.2 Hydrogen Ion Concentration

The hydrogen ion concentration more conveniently expressed as pH, is a valuable parameter in the operation of biological units. The pH of fresh domestic sewage is slightly more than that of the water supply to the community. However, the onset of septic conditions may lower the pH while the presence of industrial wastes may produce extreme fluctuations.

10.5.3 Colour and Odour

Fresh domestic sewage has a slightly soapy and earthy odour and cloudy appearance depending upon its concentration. With passage of time, the sewage becomes stale, darkening in colour with a pronounced smell due to microbial activity.

10.5.4 Solids

Though sewage contains only 0.1 percent solids, the rest being water, still the nuisance caused by the solids cannot be overlooked, as they are highly putrescible and therefore need proper disposal. The sewage solids may be classified into suspended and dissolved fractions which may be further subdivided into volatile and non-volatile solids. A knowledge of the volatile or organic fraction of solid which is putrescible becomes necessary as this constitutes the load on biological treatment units or oxygen resources of a stream when sewage is disposed of by dilution. The estimation of suspended solids, both organic and inorganic, gives a general picture of the load on sedimentation and grit removal processes in sewage treatment. Dissolved inorganic fraction is to be considered when sewage is used for land irrigation or reuse of sewage is planned.

10.5.5 Nitrogen

The principal nitrogenous compounds in domestic sewage are proteins, amines, amino-acids and urea. Ammonia nitrogen in sewage results from the bacterial decomposition of these organic constituents. Nitrogen being an essential component of biological protoplasm, its determination in wastes is necessary for proper biological treatment or land irrigation. Where nitrogen content is inadequate, it becomes necessary to supplement with addition of salts containing nitrogen. Generally domestic sewage contains sufficient nitrogen.

10.5.6 Phosphorus

Phosphorus is contributed to domestic sewage from food residues containing phosphorus and their breakdown products. The use of increased quantities of synthetic detergents add substantially to the phosphorus content of sewage. Phosphorus just as nitrogen, is an essential nutrient for biological processes. Generally domestic sewage contains adequate quantities of phosphorus to take care of the needs of the biological treatment.

10.5.7 Chlorides

Concentration of chlorides in sewage above the normal chloride content of water supply is used as an index of the strength of the sewage. The daily contribution of chlorides averages to about 8 gm per person. Based on an average sewage flow of 150 lpcd, this would result in the chloride content of sewage being 50 mg/l higher than that of the water supplied. Any abnormal increase should indicate discharge of chloride bearing wastes or saline ground water infiltration, the latter adding to the sulphates which may lead to excessive generation of hydrogen sulphide.

10.5.8 Biochemical Oxygen Demand

The biochemical oxygen demand (BOD) of sewage or of polluted water is the amount of oxygen required for the biological decomposition of biodegradable organic matter under aerobic conditions. The oxygen consumed in the process is related to the amount of decomposable organic matter. Greater reliance is placed on BOD test as compared to determination of volatile solids when putrescibility of the sewage is to be determined. The standard BOD test is carried out for a period of 5 days at 20° centigrade and is expressed as BOD_5.

10.5.9 Chemical Oxygen Demand

The Chemical Oxygen Demand (COD) test gives a measure of the oxygen required for chemical oxidation. This test does not differentiate between biologically oxidisable and nonoxidisable material. However, the ratio of the COD to BOD does not change significantly for a particular waste and hence this test could be used conveniently for interpreting performance efficiencies of the treatment units. In situations where the presence of toxic materials is likely to interfere with the BOD, this test is very useful.

10.5.10 Toxic Metals and Compounds

Some heavy metals and compounds such as chromium, copper, cyanide, which are toxic, may find their way into municipal sewage through industrial discharges. Determinations of these assume importance if such waste is to be treated by biological processes or disposed of in stream or on land.

10.6 EFFECT OF INDUSTRIAL WASTES

Wastes from industries can form an important component of sewage flows both in volume and composition. It is therefore necessary that detailed data about the nature of the industries, the quantity and the character of the wastes and their variations which may affect the sewerage system or the sewage treatment process are collected. Quantity and character of wastes are to be based on flow measurements and laboratory analyses of the composite samples. Where water reclamation is to be practised due consideration is to be given to the effect of industrial waste components on the final effluent.

Industrial waste containing solids which might clog conduits or damage pumping equipment usually require treatment prior to their entry into the sewer. These substances include ash, cinder, sand, mud, straw shavings, metal, glass, rags, feathers, tar, plastics, wood, hair, fleshings, chemical residues, etc. Condensates, on the other hand, though clear in appearance may contain high dissolved organic and mineral matter adding to the load on the secondary teatment processes and reclamation of water.

In cases where wastes high in suspended solids and BOD are to be accepted, provision should be made in the design of the treatment plant to handle such wastes. In certain instances, it is more economical to tackle the industrial waste at the source itself. Where the wastes have a high or low pH, corrective measures are necessary before admitting them to the sewers or the treatment plant. Toxic metals and chemicals having adverse effects in biological treatment processes or upon fish life in a natural water course where discharged or render the receiving water unfit as a source of water supply, should be brought down to acceptable limits at the source itself. Grease and oils in excessive amounts not only add considerably to the cost of treatment, but also pose a disposal problem. Where additional service charges are imposed by the local authorities for wastes that are overstrength in suspended solids, BOD or other characteristics, the industry may find it desirable to install equipment which will eliminate a considerable proportion of the overstrength characteristics and reduce service charges.

10.7 DUMPING CHUTES FOR NIGHTSOIL

Detachable dumping chutes are constructed at selected points in the sewerage system for disposing of the nightsoil collected manually from individual houses or through vacuum cars from collection wells in towns which are only partially sewered. These discharges have the effect of increasing the concentration of solids and BOD and, where necessary, sufficient dilution must be provided to prevent the clogging of the sewers. In such cases, the treatment plant will have to provide for the change in the characteristics of the sewage. These dumping chutes should be scrapped when the sewerage system is expanded to cover the unsewered areas.

10.8 EFFLUENT DISPOSAL AND UTILISATION

The degree of treatment provided is governed by the specific purpose for which the sewage effluent is used. Sewage effluents should preferably be used for irrigation of crops (sewage farming) with certain precautions. They may also be used for artificial recharge of ground water or for industries as process or cooling water. Another use may be at the plant itself for purposes, such as, flushing and foam control, chlorinator injector water, lawn sprinkling, fire protection (with necessary safeguards) and general plant operation.

If conditions are not suitable for using the effluent for any of these purposes, it may be disposed of in a natural body of water such as sea, river, lake, pond or in irrigation canal observing the necessary precautions.

10.9 CHOICE OF PROCESSES

Sewage treatment processes may be generally classified as primary, secondary and tertiary. The general yardstick of evaluating the performance of sewage treatment plants is the degree of reduction of BOD, SS and Total coliforms. The efficiency of a treatment plant depends not only on proper design and construction but also on good operation and maintenance. Expected efficiencies of various treatment units are given in Table 10.1.

Table 10.1

Process	Percentage Reduction SS	BOD	Total coliform
1	2	3	4
1. Primary Treatment (Sedimentation).	45—60	30—45	40—60
2. Chemical Treatment . .	60—80	45—65	60—90

1	2	3	4
3. Secondary Treatment			
(i) Standard trickling filters	75—85	70—90	80—90
(ii) High rate trickling filters.			
(a) single stage . .	75—85	75—80	80—90
(b) two stage . .	90—95	90—95	90—96
(iii) Activated sludge plants	85—95	85—95	90—96
(iv) (a) Stabilisation ponds (single cell)	80—90	90—95	90—95
(b) -do. (two cell) .	90—95	95—97	95—98

Tertiary treatment is adopted when reuse of effluent for industrial purposes is contermplated or when circumstances dictate the requirement of higher quality effluents.

Cost is the prime consideration in the selection of the treatment method. It should include the cost of installation, capitalised cost of maintenance and operation taking into account the interest charges and period of amortisation. An alternative will be to consider the annual cost covering amortisation and interest charges for the loan obtained for the installation together with the annual operating and maintenance costs. In some cases there is a component of subsidy granted by the Government for the installation of the treatment works and the maintenance cost is borne entirely by the local body or the agency concerned. Both these will have to be taken into account for making realistic comparison of the alternatives.

Other factors that may influence are ease of construction and maintenance, benefits that accrue from better environmental sanitation, location, availability of land and topographical conditions.

CHAPTER 11

PRETREATMENT—SCREENING AND GRIT REMOVAL

Pretreatment consists of separation of floating and suspended organic and inorganic material by physical processes such as (a) screening by which materials larger in size than the openings of the screening device is strained out; and (b) grit removal by which coarse particles of ash and other inert material which have subsipence velocities substantially greater than those of organic putrescible solids are removed.

11.1 SCREENING

Screening is an essential step in sewage treatment for removal of materials which would otherwise damage equipment, interfere with the satisfactory operation of treatment units or equipment or cause objectionable shoreline conditions where disposal into sea is practised. Screens are used ahead of pumping stations, meters and as a first step in all treatment works.

A screen is a device with openings generally of uniform size for removing bigger suspended or floating matter in sewage. The screening element may consist of parallel bars, rods, gratings or wiremeshes or perforated plates and the openings may be of any shape although generally they are circular or rectangular. Screens may be coarse, medium or fine.

11.1.1 Coarse Screens

They serve more as protective devices in contrast to fine screens which function as treatment devices. Coarse screens are usually bar screens and sometimes used in conjunction with comminuting devices.

A bar screen is composed of vertical or inclined bars spaced at equal intervals across a channel through which sewage flows. It is usual to provide a bar screen with relatively large openings of 75 to 150 mm ahead of the pumps for raw sewage while those preceding the primary sedimentation tanks have smaller openings of 50 mm. Bar screens with large openings are often termed coarse racks or trash racks. Their principal function is to prevent the entry of floating matter like logs, timber or large sized material, carcasses, rags, etc., that is brought in by the flowing sewage.

Bar screens are usually hand cleaned and sometimes provided with mechanical devices. These cleaning devices are rakes which periodically sweep the entire screen removing the solids for further processing or disposal. Some mechanical cleaners utilise endless chains or cables to move the rake teeth through the screen openings. Screenings are raked to a platform with perforations which permits the drainage of water back to the unit. Hand cleaned racks are set usually at an angle of 45° to the horizontal to increase the effective cleaning surface and also facilitate the raking operations. Experience indicates that the area of the vertical projections of the space between the bars measured across the direction of the flow should be about twice the areas of the sewer.

Mechanically cleaned racks are generally erected almost vertically. Such bar screens have openings 25% in excess of the cross section of the sewage channel. Their area in usually half of that required for hand raked screens. Fabrication of screens should be such that bolts, cross bars, etc., will not interfere with raking operations. Additional provision should be available for manual raking to take care of the situations where the mechanical rakes are temporarily out of order. Plants using mechanically cleaned screens have controls for (a) manual start and stop, (b) automatic start and stop by clock control, (c) high level switch, (d) high level. alarm, (e) starting switch or overload switch actuated by loss of head, and (f) overload alarm.

11.1.2 Medium Screens

Medium bar screens have clear openings of 20 to 50 mm. Bars are usually 10 mm thick on the upstream side and taper slightly to the downstream side. These mechanically raked units are used before all pumps or treatment units such as the stablization ponds. The bars used for the screens are rectangular in cross-section usually about 10 mm × 50 mm and are placed with the larger dimension parallel to the flow. A weir on the side of the screen may be used as an overflow bypass.

11.1.3 Fine Screens

Fine screens are mechanically cleaned devices using perforated plates, woven wire cloth or very closely spaced bars with clear openings of less than 20 mm. Fine screens are used for pretreatment of industrial wastes to remove materials which tend to produce excessive scum or foam on the top of digestion tank contents. Fine screens are not normally suitable for sewage because of the clogging possibilities.

Fine screens may be of the drum or disc type, mechanically cleaned and continuously operated. Fine screens have generally a net submerged open area of not less than 0·05 m² for every 1000m³ of average daily flow of sewage from a separate system, the corresponding figure being 0·075 m² for combined systems. They are also used for beach protection where sewage without any further treatment is discharged into sea for disposal by dilution.

11.1.4 Comminuting Devices

A comminuting device is a mechanically cleaned screen which incorporates a cutting mechanism that cuts the retained material enabling it to pass along with the sewage. The solids from the comminutor may, however, lead to the production of more scum in the digester.

11.1.5 Location of Screens

Screening devices are usually located where they are readily accessible because the nature of materials handled requires frequent inspection and maintenance of the installation. Where screens are placed in deep pits or channels, it is necessary to provide sufficiently wide approaches from the top and ample working space for easy access and maintenance. Provision should be made for the location of penstocks and bypass arrangements for the screens.

11.1.6 Housing of Screens

The need for a structure to house the screening equipment depends on two factors *viz.*, the design of the equipment and the climatic conditions. If climatic conditions are not severe and could be withstood by the equipment, the screen house can be omitted. Mechanically cleaned screens generally need suitable housing to protect the equipment, prevent accidents to operating personnel and improve the appearance of the treatment facility. Ventilation of the housing is necessary to prevent accumulation of moisture and removal of corrosive atmosphere.

11.1.7 Hydraulics

A screen by its very nature and function collects material which will impede flow. If the screen is cleaned continuously by mechanical arrangement, this interference will be kept to a minimum. Screens with periodic cleaning arrangements are likely to produce considerable damming effect leading to surges of relatively high flow soon after cleaning. The usually accepted design is to place the base of the screen several centimetres below the invert of the approach channel and steepen the grade of the influent conduit immediately preceding the screen.

11.1.8 Velocity

The velocity of flow ahead of and through a screen varies materially and affects its operation. The lower the velocity through the screen, the greater is the amount of screenings that would be removed from sewage. However, the lower the velocity, the greater would be the amount of solids deposited in the channel. Hence, the design velocity should be such as to permit 100% removal of material of certain size without undue depositions. Velocities of 0·6 to 1·2 mps through the open area for the peak flows have been used satisfactorily. When considerable amounts of storm water are to be handled, approach velocities of about 0·8 mps are desirable, to avoid grit deposition at the bottom of the screen, which might otherwise become inoperative when most needed during storm though lower value of 0·6 mps is used in current practice. Further, the velocity at low flows in the approach channel should not be less than 0·3 mps to avoid deposition of solids. A straight channel ahead of the screen insures good velocity distribution across the screen and maximum effectiveness of the device.

11.1.9 Head Loss

Head loss varies with the quantity and nature of screenings allowed to accumulate between cleanings. The head loss created by a clean screen may be calculated by considering the flow and the effective areas of the screen openings, the latter being the sum of the vertical projections of the openings. The head loss through clean flat bar screens is calculated from the following formula:

$$h = 0{\cdot}0729\,(V^2 - v^2) \qquad (11\text{-}1)$$

in which

h = head loss in m,

V = velocity through the screen in mps,

v = velocity before the screen in mps.

Usually accepted practice is to provide a loss of head of 0·15 m but the maximum loss with clogged hand cleaned screen should not exceed 0·3 m. For the mechanically cleaned screen, the head loss is specified by the manufacturers.

11.1.10 Quantity of Screenings

The quantity of screenings varies with the size of screen used and on the nature of sewage. Generally it has been found that the screenings from sanitary sewage vary from 0·0015 m^3/ mL with screen sizes of 10 cm to 0·015 m^3/mL in case of 2·5 cm size.

11.1.11 Disposal of Screenings

The methods of disposal of screenings could be burial, incineration or composting. The screenings should not be left in the open or transported in uncovered conveyors as it would create nuisance due to flies and insects. If conveyors are used, they should be kept as short as possible for sanitary reasons. Burial in trenches usually 7·5 cm to 10 cm deep is practised particularly in small installations. At large works, where sufficient land for burial is not available within a reasonable distance from the plant, screenings are incinerated either by utilising the sludge gas obtained from the digestion tank or by using oil fuel. Where possible, the screenings are transported and mixed with town refuse for production of compost.

11.2 GRIT REMOVAL

Grit removal is necessary to protect the moving mechanical equipment and pump elements from abrasion and accompanying abnormal wear and tear. Removal of grit also reduces the frequency of cleaning of digesters and settling tanks. It is desirable to provide screens or comminuting device ahead of grit chambers to reduce the effect of rags and other large floating materials on the mechanical equipment, in case of mechanised grit chambers. But, where sewers are laid at such depths as to make the location of grit chambers ahead of pumping units undesirable or uneconomical, only a bar screen is provided ahead of pumps, with grit chambers and other units following the pumps.

11.2.1 Composition of Grit

Grit in sewage consists of coarse particles of sand, ash and clinkers, egg shells, bone chips and many inert materials inorganic in nature. Both quality and quantity of grit varies depending upon (a) types of street surfaces encountered, (b) relative areas served, (c) climatic conditions, (d) types of inlets and catch basins, (e) amount of storm water diverted from combined sewers at overflow points, (f) sewer grades, (g) construction and condition of sewer system, (h) ground and ground water characteristics, (j) industrial wastes, (k) relative use of dumping chutes or pail depots where night soil and other solid wastes are admitted to sewers and (l) social habits. The specific gravity of the grit is usually in the range of 2.4 to 2.65. Grit is nonputrescible and possesses a higher hydraulic subsidence value than organic solids. Hence it is possible to separate the gritty material from organic solids by differential sedimentation in a grit chamber.

11.2.2 Types

Grit chambers are of two types mechanically cleaned and manually cleaned. The choice depends on several factors such as the quantity and quality of grit to be handled, headloss requirements, space requirements, topography and economic considerations with respect to both capital and operating costs. In very small plants mechanisation may be uneconomical. But for all sewage treatment plants receiving flows over 10 mld mechanised grit removal units are preferred.

11.2.2.1 *Mechanically Cleaned grit chambers*

These grit chambers are provided with mechanical equipment for collection, elevation and washing of grit which are operated either on a continuous or intermittent basis. Scraper blades or ploughs rotated by a meter drive, collect the grit settled on the floor of the grit chamber. The grit so collected is elevated to the ground level by several mechanisms such as bucket elevators, jet pump, screws and air lift. The grit washing mechanisms are also of several designs most of which are basically agitation devices using either water or air to produce washing action. In intermittently (normally once or twice a day) operated type sufficient storage capacity to hold the grit between intervals of grit elevation should be provided.

11.2.2.2 *Manually cleaned grit chambers*

These should provide for adequate capacity for storage of grit between intervals of cleaning. These tanks should be cleaned at least once a week. The simplest method of removal is by means of shovel and wheel-barrows.

11.2.3 Aerated Grit Chambers

An aerated grit chamber is a special form of grit chamber consisting of a standard spiral flow aeration tank provided with air-diffusion tubes placed on one side of the tank, 0.6 to 1m from the bottom. The grit particles tend to settle down to the bottom of the tank at rates dependent upon the particle size and the bottom velocity of roll of the spiral flow, which in turn is controlled by the rate of air diffusion through the diffuser tubes and the shape of the tank. The heavier grit particles with their higher settling velocities drop down to the floor whereas the lighter organic particles are carried with the roll of the spiral motion and eventually out of the tank. The velocity of roll, however, should not exceed the critical velocity of scour of grit particles. Normally a transverse velocity of flow, not exceeding 0.4 to 0.6 mps at the top of the tank should satisfy this requirement for differential scour. No separate grit washing mechanism or control device for horizontal velocity is necessary in aerated grit chambers.

11.2.4 Design Data

The basic data essential for a rational approach to the design of grit chambers are hourly variations of sewage flow and typical values for minimum, average and peak flows. Since the grit chamber is designed for peak flows and the flowthrough velocity is maintained constant within the range of flow, successful design and operation of grit chamber calls for a fairly accurate estimation of the flows.

The quantity and quality of grit varies from sewage to sewage. Data relating to these two factors is very useful in proper design of grit collecting, elevating and washing mechanisms. In the absence of specific data, grit content may be taken as 0.025 to 0.075 m^3/mL for domestic sewage and 0.06 to 0.12 m^3/mL for combined sewage. The quantity may increase three to four fold during peak flow hours which may last for 1 to 2 hours.

11.2.5 Design of Grit Chambers

11.2.5.1 *Settling velocity or hydraulic subsidence value*

Grit chamber may be designed on a rational basis, by considering it as a sedimentation basin, where discrete particles settle with their own settling velocities. The settling velocity is governed by the size and specific gravities of grit particles to be separated and the viscosity of the sewage. The size of separation based on the minimum size of grit to be removed is 0.20 mm although 0.15 mm is preferred for conditions where considerable amount of ash is likely to be carried in the sewage. The specific gravity of the grit may be as low as 2.4 but for design purposes a value of 2.65 is used. The well known Stokes law on sedimentation expresses the relationship between settling velocity, size and density of particle settled, density and viscosity of liquid and is given by the equations :

$$V_s = \frac{g}{18}\frac{(\rho_s - \rho)}{\mu}d^2 \text{ or } \frac{g}{18}\frac{(S_s-1)}{\gamma}\cdot d \quad \dots(11-2)$$

Where Vs = settling velocity in cm/sec of particle size 'd' cm

g = gravitational constant in cm/sec ^{2},

μ= absolute or dynamic viscosity of the fluid in centipoise†

ρ_s = mass density of particle (gm/cc),

ρ = mass density of liquid (gm/cc),

Ss $=\rho s/\rho$ = specific gravity

$\gamma = \mu/\rho$=kinematic viscosity in centistokes*

† 1 Centipoise = 10^{-2} poise or 10^{-2} (dynes) (sec)/cm ^{2} = 10 ^{2} gm mass/(cm) (sec)

*1 Centistoke = 10^{-2} stoke or 10^{-2} cm^2/sec.

Stokes law holds good for settling of particles of diameters less than 0·1 mm in which case the viscous force dominates over inertial force. This is called "stream line" settling. If the particles settling are larger than 1·0 mm, the nature of settling is called "turbulent" and governed by the Newtons equation which is :

$$V_s = \sqrt{3\cdot 3g.\frac{\rho_s - \rho}{\rho}\cdot d} \dots\dots \quad (11-3)$$

Grit particles lie between 0·1 mm and 1·0mm. The zone of settling corresponding to this range of particles is called "transition zone" of settling. The relationship between settling velocity, size and density of particle, density and temperature of liquid medium in this transition zone is given by Hazen's modified equation :

$$V_s = 60\cdot 6\ (\rho_s - \rho)\ \frac{3T+70}{100}$$

$$= 60.6\ (S_s - 1)\ d.\ \frac{3T+70}{100}$$

where. V_s = settling velocity in cm/sec,

S_s and d are as defined previously and

T = Temperature of liquid in °C

For specific gravities of grit equal to 2·65 and liquid equal to 1 .0,

$$V_s = \frac{60.6(2\cdot 65-1)}{1}.d\frac{(3T+70)}{100} = d(3T+70)\dots(11\cdot 4)$$

The above formula in case of organic solids whose specific gravity is assumed as 1·20 will become

$$V_s = 0\cdot 12\ d\ (3T+70)\dots\dots\dots\dots(11.5)$$

The settling velocity, ie. the rate of subsidence of a particle in a liquid medium is also denoted as Hydraulic Subsidence Value (HSV).

11.2.5.2 *Overflow rate*

Efficiency of grit removal can be expressed as the percentage removal of grit above a specified particle size. A grit chamber designed to remove 100% of grit particles of smallest size would also remove all grit particles larger than this. To obtain a 100% removal of the smallest size particles, it would be theoretically necessary for the detention time in the tank to equal the time required for the minimum sized particles to reach the tank bottom. In other words the conditions should be "ideal" for settling of such particles. It can also be shown that the settling velocity "V_s" of the minimum-sized particle is equal to surface loading rate (Q/A) or overflow rate in order to obtain a theoretical 100% removal of the particles.

The table 11·1 gives settling velocities of different size particles of specific gravity 2.65 (mineral matter) and 1·20 (organic matter) and corresponding overflow rates for 100% removal of these particles based on Hazen's modified equation.

Table 11.1

Settling velocities and overflow rates for grit removal devices

Temp=10°C

Diameter (mm) of particles	Settling velocity cm/Sec		Overflow rate in an ideal grit chamber m³/d/m²	
	Specific 2·65	gravity 1·20	Specific gravity	
			2·65	1·20
0·20	2·0	0·24	1700	210
0·18	1·8	0·22	1600	190
0·15	1·5	0·18	1300	160

These values should be corrected for any other temperature by the factor $\frac{3T+70}{100}$·However, in practice, there are turbulence and short circuiting effects due to several factors such as eddy, wind currents and density currents. Hence these overflow rates should be diminished to account for the basin performance. Generally, values of two thirds to one half are used in design depending on the type of grit chamber. These values are much higher than those needed for organic solids of specific gravity 1·2. The surface area of grit chambers may be worked out based on the reduced loading rates.

11.2.5.3 *Detention period*

A detention period of 60 sec is usually adopted.

11.2.5.4 *Bottom scour and flow through velocity*

Bottom scour is an important factor affecting grit chamber efficiency. The scouring process itself determines the optimum velocity of flow through the unit. This may be explained by the fact that there is a "critical" velocity of flow 'V_c' beyond which particles of a certain size and density once settled, may be again placed in motion and reintroduced into the stream of flow. The critical velocity for scour may be calculated from modified Schield's formula :

$$V_c = 3 \text{ to } 4\cdot 5\sqrt{g.(S_s - 1)\,d} \quad \dots\dots(11-6)$$

For a grit particle size of 0·2 mm, the formula gives critical velocity values of 17·1 to 25·6 cm/sec. In actual practice, a horizontal velocity of flow of 15 to 30 cm/sec is used at peak flows. The horizontal velocity of flow should be maintained constant at other flow rates also to ensure that only organic solids and not the grit are scoured from the bottom. Bottom scour is an important factor particularly affecting the grit chamber efficiency.

11.2.5.5 *Velocity control devices*

Numerous devices have been designed in an attempt to maintain a constant horizontal velocity of flow through grit chambers in the recommended range of 15 to 30 cm/sec. Since none of the control devices designed so far have been able to maintain the velocity at a constant level at all flows, a limit of variance in the velocity of 5-10% above and below the desired velocity of flow is recommended. Multiple channels with the total capacity to carry the maximum flow and velocity control either manually or by means of side-flow-weirs in a distribution box or a diversion channel is sometimes adopted but is not economical. A satisfactory method of controlling velocity of flow through the grit channels is by using a *control* section which placed at the end of the channel, varies the cross sectional areas of flow in the section in direct proportion to the flow.

As for example, for a flow of 5 cumecs, the cross-sectional area of flow should be 5m² and when flow decreases to 3 cumecs the cross-sectional area of flow should be reduced to 3 m² to maintain the velocity of flow constant at 1 mps. *Such control sections include* proportional flow weirs, Sutro weirs, Parshall flumes, Palmer Bowlus flumes etc., of which the former three are commonly used.

(a) PROPORTIONAL FLOW WEIR

The proportional flow weir is a combination of a weir and an orifice. It maintains a nearly constant velocity in the grit channels by varying the cross sectional area of flow through the weir so that the depth is proportional to flow (Fig. 11-1a). The sides are so curved that the area decreases as the three half power of the increasing depth of flow over the weir. Hence the rate of flow over the weir will vary directly as the head over the weir.

Discharge Q in lps over the weir is given by the expression

$$Q = 1570C \sqrt{2g}\ w.\ h^{\frac{1}{2}} h \ldots\ldots\ldots\ldots (11-7)$$

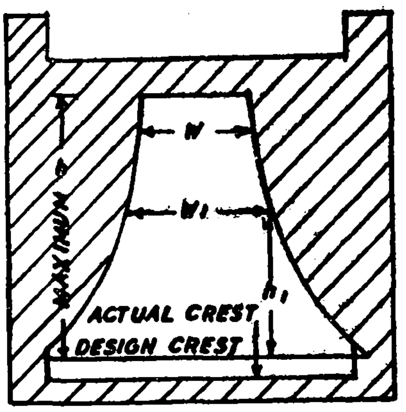
where w is the width of the opening at a height h, in metres and g in m/sec² and C is the discharge coefficient. With a discharge coefficient of 0·6, the expression becomes

$$Q = 4170\ (wh^{1/2})\ h \ldots\ldots\ldots\ldots\ldots\ldots (11-8)$$

Hence depth may be made to vary directly as discharges if $(wh^{1/2})$ is made a constant. For different values of h, corresponding values of w can be determined and hence the parabolic curvature of the sides of weir could be worked out. The simple form of the curve would result in the width of the weir "b" approaching infinity as the head approaches zero. Since this is not practical, the base is limited to a convenient width "b" and for a small height "a" over the crest, the cross section is kept rectangular. The minimum value of vertical height "a" recommended is 25 mm. The weir shall be set 100 mm to 300 mm above the bottom of grit chamber to provide for grit storage or for operation of mechanical grit scraper. The weir should also be set at such an elevation as to provide a free fall into the outlet channel as it cannot function under submerged conditions. Each grit chamber should be provided with a separate control weir.

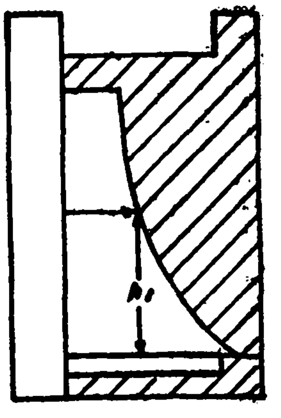
(b) SUTRO WEIR

The Sutro weir is a half proportional flow weir cut symetrically and centrally along the vertical axis as illustrated in Fig. 11-1(b). The orifice has a straight horizontal bottom forming the weir.



PROPORTIONAL FLOW WEIR

FIG. 11·1(a)



SUTRO WEIR

11·1(b)

(c) PARSHALL FLUME

A parshall flume is an open constricted channel which can be used both as a measuring device and also as a velocity control device, more commonly used for the latter purpose in grit chambers. The flume has a distinct advantage over the proportional flow weir, as it involves negligible head loss and can work under submerged conditions upto certain limits. The limits of submergence are 50% in case of 150mm throat width and 70% for wider throat widths upto 1 m. Another advantage is that one control section can be installed for 2 to 3 grit chambers. The flume is also self cleansing and there is no problem of clogging. As the parshall flume is a rectangular control section, the grit chamber above it must be designed to approach a parabolic cross section. However, a rectangular section with a trapezoidal bottom may be used with a parshall flume in which case the variations in velocity at maximum and minimum flow conditions from the designed velocity of flow should be within permissible limits as given by the following equations :

$$Q = 2264W(H_A)^{3/2} \quad \ldots \quad (11-9)$$

$$D+Z = 1.1\, H_A \text{ and} \quad \ldots \quad (11-10)$$

$$\frac{D_{min.}}{D_{max.}} = \frac{1.1\,(Q_{min}/2264\,W)^{2/3} - Z}{1.1\,(Q_{max}/2264\,W)^{2/3} - Z} \quad (11-11)$$

$$D = 1.1 \left[\frac{Q}{2264\,W}\right]^{2/3} - Z \quad (11-12)$$

$$b = \frac{Q\,max}{1000\,D_{max}\,V_{max}} = \frac{Q\,min}{1000\,D_{min.}\,V_{min}} \quad (11-13)$$

$$v = \frac{Q}{1000\,b.\,D.} \quad \ldots \quad (11-14)$$

where

Q = rate of flow in lps;

Q_{min} = minimum rate of flow in lps;

Q_{max} = maximum rate of flow in lps;

W = throat width in m;

H_A = depth of flow in upstream leg of the flume at one-third point in m;

Z = a constant in m;

D = depth of flow in the grit chamber in m;

b = width of grit chamber in m; and

v = velocity of flow in mps at a particular depth of flow.

Recommended throat widths for different ranges of flow alongwith the dimensions of the various elements of the flume (Fig. 11-2) for the different throat widths are given in Table 11-2 which should be strictly adhered to.

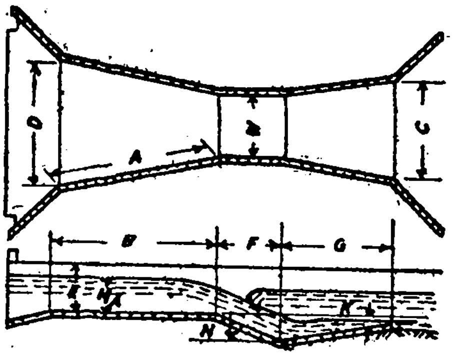


FIG. 11·2 DIMENSIONS FOR PARSHALL FLUME.

Table 11.2

Dimensions of Parshall Flume (mm)

*Flow range Q_{max} (mLd)	W	*A	**B	C	***D	F	G	K	N
Upto 5. . .	75	460	450	175	255	150	300	25	56
5—30 . .	150	610	600	315	391	300	600	75	113
30—45 . .	225	865	850	375	566	300	750	75	113
45—170 . .	300	1350	1322	600	831	600	900	75	225
170—250 . .	450	1425	1397	750	1010	600	900	75	225
250—350 . .	600	1500	1472	900	1188	600	900	75	225
350—500 . .	900	1650	1619	1200	1547	600	900	75	225
500—700 . .	1200	1800	1766	1500	1906	600	900	75	225
700—850 . .	1500	2100	2060	2100	2625	600	900	75	225
850—1400 . .	2400	2400	2353	2700	3344	600	900	75	225

* For average flow and peak factors, see 3.3.1

** Value should be equal to 1·5 x $(Q\ max)^{1/3}$ but not less than those shown in the table.

*** For higher values of B (than shown in the table), the values of D also to be increased to keep D/B ratio same as in Table.

A typical example is shown in Appendix-12.

11.2.5.6 *Number of units*

In case of manually cleaned grit chambers at least two units should be provided. All mechanically cleaned units should be provided with a manually cleaned unit to act as a bypass.

11.2.5.7 *Dimensions of each unit*

The surface areas required for each unit is worked out on the basis of the overflow rate chosen as discussed in *11.2.5.2*. The breadth of tank is fixed with reference to the control device adopted. The length is then worked out on the basis of the selected overflow rate. In case of mechanised grit chambers, the horizontal dimension may be readjusted to suit the standard sizes of the mechanical equipment after ensuring that the flow-through velocity is within the prescribed limits. The depth of flow is determined by the horizontal velocity and the peak flow. Additional depth for storage of grit between intervals of cleaning should be provided in case of intermittent cleaning. A free board of 150 to 300 mm should be provided. Bottom slopes are based on the type of scraper mechanism used.

11.2.6 **Loss of Head**

Loss of head in a grit chamber varies from 0.06 m to 0.6 m depending on the device adopted for velocity control.

11.2.7 **Disposal of Grit**

Clean grit is characterised by the lack of odours. Washed grit may resemble particles of sand and gravel, interspersed with particles of egg shell, and other similar relatively inert materials from the households. Grit washing mechanism has to be included whenever the detention time is more and flow through velocity is less. Unless washed, it may contain considerable amount of organic matter. This becomes an attraction to rodents and insects and is also unsightly and odorous. The grit may be disposed of by dumping or burying or by sanitary land fill. The ultimate method used however depends upon the quantity and characteristics of the grit, availability of land for dumping filling, or burial. In general, unless grit is washed, provision for burial should be made.

CHAPTER 12

SEDIMENTATION AND CHEMICAL TREATMENT

The purpose of sedimentation of sewage is to separate the settleable solids so that the settled sewage if discharged into water courses, does not form sludge banks and when used for irrigation, does not lead to excessive organic loading. Sedimentation of sewage also reduces the organics load on secondary treatment methods. Sedimentation tanks, also called plain sedimentation or primary settling tanks, are located after screens and grit chambers. Secondary sedimentation or secondary settling tanks find use in settling of the effluents from secondary treatment operations like trickling filter or activated sludge where the flocculated solids produced by biological treatment are removed. Intermediate settling tanks are used between primary and secondary settling tanks or remove the flocculated solids in multi-stage biological treatment units.

Septic tanks, Imhoff tanks and clardigesters are combination units where sedimentation and digestion of settleable solids are combined in single units.

12.1 CHARACTERISTICS OF SETTLEABLE SOLIDS

The settleable solids to be removed in sedimentation tanks are mainly organic in nature, dispersed or flocculated. The specific gravity of these suspended solids may vary from 1·01 to 1·20. Generally raw sewage is a dilute heterogeneous suspension of low specific gravity solids ranging from fully dispersed to completely flocculated ones. In primary sedimentation, the bulk of the solids reaching the tank is in a compeletely flocculated stage or susceptible to flocculation due to fluid motion within the tank. Hence laws of sedimentation governing discrete and non-flocculent particles cannot be readily applied for purposes of design. Since the particles are subject to flocculation, the sedimentation tank cannot be designed on the basis of surface area alone but will have to take into consideration displacement or detention time.

12.2 DESIGN CONSIDERATION

12.2.1 Factors Influencing Design

Several factors such as flow variations, density currents, solids concentration, solids loading, area, detention time and overflow rate influence the design and performance of sedimentation tanks. In the design of some plants, only a few of these factors may have significant effect on performance while in others, all of them may play an important role. Sedimentation tanks are designed for average flow conditions. Hence during peak flow periods, the detention period gets reduced with increase in the overflow rate and consequent overloading for a short period. If hourly flow variations are wide as in the case of some industrial waste flows, it may be necessary to build an equalization tank, a head of the treatment units so that uniform-loading is made possible in all treatment units.

12.2.2. Design Criteria

Sedimentation tanks can be designed both for continuous and intermittent operation, the former being economical and requiring negligible head as against the fill and draw tanks which require higher heads. Hence use of fill and draw sedimentation tanks is restricted to small installations in industrial waste treatment only.

As already discussed, both surface loading and detention period are important in the design of sedimentation tanks for sewage as it contains both granular and flocculated particles. Besides, criteria like depth, displacement velocity, inlet and outlet, weir loading and sludge removal have also to receive careful consideration in the design of these tanks.

12.2.2.1 *Overflow rate or surface loading rate*

The overflow rate represents the hydraulic loading per unit surface area of tank in unit time, expressed as $m^3/d/m^2$. Overflow rates must be checked both at average plant flows and peak flow. The overflow rates to be adopted for the different settling tanks are given in Table 12-1. The smaller values in the ranges given are applicable to small plants of capacities less than 5mLd.

12.2.2.2 *Detention period*

The rate of removal of BOD and SS is maximum during the first 2 to 2½ hours of settling and thereafter decreases appreciably. Hence increase in the detention time beyond 2 to 2½ hours will not increase the percentage removal of BOD or SS proportionately. Longer detention beyond 4 hours may affect the tank performance adversely due to setting in of septic conditions, particularly in tropical climates. Experience has shown that a detention period of 2 to 2½ hours for primary settling tanks and 1½ to 2 hours for secondary settling tanks will produce the optimum results. Longer detention may be necessary in case of industrial wastes with or without chemical precipitation.

12.2.2.3 *Weir loading*

Weir loading influences the removal of solids in sedimentation tank, particularly in secondary settling tanks where flocculated solids are settled. There is no positive evidence that weir loading has any significant effect on removal of solids in primary settling tanks. However, certain loading rates based on practice are recommended both for primary as well as secondary tanks. For all primary, intermediate and secondary settling tanks, in all cases, except in the case of secondary tanks for activated sludge process, weir loading not greater than 100 $m^3/d/m$, for average flows is recommended. For secondary settling tanks in activated sludge or its modifications, the weir loading shall not exceed 150$m^3/d/m$. The loading should however ensure uniform withdrawal over the entire periphery of the tank to avoid short circuiting or dead pockets. Peformance of existing sedimentation tanks can be improved by merely increasing their weir length.

This restriction in weir overflow rate requires special outlet weir design including a total weir length several times the tank width, for rectangular tanks and often two weirs with an outlet channel between them for circular tanks. Very long weirs cannot be maintained truly level over their full length, except perhaps at considerable expense; and satisfactory distribution of flow is more readily obtained by forming indentations at regularly intervals such as shallow-V-notches say 50mm deep spaced 0.15 to 0·3 m apart. In addition to the head above the V-notches, a reasonable free fall of 0·05 to 0·15 m should be allowed for maximum flows depending in part, on the total head available.

12.2.2.4 *Depth*

The depth sets the detention time in the settling tank and also influences sludge thickening in secondary settling tanks of activated sludge plants. The depth recommended for horizontal flow tanks are given in Table 12-1. In vertical flow tanks, depth may be 2·0 m excluding hoppers.

12.2.2.5 *Sludge removal*

Sludge can be removed manually, hydrostatically or mechanically from the sedimentation tanks. Mechanical cleaning of sludge should be preferred to manual cleaning even in small plants, where power is available for running the plant machinery. Even where power is not available or inadequate or exorbitantly costly, hydrostatic removal should be adopted to avoid manual handling of sludge to prevent exposure of workers to health hazards.

Manual removal requires the tank to be put out of commission for dewatering. Sludge is then flushed by a jet of water into sumps. Workers may also have to enter the tank and push the sludge into sumps by means of brushes and squeeges. The sludge collected in the sumps is withdrawn from the tank by gravity, hydrostatic pressure or pumping. The slope of the tank floors should be gentle, not more than 1 to 2 percent, towards the sump for men to walk on the floor. Tank capacity should also provide for storage of sludge between intervals of cleaning not exceeding 7 days in tropical climates. Manual cleaning has been largely given up in favour of mechanical cleaning in modern practice.

Tanks are provided with hopper bottoms for hydrostatic sludge removal. Generally horizontal flow tanks are provided with rectangular hoppers and vertical tanks with circular or square types. Side slopes of the hoppers should be of the order of 1.2:1 to 2:1 preferably with values greater than 1.7:1 and 1·5 :1 for pyramidal and conical hoppers respectively. The floor of the hoppers should not be wider than 0·6 m.

Mechanical sludge scraping is best suited for circular or square tanks and occasionally adopted in rectangular tanks. The scrapers or ploughs push the sludge along the tank bottom to sludge collecting channel or pocket from where it is either pumped directly or gravitated to a sludge sump for further disposal.

In rectangular tanks, sludge hoppers are generally placed at the inlet end. But they may be placed at mid-length in long tanks or at the outlet end in case of secondary settling tank of activated sludge plant. The sludge scraping mechanism may be of a moving bridge type or flight scrapers mounted on endless chain conveyors. The linear conveyor speed should not exceed 0.010 to 0.015 mps. In case of flight scrapers, the maximum width of tank is greater than twice the depth. Multiple flight scrapers are placed side by side, in which case the width of tank could be increased upto a maximum 30 m. When multiple flight scrapers are used, the receiving sludge hoppers is designed as a trough with transverse collectors to convey the sludge to a single outlet pocket. A bottom slope of 1% is recommended for mechanical scraping of sludge.

The most common type of sludge scraping in circular or square tanks consists of a revolving mechanism with radial arms having ploughs or blades set at an angle just above floor level. The ploughs push the sludge to a central hopper as the arms are rotated. Sludge from the central hopper is removed to a sludge sump by the side of the tank from where it is pumped. For small dia upto 9 m the revolving bridge is spanned across the tank dia while for larger sizes it is supported on the tank wall on one side and on a hollow pillar at the centre of the tank on the other side which also serves as an inlet. Drive motors can be either stationary or movable in the case of traction drive and are placed above the tank.

The rotating mechanism of the sludge scraper for square tanks is similar to that of circular tanks except for additional pivoted corner blades for removing sludges from the corners. All rotary mechanisms are operated at a low speed of 1 to 2 rph.

The interval between sludge removal should be preferably less than 4 hours and never exceed 12 hours. Light flocculent sludges such as the activated sludge or mixture of activated sludge and primary sludge are scraped and removed continuously from the tank to avoid septicity. The paripheral speed of the scraper should be between 2·5 to 4 cm/sec.

Where sludge is removed intermittently with intervals larger than 4 hrs, provision for sludge storage in the hoppers should be made. Sludge conveyor pipes should not be less than 200 mm in dia. Hopper volumes should be excluded from the effective sedimentation volume of the tank.

12.2.2.6 *Inlets and Outlets*

Performance of sedimentation tanks is very much influenced by inlet devices which are intended to distribute and draw the flow evenly across the basin. All inlets must be designed to keep down the entrance velocity to prevent formation of eddy or inertial currents in the tank to avoid short circuiting. Design should ensure least interference with the settling zone to promote ideal settling conditions. Choice of inlet and outlet design depends on the geometry of sedimentation tank and the mode of entry and exit from the tank.

In horizontal flow rectangular tanks, inlets and outlets are placed opposite each other separated by the length of tank with the inlet perpendicular to the direction of flow.

In the design of inlets to rectangular tanks the following methods are used to distribute the flow uniformly across the tank:

(a) Multiple pipe inlets with baffle boards of depth 0·45 to 0·6 m in front of the inlets, 0·6 to 0·9 m away from it, and with the top of baffle being 25mm below water surface for the scum to pass over;

(b) channel inlet with perforated baffle side wall between the tank and the channels, or

(c) inlet channel with submerged weirs discharging into tank followed by a baffle board inside the tank.

A stilling chamber is necessary ahead of inlets if the sewage is received under pressure from pumping mains.

Outlet is generally an overflow weir located near the effluent end, preferably adjustable for maintaining the weir at a constant level. V-notches are provided on the weir to provide for uniform distribution of flow at low heads of discharge over the weir. Weir lengths could be increased by placing outlet channel inside the tank with weirs on both sides. Scum baffles are provided ahead of outlet devices to prevent the escape of scum with the effluent.

In radial flow circular tanks the usual practice is to provide a central inlet and a peripheral outlet. The central inlet pipe may be either a submerged horizontal pipe from wall to centre or an inverted siphon laid beneath the tank floor. An inlet baffle is placed concentric to the pipe mouth generally with a diameter of 10–20% of the tank diameter and extending 1 to 2 m below water surface. Where the inlet pipe discharges into a central hollow pillar, the top of the pillar is flared to provide adequate number of inlet diffusion ports through which sewage enters the tank with an entry velocity of 0·10 to 0·25 mps through the ports. The entry ports are submerged 0·3 to 0·6 m below water surface.

The outlet is generally a peripheral weir discharging freely into a peripheral channel. The crest of the weir is provided with V-notches for uniform draw off at low flows. In all primary settling tanks a peripheral scum baffle extending 0·20 to 0.30 m below water surface is provided ahead of effluent weir. If the length of the peripheral weir is not adequate, a weir trough mounted on wall brackets near the periphery with adjustable overflow weir on both sides is provided to increase the length of weir.

12.2.2.7 *Scum removal*

One distinct feature of primary settling tank is the skimming device which, though desirable is not normally provided in intermediate or secondary settling tanks. The skimming device could be operated by the same scraper mechanism used for sludge scraping at the bottom of the tank. It generally consists of a skimmer arm to which a scraper blade is attached and moved, partly submerged and partly projecting above the water surface, from the outlet end towards the inlet end in case of rectangular tanks or in a circular path in the case of circular tanks. The floating scum is thus collected at the forward end of the scraper blade and moved till it is tripped manually or automatically into a scum trough which discharges the scum to a sump outside the tank from where it is removed for burial, burning or feeding to the digester. A scum baffle at least 0·15 m above and extending to at least 0·30 m below water level is provided along the periphery, ahead of outlet device, to prevent the escape of scum with effluent.

12.2.2.8 *Types and shapes*

Circular tanks are more common than rectangular or square tanks. Upflow tanks have been used for sewage sedimentation but horizontal flow types are more popular. Rectangular tanks need less space than circular tanks and could be more economically designed where multiple units are to be constructed in a large plant. They can form a more compact layout with the rectangular secondary treatment units such as aeration tanks in the activated sludge system.

For rectangular tanks, maximum length and widths of 90 and 30 m respectively with length to width ratios of 1·5 to 7·5 and length to depth ratios of 5 to 25 are recommended. A minimum depth of 2 m in case of primary settling tanks and 2·5 m in case of secondary settling tanks for activated sludge should be provided. Bottom slopes of 1% are normally adopted. Peak velocities greater than 1·5 mph should be avoided.

Diameters of circular tanks vary widely from 3 to 60 m although the most common range is 12 to 30 m. Diameters and depths could be chosen at the discretion of the designer in conformity with the manufactured sizes of scraper mechanisms in the country. The water depth varies from 2m for primary to 3·5m for secondary settling tanks. Floors are sloped from periphery to centre at a rate of 7·5 to 10%. The inlet to the tank is generally at the centre and the outlet is a peripheral weir, the flow being radial and horizontal from centre to the periphery of the tank. Multiple units are arranged in pairs with feed from a central control chamber.

12.3 PERFORMANCE

Primary sedimentation of domestic sewage may be expected to accomplish 30 to 45% removal of BOD and 45 to 60% removal of SS depending on concentration and characteristics of solids in suspension. Secondary settling tanks, if considered independently, remove a very high percentage of flocculated solids, even more than 99%, particularly following an activated sludge unit where a high mixed liquor suspended solids is maintained in the aeration chamber. However, the efficiency of secondary biological treatment process is always defined in terms of the combined efficiency of the treatment units and its secondary settling tank with reference to the characteristics of the incoming sewage.

12.4 CHEMICAL PRECIPITATION

Chemical precipitation of sewage or industrial waste is analogous to coagulation in water purification. Certain salts of heavy metals like iron and aluminium when added to sewage containing alkaline substances develop heavy precipitates which bring down with them colloidal suspensions. This phenomenon may be due to mere mechanical entrainment of colloidal particles or due to neutralisation of colloids due to large surface area presented by the precipitates. However, the exact mechanism by which the colloidal suspensions are removed is still not fully understood. The method produces intermediate results between plain sedimentation and secondary treatment. With proper dosages of chemicals this treatment method may be expected to remove 60 to 80% of SS and 45 to 65% of BOD when it is not preceded by any plain sedimentation.

Chemical precipitation will not, however remove dissolved solids. On the contrary they may add to the dissolved solids content of sewage due to addition of chemicals.

As compared to secondary biological treatment methods such as trickling filter or activated sludge, chemical treatment will be less efficient and will work out uneconomical and is therefore recommended only when

(i) seasonal variations in strength and volume of sewage is high;

(ii) intermediate treatment between plain sedimentation and secondary biological treatment is adequate;

(iii) suspended solids from industrial wastes, which are not amenable for biological treatment are to be precipitated out; and

(iv) sludge conditioning for dewatering is needed.

The main advantage of this method is its suitability for seasonal and batch operation. However, the large volumes of sludge (2 to 3 times) produced pose serious disposal problems.

12.4.1 **Chemicals Used**

The most commonly used chemicals are ferrous sulphate; ferric chloride, ferric sulphate chlorinated copperas, alum, aluminium chloride, lime and sodium carbonate. Choice of chemical and its dosages depends on cost of chemical degree of treatment required and the characteristics of the waste, pH being one of the more important factors. Optimum dosages is determined by conducting precipitation tests in the laboratory.

12.4.1.1 *Iron salts*

Ferric salts are better coagulants than ferrous salts because of their higher valency and their efficiency over a wider pH range. Ferric salts are effective at all pH values above 3, the efficiency increasing with increase in pH, while the useful pH range of ferrous salts is above 10. But when waste waters are highly alkaline due to presence of trade wastes, it may be cheaper to use larger dosage of ferrous salts as they are relatively cheaper. Chlorinated copperas which is an equimolecular mixture of ferric sulphate and ferric chloride formed by the addition of chlorine to ferrous sulphate is also used in place of ferric salts.

12.4.1.2 *Aluminium salts*

Aluminium chloride and sulphate of alumina (filter alum) are the commonly used aluminium salts. Where alum is used, the sludge produced is greater in volume and also bulky than with iron salts making it less easily settleable.

12.4.1.3 *Lime and sodium carbonate*

These are used for pH adjustment to favourable ranges of coagulants especially when sewage is highly acidic. Lime is sometimes used independently as a precipitant, particularly when iron pickling liquors are present in sewage. The action may be due to formation of calcium carbonate floc or reactions with small amounts of aluminium or iron salts present in sewage. Lime incidentally helps in grit separation, oil and grease removal and is perhaps the cheapest chemical used in chemical precipitation.

12.4.2 **Unit Operations**

The process consists of the three unit operations viz., proportioning and mixing of chemicals, flocculation and sedimentation.

12.4.2.1 *Mixing*

The required dose of chemical is weighed and fed to sewage by means of proportioning and feeding devices, ahead of the mixing unit. Mixing is accomplished in a rapid or flash mixing unit provided with paddles, propellers or by diffused air and having detention period of 0·5 to 3 minutes. The paddles or propellers are mounted on a vertical shaft and driven by a constant speed motor through reduction gears. The size and speed of the propeller is so selected as to give a propeller capacity of twice the maximum flow through the tank. The shaft speed is generally of the order of 100—120 rpm and power requirement is about 0·1 kw/mLd.

12.4.2.2 *Flocculation*

The principle of flocculation in sewage is similar to flocculation in water purification. The floccules that are formed after flash mixing with chemicals are made to coalesce into bigger sizes by either air flocculation or mechanical flocculation. Both diffused air and mechanical vertical draft tube are used for air flocculation. Revolving paddle type is the most common of the mechanical flocculators. The tanks are usually in duplicate with a detention period of 30--90 minutes depending upon results required and the type of sewage treated. However, the dose of chemical required as well as the flocculation period are best determined by laboratory test followed by pilot plant studies for optimum results. The paddles are mounted either on a horizontal or vertical shaft. The peripheral speed of the paddles is kept in the range of 0·3 to 0·45 mps. The flow-through velocity through the flocculator should be in the range of 15 to 25 cm/sec to prevent sedimentation.

In case of domestic sewage and certain industrial wastes, mechanical flocculation without addition of chemicals will induce self flocculation of the finely divided suspended solids and hence increase the efficiency of sedimentation.

12.4.2.3 *Sedimentation*

The flocculated sewage solids are settled out in a subsequent sedimentation tank. The design features of these tanks are similar to secondary settling tanks as discussed in 12.2.2. Usually detention period of 2 hrs and an overflow rate of not more than $50 m^3/d/m^2$ for average flows is adopted in the design of these sedimentation tanks.

Table 12.1

Design Parameters for settling Tank Overflow rate $m^3/d/m^2$

	Average	Peak	Depth in m
Primary settling only	25—30	50—60	3·0—3·5
Primary settling followed by secondary treatment.	35—50	80—125	3·0—3·5
Primary settling with activated sludge Return.	25—35	50—60	3·5—4·5
Secondary settling for trickling filter.	10—25	40—50	3·0—3·5
Secondary settling for Activated Sludge (Excluding Extended Aeration).	15—35	40—50	3·5—4·5
Secondary settling for Extended Aeration	8—15	35	3·5—4·5

In the case of secondary clarifiers following trickling filters, the design overflow rate must include recirculated flows when clarified effluent is used for recirculation. In the case of activated sludge plants, however, the design overflow rate is to be applied only to the sewage flow and not to the mixed liquor flow to the settling tank. Secondary settling tank of activated sludge plants must be designed not only for hydraulic overflow rates but also for solids loading rates so that adequate sludge thickening may take place and there may be concentrated sludge return. Solids loading rates (based on MLSS i.e. mixed liquor flow to the settling tanks) must be in the range 100—150 $kg/d/m^2$ for conditions of average flow and should not exceed 250 $kg/d/m^2$ for peak flow conditions. The surface area for activated sludge settling tanks is to be designed for both overflow rate and solids loading rate and the larger value adopted.

CHAPTER 13

ACTIVATED SLUDGE PROCESS

The Activated Sludge Process is an aerobic, biological sewage treatment system. The essential units of the process are an aeration tank, a secondary settling tank, a sludge return line from the secondary settling tank to the aeration tank and an excess sludge waste line.

13.1 PROCESS MECHANISM

In the activated sludge process, raw sewage or more usually settled sewage, is aerated in an aeration tank for a period of some hours. During the aeration, the micro-organisms in the sewage multiply by assimilating part of the influent organic matter; in this process, part of the organic matter is synthesised into new cells and part is oxidised to derive energy. The synthesis reaction, followed by subsequent separation of the resulting biological mass and the oxidation reaction are the main mechanisms of BOD removal in the activated sludge process. The BOD removal is evaluated based on the BOD_5 of the aeration tank influent and the BOD_5 of the final effluent after sludge separation. The biological mass generated in the aeration tank consists of zoogleal bacteria, protozoa rotifers, etc. The biomass is generally flocculent and quick settling. It is separated from the aerated sewage in a secondary settling tank and is recycled continuously to the aeration tank as an essential feature of the process. The mixture of recycled sludge and sewage in the aeration tank is referred to as 'mixed liquor'. The recycling of sludge helps in the initial build up of a high concentration of active micro-organisms in the mixed liquor which accelerates BOD removal. Once the required concentration of micro-organisms in the mixed liquor has been reached, its further increase is prevented by regulating the quantity of sludge recycled and wasting the excess from the system.

The mixed liquor suspended solids (MLSS) content is generally taken as an index of the mass of active micro-organisms in the aeration tank. However, the MLSS will contain not only active micro-organisms but also their dead cells as well as inert organic and inorganic matter derived from the influent sewage. The mixed liquor volatile suspended solids (MLVSS) value is also used and is preferable to MLSS as it eliminates the effect of inorganic matter.

13.2 PROCESS VARIABLES

The main variables of the activated sludge process are the loading rate, the mixing regime and the flow scheme.

13.2.1 **Loading Rate**

The loading rate expresses the rate at which sewage is applied in the aeration tank. A loading parameter that has been developed empirically over the years is the hydraulic retention time (HRT) which is expressed as follows :

$$\text{HRT(Hours)} = \frac{V}{Q \times 1000} \times 24 \quad \ldots \quad (13\text{-}1)$$

where V = Volume of aeration tank, m^3

Q = Sewage inflow, mLd (sludge recycle excluded)

Another empirical loading parameter is volumetric loading which is defined as the BOD_5 applied per unit volume of aeration tank or

$$\text{Volumetric loading, kg } BOD_5/m^3 = \frac{Q \times La}{V} \quad \ldots (13\text{-}2)$$

where La = influent BOD_5 to aeration tank, mg/l.

A rational loading parameter is the organic loading rate which is also referred to as food to microorganisms ratio (F/M). The organic loading rate is defined as the ratio of kg BOD_5 applied per day (*representing microbial feed*) to kg. MLSS in aeration tank (representing micro-organisms) or

$$F/M = \frac{Q \times La}{(V/1000).\ X_t} \quad \ldots \quad (13\text{-}3)$$

where X_t = MLSS, mg/l as before. The F/M ratio is the main factor controlling BOD removal. Lower the F/M value, the higher will be the BOD removal in the plant. The F/M can be varied by varying the MLSS concentration in the aeration tank.

A third parameter that could be used for checking the design of activated sludge systems is the Solids Retention Time (SRT) also known as Mean Cell Residence time (MCRT) or the sludge Age which is given by

$$\frac{\text{kg MLSS under aeration}}{\text{(Kg SS wasted + kg SS lost in the final effluent) per day}} \quad \ldots (13\text{-}4)$$

Typical values of loading parameters for various activated sludge Systems is furnished in Table 13.1

13.2.2 **Mixing Regime**

The mixing regime employed in the aeration tank may be plug flow or completely mixed flow.

Plug flow implies that the sewage moves down progressively along the aeration tank, essentially unmixed with the rest of the tank contents. Complete mix flow involves the rapid dispersal of the incoming sewage throughout the tank. In the plug flow system, the F/M and the oxygen demand will be highest at the inlet end of the aeration tank and will then progressively decrease. In the complete mix system, the F/M and the oxygen demand will be uniform throughout the tank.

13.2.3 **Flow Scheme**

The flow scheme involves the pattern of sewage addition and sludge return to the aeration tank and also the pattern of aeration. Sewage addition may be at a single point at the inlet end of the tank or it may be at several points along the aeration tank. The sludge return may be directly from the settling tank to the aeration tank or through a sludge reaeration tank. Air may be applied uniformly along the whole length of the tank or it may be tapered from the head of the aeration tank to its end.

13.3 CONVENTIONAL SYSTEM AND MODIFICATIONS

The conventional system represents the early development of the activated sludge process. Over the years, several modifications to the conventional system have been developed to meet specific treatment objectives by modifying the process variables discussed in 13.2.

13.3.1 **Conventional System**

The Conventional system is always preceded by primary settling. The plant itself consists of an aeration tank, a secondary settling tank, a sludge return line and an excess sludge waste line (Fig. 13-1(a)).

The HRT, volumetric loading and F/M usually adopted for the conventional system are given in Table 13-1. The BOD removal in the process is 85—95 percent. The plant employs a plug flow regime which is achieved by a long and narrow configuration of the aeration tank with length equal to 5 to 50 times the width. The sewage and mixed liquor are let in at the head of the tank and withdrawn at its end. Because of the plug flow regime, the oxygen demand at the head of the aeration tank is high and then tapers down. However, air is supplied in the process at a uniform rate along the length of the tank. This leads to either oxygen deficiency in the initial zone or wasteful application of air in the subsequent reaches.

The main limitation of the conventional system is that the aeration tank volume requirement is high. Further, there is a lack of operational stability at times of excessive variation in rate of inflow and in influent strength. For historical reasons, the conventional system is the most widely used type of the activated sludge process. Capacities of plants in use range from 5 mLd to above 1000 mLd.

13.3.2 **Tapered Aeration**

Tapered aeration involves a minor modification of the conventional system in that the air is supplied at a high rate at the head of the aeration tank to meet the high oxygen demand there and then gradually reduced to match the reducing oxygen demand. The main advantage of tapered aeration is the optimal application of air. The method lends itself readily for the upgrading of conventional plants employing diffused air aeration.

13.3.3 **Step Aeration**

In step aeration, the settled sewage is introduced along the length of the aeration tank in several steps while the return sludge is introduced at the head (Fig 13 1(b)). The loading criteria adopted for the system are shown in Table 13.1 The MLSS concentration decreases at each point of sewage addition and the F/M ratio is calculated based on the average MLSS concentration in the tank. Because of the step addition of sewage, the oxygen demand in the system is fairly uniformly spread over the length of the aeration tank and the air supplied uniformly as in the conventional system is efficiently used. The process enables an appreciable reduction in the aeration tank volume without lowering the BOD removal efficiency Step aeration method has considerable capacity to absorb shock organic leadings. The method has found application for larger plants of capacities upto about 1000 mLd.

13.3.4 **Contact Stabilisation**

The process is also called biosorption. In this process sewage may be treated either after primary settling or without primary settling. The sewage is aerated along with return sludge for a comparatively short period of 0.5—1.5 hrs. when the sludge adsorbs the organic matter in the sewage. The mixed liquor is then settled in a secondary settling tank. The return fraction of the sludge withdrawn from the settling tank is reaerated in a separate sludge reaeration tank for a period of 3—6 hrs. before it is fed back into the contact aeration tank (Fig 13—1(c)). During the sludge reaeration, the adsorbed organics are stabilised restoring the adsorptive capacity of the sludge.

The loading criteria for the process are given in Table 13—1. It is to be noted that in determining F/M for the process, both the SS in the contact aeration tank and the SS in the sludge reaeration tank are to be taken into account.

The contact stabilisation process is quite effective in the removal of colloidal and suspended organic matter, but it is not very effective in removing soluble organics. The method is well suited for the treatment of fresh domestic sewage containing only a low percentage of soluble BOD. Compared to the conventional system, the contact stabilisation process has greater capacity to handle shock organic loadings because of the biological buffering capacity of the sludge reaeration tank. The process also, presents greater resistance to toxic substances in the sewage as the biological

SCHEMATIC DIAGRAMS OF ACTIVED SLUDGE TREATMENT WITH DIFFERENT MODIFICATION

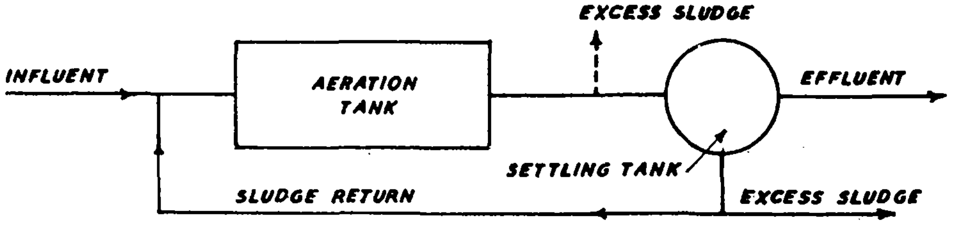


FIG. 13.1 CONVENTIONAL ACTIVATED SLUDGE

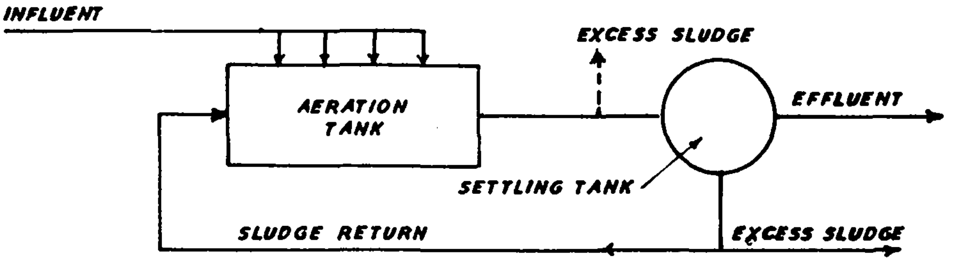


FIG. 13.2 STEP AERATION

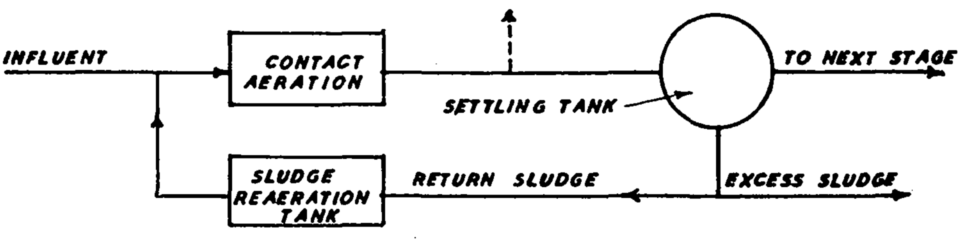


FIG. 13.3 CONTACT STABILISATION

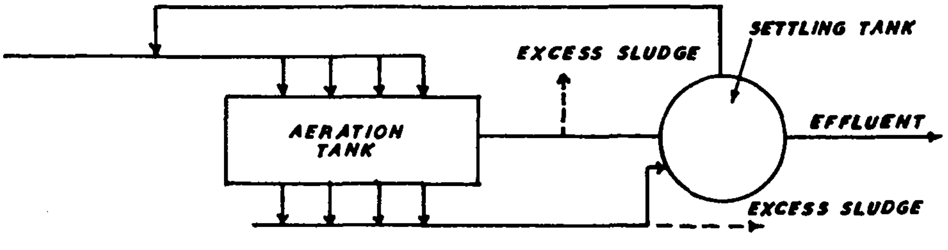


FIG. 13.4 COMPLETE MIX PLANT

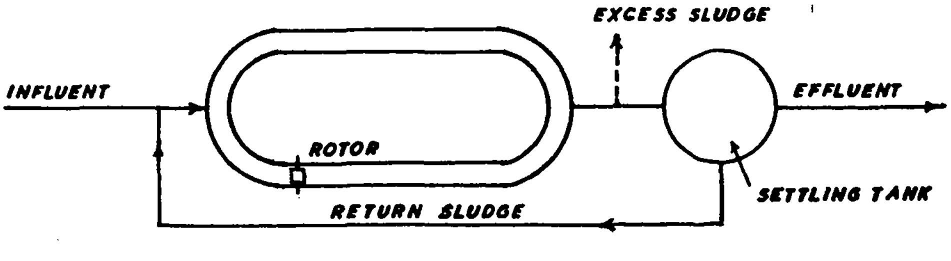


FIG. 13.5 OXIDATION DITCH

Table 13.1

Characteristics and Design Parameters of Different Activated Sludge Systems

Process type	Flow Regime	MLSS mg/l	MLVSS / MLSS	F/M	HRT hrs.	Volumetric Loading kg BOD_5 per m^3	SRT (days)	Q_r / Q	BOD removal percent	kg O_2 / kg BOD_5 Removal	Air requirement per kg. BOD_5 M^3
1	2	3	4	5	6	7	8	9	10	11	12
Conventional	Plug	1500 to 3000	0·8	0·4 to 0·2	4 to 8	0·3 to 0·7	5 to 15	0·25 to 0·5	85 to 95	0·8 to 1·1	40 to 100
Tapered Aeration.	Plug	1500 to 3000	0·8	0·4 to 0·2	4 to 8	0·3 to 0·8	5 to 15	0·25 to 0·5	85 to 95	0·7 to 1·0	50 to 75
Step Aeration	Plug	2000 to 3000	0·8	0·4 to 0·2	3 to 5	0·7 to 1·0	5 to 15	0·25 to 0·75	85 to 95	0·7 to 1·0	50 to 75
Contact Stabilisation	Plug	1000 to 3000(1) 3000 to 6000 (2)	0·8	0·5 to 0·2	0·5 to 1·5(1) and 3 to 6 (2)	1·0 to 1·2	5 to 15	0·25 to 1·0	85 to 95	0·7 to 1·0	50 to 75 (3)
Complete mix.	Complete mix	3000 to 6000	0·8	0·6 to 0·2	3 to 5	0·8 to 2·0	5 to 15	0·25 to 1·0	85 to 95	0·7 to 1·0	50 to 75
Modified Aeration	Plug	300 to 800	0·8	5·0 to 1·5	1·5 to 3	1·2 to 2·4	0·2 to 0·5	0·05 to 0·15	60 to 75	0·4 to 0·6	25 to 50
Extended Aeration	Complete mix	3000 to 8000	0·5 to 0·6	0·15 to 0·05	18 to 36	0·2 to 0·4	20 to 30	0·35 to 1·5	90 to 98	1·0 to 1·2	100—135

1. In contact Aeration Tank.
2. In sludge Reaeration Tank.
3. Divided equally between contact aeration tank and sludge reaeration tank.

mass is exposed to the main stream of sewage containing the toxic constituents only for a short time. The air requirements of the process are the same as for the conventional system, the air supply being divided equally between the contact aeration tank and the sludge reaeration tank. However the total aeration tank volume required (sludge reaeration tank plus contact aeration tank) is only about half. The process, therefore, presents an effective method of uprating existing conventional type plants when sewage characteristics are satisfactory. The process has found application in medium sized plants with capacities upto 40 mLd.

13.3.5 Complete Mix

The complete mix activated sludge plant employs a completely mixed flow regime. In a rectangular tank, complete mixing is achieved by distributing the sewage and the return sludge uniformly along one side of the tank and withdrawing the aerated sewage uniformly along the opposite side (Fig. 13—1(d)). In a circular or square tank complete mixing is achieved by mechanical aerators with adequate mixing capacity installed at the centre of the tank. The loading parameters of the process are shown in Table 13-1.

The complete mix plant has the capacity to hold a high MLSS level in the aeration tank enabling the aeration tank volume to be reduced. The plant has increased operational stability at shock organic loadings and also increased capacity to treat toxic biodegradable wastes like phenols. The plant is less liable to upset by slugs of flows of toxic wastes. The complete mix process has been used mainly for small plants with capacity less than 25 mLd.

13.3.6 Modified Aeration

The modified aeration process is also termed minimal solids aeration. The flow scheme for the process is similar to that of the conventional system. However, primary settling is dispensed with frequently. The process is distinguished by short aeration period, high volumetric loading, high F/M, low percentage of sludge return and low concentration of MLSS (Table 13-1). The air requirements are lesser than in the conventional system.

Modified aeration develops dispersed biological growth which does not flocculate and quickly settle. Alum and polyelectrolytes are sometimes employed to improve secondary settling. The BOD removal in the process is only 60—75 percent. The method is suitable when only intermediate quality effluent is desired as for example, when the effluent is to be used for sewage farming. The process is attended by substantial savings in construction and aeration costs. The process has been employed mainly in large plants with capacities above 200 mLd.

13.3.7 Extended Aeration

The flow scheme of the extended aeration process and its mixing regime are similar to that of the complete mix process. The oxidation ditch (Fig 13.1(c)) also conforms to the extended aeration principle and relies on aerating the mixed liquor in an endless ditch.

Primary settling is omitted in the extended aeration process, but communition is often provided for screenings. The process employs low organic loading, long aeration time, high MLSS concentration and low F/M. The BOD removal efficiency is high. Because of long detention in the aeration tank, the mixed liquor solids undergo considerable endogenous respiration and get well stabilised. The excess sludge does not require separate digestion and can be directly dried on sand beds. Also the excess sludge production is a minimum.

The air requirements for the process are high and the running costs are also therefore quite high. However, operation is rendered simple due to the elimination of primary settling and separate sludge digestion. The method is, therefore, well suited for small communities having sewage flows less than 4 mLd.

13.4 DESIGN CONSIDERATION

The items for consideration in the design of an activated sludge plant are aeration tank capacity and dimensions, aeration facilities, secondary sludge recycle and excess sludge wasting.

13.4.1 **Aeration Tank**

The aeration tank capacity is determined from the F/M and MLSS values selected for the plant. The F/M and MLSS levels generally employed in different types of the activated sludge process are given in Table 13-1 along with their corresponding BOD removal efficiencies. The lower F/M values are recommended also when winter operating temperatures are low and near freezing point. The tank capacity is determined based on the organic loading formula given in 13.2.1. The capacity obtained should also be checked against the empirical design criteria for HRT and volumetric loading.

Except in the case of extended aeration plants and complete mix plants, the aeration tanks are designed as long narrow channels. This configuration is achieved by the provision of round-the-end baffles in small plants when only one or two tank units are proposed and by construction as long and narrow rectangular tanks with common intermediate walls in large plants when several units are proposed. In extended aeration plants other than oxidation ditches and in complete mix plants the tank shape may be circular or square when the plant capacity is small or rectangular with several side inlets and equal number of side outlets, when the plant capacity is large. The oxidation ditch has certain special features and its design is therefore discussed separately in 13.5.

The width and the depth of the aeration channel depends on the type of aeration equipment employed. The depth controls the aeration efficiency and usually ranges from 3 to 4·5 m, the latter depth being found to be more economical for installations treating more than 50 mLd. Beyond 70 mLd duplicate units are preferred. The width controls the mixing and is usually kept between 5 and 10 m. Width-depth ratio should be adjusted to be between 1·2 to 2.2. The length should not be less than 30 or not ordinarily longer than 100 m in a single section length before doubling back. The horizontal velocity should be around 1.5 m/min. Excessive width may lead to settlement of solids in the tank. Triangular baffles and fillets are used to eliminate dead spots and induce spiral flow in the tanks. Tank free-board is generally 0·5 m.

Due consideration must be given in the design of aeration tanks to the need for emptying them for maintenance and repair of the aeration equipment. Intermediate walls should be designed for empty conditions on either side. The method of dewatering should be considered in the design and provided for during construction.

The inlet and outlet channels of the aeration tank should be designed to maintain a minimum velocity of 0·2 mps to avoid deposition of solids. The channels or conduits and their appurtenances should be sized to carry the maximum hydraulic load to the remaining aeration tank units when any one unit is out of operation.

The inlet should provide for free fall into the aeration tank when more than one tank unit or more than one inlet is proposed. The free fall will enable positive control of the flows through the different inlets. Outlets usually consist of free-fall weirs. The weir length should be sufficient to maintain a reasonably constant water level in the tank. When multiple inlets or multiple tanks are involved, the inlets should be provided with valves, gates or stop planks to enable regulation of flow through each inlet.

13.4.2 **Oxygen Requirements**

Oxygen is required in the activated sludge process for the oxidation of a part of the influent organic matter and also for the endogenous respiration of the microorganisms in the system. The former is a function of the BOD removed, while the latter is a function of the MLSS in the aeration tank. The total oxygen requirement of the process may be formulated as follows :

$$O_2 \text{ required, kg/day} = a'\,Q(L_a - L_e) + b'\,(VX_t/1000) \quad . \quad . \quad . \quad . \quad . \; (13\cdot5)$$

where a' and b' are constants determined experimentally and other symbols are as before. For municipal sewage, the values of a' and b' have been reported to be 0·4 to 0·65 and 0·1 to 0·3 respectively.

The formula does not allow for nitrification but allows only for carbonaceous BOD removal. The extra oxygen requirements for nitrification is 4·56 kg O_2 / per kg NH_3—N oxidised to NO_3—N.

The total oxygen requirements per kg BOD_5 removed for different activated sludge processes are given in Table 13·1. The amount of oxygen required for a particular process will increase within the range shown in the table as the F/M value decreases.

13.4.3 Aeration Facilities

The aeration facilities of the activated sludge plant are designed to meet the calculated oxygen demand of the process while maintaining in the aeration tank a minimum DO of about 1-2 mg/l which is necessary for proper development of biological sludge. In addition to supplying dissolved oxygen, the aeration devices have also to provide adequate mixing and agitation so that the mixed liquor suspended solids do not settle down. Aeration devices usually consist of either diffused air aerators or mechanical aerators.

13.4.3.1 *Diffused air aeration*

Diffused air aeration involves the introduction of compressed air into the sewage through submerged diffusers or nozzles. The aerators may be of the fine bubble or coarse bubble type. In the former compressed air is released at or near the bottom of the aeration tank through porous tubes or plates made of aluminium oxide or silicon oxide grains cemented together in a ceramic matrix.

The permeability of a diffuser plate is defined as the volume of free air in m³/min that will pass through 1 m² of diffuser at 50 mm differential pressure under dry conditions at temperature of 21° C. Recommended permeabilities lie between 12 and 24, larger ratings tending to give uneven distribution.

Standard Ceramic plate diffusers have dimensions 0·3 m × 0·3 m×25 mm with pores of about 0·3 mm dia corresponding to the permeability grading of 12. Such plates under water, pass about 1·2m³ of air/min/m² with pressure losses between about 100mm and 200mm of water. For the best uniformity of distribution and prevention of clogging problems a minimum of 0·6 m³ of air/min/m² of plate surface under water is advisable and should not exceed 2·5 m³ because of pressure losses, poor air economy and more rapid clogging due to corrosion.

The air supply should be at least 0·25m³ per min per metre length of channel. Spacing of the diffusers should be 0·6 and preferably 1 m apart between centres to avoid rising streams of bubbles.

Other types of diffusers using porous material having the form of long tubes or mushroom shaped domes installed at a level 0·3 to 0·6 m above the floor of the aeration tank are preferred sometimes to the diffuser plates which are placed at the floor level because of the ease of cleaning and replacement with the unit under operation. 180 mm dome diffusers pass about 0·85—1·4 m³ of air per hour through each dome. Porous diffusers are liable to clogging from the inside by the dust carried in the air and also clogging from the outside by the suspended solids in the sewage. The air supplied must therefore be free of dust and frames supporting the diffusers must not form rust which cause choking. Air supplied to porous diffusers should contain less than 0·02 mg of dust per m³. Troubles due to clogging from the inside can be reduced by providing air filters and those due to clogging from outside can be avoided by providing adequate air pressure below the diffusers at all times. In spite of such precautions, fine bubble diffusers will require periodical cleaning. For cleaning plate diffusers, the aeration tank will have to be emptied. Swinging pipe diffusers with filters which can be swung out of the tank have the advantage that they can be cleaned regularly without shutting down and emptying the tank. Tube diffusers are therefore preferred to plate diffusers.

Successful aeration has also been obtained by introducing the air directly through comparatively large holes in pipes located at the bottom along one side of the tank or even through open ends of vertical pipes of about 15—40 mm dia and about 0·6 m apart. In these cases, aeration not only depends upon air bubbles but also upon the rapid circulation of the whole contents of the tank with the aid of coarse air bubbles, whereby the exposed water surfaces are renewed in a matter of seconds and the oxygen is also rapidly absorbed from the atmosphere. The arrangement of diffuser plates should be such that the influent half of the aeration channel has 30% greater concentration of plates than in the effluent half of the tank in order to obtain a uniform life, since the diffuser at the inlet end tends to clog more rapidly.

Coarse bubble aerators consist of proprietary devices such as Monosparj, deflectofuser, Discfuser, etc. They have slightly lower aeration efficiency than fine bubble aerators but are cheaper in first cost and are less liable to clogging and do not require filtration of air. Air diffusers are generally placed along one side of the aeration tank, helping to set up a spiral flow in the tank which improves mixing and prevents the solids from settling. They are located 0·3 to 0·6 m above tank floor to aid in tank cleaning and reduce clogging during shutdown.

The quantity of oxygen to be delivered through the diffuser system depends on the oxygen demand of the sewage and the efficiency of oxygen transfer of the diffusers will the latter being controlled by the size of the air bubble produced and the depth of submersion of the diffusers. The oxygen transfer efficiency at 1 to 2 mg/l of DO in aeration tank varies from 5 to 15 percent for most diffusers with 8 percent being common for fine bubble diffusers and 6 percent for coarse bubble diffusers. The quantity of air to be delivered through the diffuser system can be worked out from the quantity of oxygen to be delivered assuming 23·2 percent oxygen in air and air density of 1·43 kg/m³ under standard conditions. The air requirements for dfferent types of activated sludge system are given in Table 13—1.

The air delivery systems are designed to deliver 1·5 times the normal air requirements and compressors are installed in multiple units to enable increase or decrease of the air supply. The pressure developed by the air compressors should equal the depth of sewage to the diffuser units plus losses in diffuser plus about 25% extra for losses in transmission or about 0·4—0·65 kgf/cm² total. Air-pipings are designed for velocities of 6 to 30 mps for pipe diameters of 25 mm—1500 mm. Air header pipes should be located above the tank water level to avoid back siphonage when the compressors trip.

The advantages of flexibility in the aeration arrangements are given particular attention for larger plants. Greater flexibility requires costlier air supply piping systems—additional fittings, valves, airflow meters, perhaps less efficient compressor operation, etc.—but the added costs may be justified by improved purification efficiencies. Automatic dissolved oxygen or redox potential measurements may also be provided.

With the development of efficient mechanical surface aerators requiring very little operational attention diffused air aeration is falling out of use.

13.4.3.2 *Mechanical aerators*

Mechanical aerators were linked to small installations in the past but with recent improvements in their design, they are being increasingly used for large plants in preference to diffused air aeration systems. Some of their advantages are higher oxygen transfer capacity, absence of air piping and air filter and simplicity of operation and maintenance.

Mechanical aerators generally consist of large diameter impeller plates revolving on vertical shaft at the surface of the liquid with or without draft tubes. A hydraulic jump is created by the impellers at the surface causing air entrainment in the sewage. The impellers also induce mixing. The speed of rotation of the impellers is usually 70—100 rpm. The agitator-sparjer is a special mechanical aerator system involving the release of compressed air at the bottom of the aeration tank in large bubbles and the breaking up of the bubbles into fine bubbles by submerged turbine rotors located above the air outlets. The turbine rotors also provide mixing.

Mechanical aerators are rated based on the amount of oxygen they can transfer to tap water under standard conditions of 20°C, 760 mm kg barometric pressure and zero DO.

The oxygen transfer capacity under field conditions can be calculated from the standard oxygen transfer capacity by the formula.

$$N = N_S \frac{C_S - C_L}{9 \cdot 17} \times 1 \cdot 024^{T-20} \times \alpha \quad \ldots(13\text{-}6)$$

Where

N=oxygen transferred under field conditions, kg O_2/hr.

N_S=oxygen transfer capacity under standard conditions, kg O_2/hr;

C_S=dissolved oxygen saturation value for sewage at operating temperature.

C_L=operating DO level in aeration tank usually 1 to 2 mg/l

T=Temperature, °C.

α=Correction factor for oxygen transfer for sewage, usually 0·8 to 0·85.

The oxygen transfer capacity of mechanical aerators under standard conditions is about 1·9 kg/kwh compared to 1·3 kg/kwh for sparjer system, 1·5 kg/kwh for diffused air fine bubble aerators and 0.9 kg/kwh for diffused air coarse bubble aerators. Oxygen transfer capacities of surface aerators should be supported by actual test data for the model and size offered.

13.4.3.3 *Mixing requirements*

The aeration equipment have also to provide adequate mixing in the aeration tank to keep the solids in suspension. Mixing considerations require that the power input in mechanical aerators should not be less than 0·015—0·026 kw/m³ of tank volume. The power input of mechanical aerators derived from oxygenation considerations should be checked to satisfy the mixing requirements and increased where required.

13.4.4 Measuring Devices

Devices should be installed for indicating flow rates of raw sewage or primary effluent, return sludge and air to each aeration tank. For plants designed for sewage flow of 10 mLd or more integrating flow recorders should be used.

13.4.5 Secondary Settling

Secondary settling assumes considerable importance in the activated sludge process as the efficient separation of the biological sludge is necessary not only for ensuring final effluent quality but also for return of adequate sludge to maintain the MLSS level in the aeration tank. The secondary settling tank of the activated sludge process is particularly sensitive to fluctuations in flow rate and on this account it is recommended that the units be designed not only for average overflow rate but also for peak overflow rates. The high concentration of suspended solids in the effluent require that the solids loading rate should also be considered.

The recommended overflow rates and solids loading rates for secondary settling tanks of activated sludge have been given in Table 12-1.

Sludge thickening is a special function of the secondary settling tank of an activated sludge plant as thickened sludge is an advantage in sludge recycling. Sludge thickening is effected by providing adequate tank depth as indicated in Table 12-1.

13.4.6 Sludge Recycle

Control of the sludge recycle rate is the method of maintaining the desired MLSS and F/M level in the aeration tank which determine the degree of purification. The rate of sludge recycle can be formulated in terms of an empirical measurement known as sludge volume index (SVI). The index is defined as volume occupied in ml by one gm of solids in the mixed liquor after settling for 30 mins and is determined experimentally.

The rate of return sludge required to maintain a desired MLSS concentration in the aeration tank is given by the expression :

$$\frac{Q_r}{Q} = \frac{X_t}{\frac{10^6}{SVI} - X_t} \qquad \text{......(13.7)}$$

where Q_r=return flow, mLd, and other notations are as before.

Return sludge ratios employed in different activated sludge systems are shown in Table 13-1. The return sludge has always to be pumped and the pump capacities should be designed for a minimum return sludge ratio of 0·50—0·75 for large plants and 1·0—1·5 for smaller plants irrespective of theoretical requirements. The required capacity should be provided in multiple units to permit variation of return sludge ratio as found necessary during operation.

13.4.7 Excess Sludge Wasting

The sludge generated in the aeration tank has to be wasted to maintain a steady level of MLSS in the system. The excess sludge quantity will increase with increasing F/M and decrease with increasing temperature. In the case of domestic sewage, the excess sludge to be wasted will be about 0·50—0·75 kg per kg BOD_5 removed for the conventional system and its variations operating with F/M values of 0·2—0·5 and about 0·4—0.5 kg per kg BOD_5 removed in the case of extended aeration plants having no primary settling. The volume of sludge to be wasted will depend on the suspended solids concentration in the waste stream.

Excess sludge may be wasted either from the sludge return line or directly from the aeration tank as mixed liquor. The latter procedure is to be preferred as the concentration of suspended solids will then be fairly steady in the waste stream making control easy. The waste sludge is either discharged into the primary settling tank or thickened in a sludge thickening unit and digested directly. In extended aeration plants the excess sludge is taken to sludge drying beds directly and the sludge filtrate discharged into the effluent stream.

13.5 DESIGN OF OXIDATION DITCH

The oxidation ditch is one form of an extended aeration system having certain special features like an endless ditch for the aeration tank and a rotor for the aeration mechanism. The ditch consists of a long continuous channel usually oval in plan (Fig. 13-1(e)). The channel may be earthen with lined sloping sides and lined floor or it may be built in concrete or brick with vertical walls. The sewage is aerated by a surface rotor placed across the channel. The rotor will not only aerate the sewage but will also impart a velocity of 0·3—0·4 mps to the mixed liquor preventing the biological sludge from settling out. The areation rotors are generally of the cage type but may also be of the angle iron type. Cage rotors usually have a dia of 70 cm and a speed of 75 rpm. The standard oxygen transfer capacity is 2·8 kg O_2/m length per hr at 16 cm depth of immersion. This rotor has been found to impart adequate circulation for 120-150 m^3 of ditch volume per metre length of rotor. Power requirements per m length is about 1.35 kw at the rpm and immersion depth stipulated. The ditch volume is designed based on the criteria given for extended aeration systems in Table 13-1. The width of ditch is designed to accommodate the length of aerator required to meet the oxygen demand either as a single rotor or as multiple rotors. The depth of ditch is kept as 1.0. to 1.2m and the length of the ditch is designed to give the required aeration tank volume.

The raw sewage and return sludge are discharged into the ditch upstream of the rotors. The outlet of the ditch should be located geometrically opposite to the inlet. The outlet weir should be baffled and should be of adequate length so that the ditch water level does not rise excessively and overload the rotors at periods of peak flow.

Oxidation ditches may be operated intermittently or continuously. In intermittent operation, the oxidation ditch functions also as a settling tank. Intermittent operation cycles are (i) closing inlet and aerating the sewage, (ii) stopping the rotor and letting the contents settle, and (iii) letting in fresh sewage which displaces an equal quantity of clarified effluent. Sludge is wasted from the mixed liquor or a sludge sump at the bottom of the ditch. Intermittent operation is adopted only in small plants. In continuous operation, the oxidation ditch is followed by a secondary settling unit as in any other extended aeration system.

13.6 NITRIFICATION

Activated sludge plants are ordinarily designed for the removal of only carbonaceous BOD. However, there may be incidental nitrification in the process. Nitrification will consume part of the oxygen supplied to the system and reduce the DO level in the aeration tank. Nitrification will also lead to subsequent denitrification in the secondary settling tank causing a rising sludge problem also called blanket rising. Nitrification is aided by low F/M and long aeration time. It may be pronounced in extended aeration plants especially in hot weather. At the other extreme in the contact stabilisation process and in the modified aeration plant, there may be little or no nitrification.

Nitrification though generally not desired may be required in specific cases, e.g. when ammonia has to be eliminated from the effluent in the interest of pisciculture or when nitrification-cum-denitrification is proposed for elimination of nitrogenous matter from the effluent for control of eutrophication. In such cases, the trend is towards the design of two stage plants with only carbonaceous BOD removal in the first stage and nitrification in the second stage. The first stage unit is designed with high F/M to prevent nitrification. The two stage process eliminates the problem of denitrification and rising sludge in the first stage settling tank.

13.7 OPERATION

The most important aspect in the operation of an activated sludge plant is the maintenance of proper F/M which is achieved by increasing or decreasing the MLSS levels in the aeration tank to suit the influent BOD_5 loads. The MLSS in the aeration tank can be regulated by controlling the rate of sludge return based on SVI determined experimentally. Excess sludge wasting is generally controlled based on experience. In small extended aeration plants, sludge is often wasted only intermittently when the sludge fills up the settling tank and starts showing up in the effluent.

The quick settleability of sludge is an important factor in the efficient performance of the activated sludge plant. The SVI serves also as an index of sludge settleability. SVI values of 80-150 are considered satisfactory in plants operating with MLSS of 800-3500 mg/l. Sludge with poor settling characteristics is termed bulking sludge. Sludge bulking results in poor effluent due to the presence of excessive suspended solids and also in rapid loss of MLSS from aeration tank. Sludge bulking is generally due to inadequate air supply, low pH or septicity and also due to growth of filamentous organisms consequent to the presence of industrial wastes containing high concentration of carbohydrates in sewage. Sludge bulking is controlled by eliminating the causes and by application of chlorine either to the sewage or to the return sludge to control filamentous growths. Chlorine requirements are 0·2 to 1·0 percent of dry solids weight in return sludge.

Occasionally, the secondary settling tank may function poorly even when the sludge volume index is satisfactory and sludge may rise up in the tank and escape with the effluent. Rising sludge may be due to denitrification in the settling tank releasing nitrogen bubbles which buoys up the sludge. The problem can be overcome by increasing the return sludge rate increasing the speed of the sludge scraper mechanism and increasing the sludge wasting rate.

13.8 ROTATING BIOLOGICAL CONTACTOR

This is a comparatively simple system operating on the principle of moving media and hence discussed here. These units can be adopted for small to medium populations less than 100,000.

The rotating biological contactor also called 'Biological disc unit' may be likened to a horizontal biofilter with rotating media. The unit consists of a number of rotating discs which are partially submerged in a semicircular tank receiving the raw sewage. The discs serve as an excellent support for the biological film which grows at the expense of the organic matter thus bringing about the required stabilisation. The discs while rotating alternately dip into the sewage and aerated when exposed to the atmosphere.

The essential features of the unit are—

(a) a main shaft of sufficient rigidity on which the circular discs are mounted. These discs could be of asbestos cement sheets or PVC sheets;

(b) a tank in which the discs fixed to the rotating shaft are half submerged in the sewage; and

(c) a driving mechanism consisting of a motor and a reduction gear.

The detention periods range from 1 to 1.5 hours in the disc chamber and about one hour in the settling basin. Diameter of discs vary from 1 to 3m with clear spacing between discs of 2.5 cm, speed of rotation of discs 5 rpm and submergence of discs of 50%. The discs are made of either AC or PVC.

Reductions of 90% in BOD and SS could be expected. The energy consumption varies from 0.6 to 1.2 kw hr per kg. of BOD removed with a loss of head of less than 2.5 cm through the unit.

CHAPTER 14

TRICKLING FILTERS

The trickling filter consists of a permeable bed of media through which the sewage or liquid waste is allowed to percolate. The filter media is of broken rock, gravel, blast furnace slag or inert synthetic material. The filter is generally circular and the sewage is evenly distributed on the surface of the filter having underdrains for collecting the treated effluent.

The trickling filter is always preceded by primary sedimentation so that the settleable solids in the sewage may not clog the filter. The sedimentation tanks should have skimmers to remove the scum. The trickling filter is always followed by a final settling tank to remove from the filter effluent the settleable organic solids produced in the filtration process. It is advantageous to provide skimming devices for the final settling tanks also. The trickling filter serves both to oxidise and bioflocculate the organic material in sewage and their efficiency is assessed on the total reduction in BOD effected through the filter and the subsequent settling tank, since the effluent quality is reckoned after the settlement of the bioflocculated solids.

As sewage trickles through the filter media a biological slime consisting of aerobic bacteria and other biota builds up around the media surfaces, normally in a period of two weeks, making the filter ready for operation. Organic material in the sewage is absorbed on the biological slime where they are partly degraded by the biota thus increasing the weight and thickness of the slime. Eventually there is a scouring of the slime and a fresh slime layer begins to grow on the media. This phenomenon of scouring of the slime is called sloughing or unloading of the filter.

Filter sloughing aids ventilation by keeping the filter media open. It aslo continuously renews the biota, maintaining it active, which is essential for the efficient functioning of the filter. The degree of filter sloughing will depend on the organic loading which will control the growth of the slime and the hydraulic loading which will influence its scour.

Trickling filters are used for the biological treatment of domestic sewage and industrial wastes which are amenable to aerobic biological processes. They find use for complete treatment of moderately strong wastes and as roughing filter for very strong wastes prior to activated sludge units. Trickling filters possess a unique capacity to handle shock loads and provide dependable peformance with a minimum of supervision. They are particularly suited for plants of capacities less than 5 mLd.

14.1 TYPES OF FILTERS

Trickling filters are classified as low rate and high rate based on hydraulic and organic loading rates. Although there is no well demarcated practice, the following ranges of loadings are usual for low rate and high rate filters.

	Hydraulic loading in $m^3/d/m^2$	Organic loading as BOD_5 in $g/d/m^3$
Low rate filters .	1 to 4	80 to 320
High rate filters .	10 to 30 (including recirculation)	500 to 1000 (excluding recirculation)

The hydraulic loading rate is the total flow including recirculation applied on unit area of the filter in a day, while the organic loading rate is the 5-day 20°C BOD, excluding the BOD of the recirculant, applied per unit volume in a day. Much higher organic loadings than indicated above have been used in roughing filters.

Recirculation is not generally adopted in low rate filters. Media depths for low rate filters range from 1.8 to 3m. They require larger media volumes than high rate filters and are higher in capital costs. However, they are easy to operate and give consistently good quality effluent and are preferred when plant capacities are small as in the case for institutions.

In contrast to the low rate filters, in high rate filters a part of the settled or filter effluent is recycled through the filter. Recirculation has the advantage of bringing the organic matter in the waste in contact with the biological slime more than once, thus increasing the efficiency of the filters. It enables higher hydraulic loading and thereby reduces filter clogging and aids uniform distribution of organic load over the filter surface. It also helps to dampen the variations in the strength and the flow of sewage applied on the filter. The ratio of the recycled flow to the sewage flow is known as the recirculation ratio. Recirculation ratios usually range from 0.5 to 3 and values exceeding 3 are considered to be uneconomical in the case of domestic sewage but ratios of 8 and above have been used with industrial wastes. High rate filters, may be single stage or two stage. Media depths of 0.9 to 2.5 m have been used for high rate filters with an optimum range of 1.5 to 2.0 m for the first stage and 1 to 2 m for the second stage filters. Single stage units consist of a primary settling tank, the filter, secondary settling tank and facilities for recirculation of the effluent. Settling before recirculation is carried out either in the primary or the secondary clarifier as shown in (a), (b) and (c) of Fig. 14-A. Recent studies have shown that for municipal sewage, direct recirculation without settling is as effective as recirculation of settled effluent.

Two stage filters consist of two filters in series with a primary settling tank, an intermediate settling tank which may be omitted in certain cases and a final settling tank. Recirculation facilities are provided for each stage. The effluent from the first stage filter is applied on the second stage filter either after settlement or without settlement. Some of the common flow diagrams are shown in Fig. 14-B. In 14-B (a) an intermediate clarifier is used for settling the first stage effluent before it is applied to the second stage filter and the recirculation is only through the settling tanks. In 14-B(b) the intermediate settling is omitted but the recirculation flows are settled. In 14-B(c), which is known as the series-parallel system, part of the settled raw sewage is applied directly to the second stage filter increasing the efficiency of that stage. In 14-B(d), there is neither intermediate settling nor settling of filter effluent prior to recirculation.

Two stage filtration will provide a higher degree of treatment than the single stage for the same total volume of media. Two-stage units are used for strong sewage when the effluent BOD has to be less than 30 mg/l.

14.2 PROCESS DESIGN

The important considerations in the design of trickling filters are the organic loading rate and the recirculation ratio. Once the organic loading rate is selected, the required filter volume can be calculated. The depth and surface area of the filter are then suitably chosen to secure hydraulic loading rates within the prescribed limits.

A number of equations are available for the determination of plant efficiencies based on the organic loading rates and recirculation ratios. Of these, the equations developed by Rankin and the National Research Council of USA (NRC) are commonly used for the design of trickling filters.

14.2.1 Rankin's Equations

Rankin developed a set of equations for the performance of high rate filters of various flow diagrams based on the requirements of the Ten State Standards.

For Single Stage Filters, the Ten State Standards states that the BOD of the influent to the filter (recirculation included) shall not exceed three times the BOD of the required settled effluent, Rankin's equation for single stage high rate filters of flow diagram 14-A(a) and 14-A(b) express the above relation as

$$S_2+R_1S_4=3(1+R_1)S_4 \quad \text{or} \quad S_4=\frac{S_2}{3+2R_1} \quad \ldots(14\text{-}1)$$

$$\text{and} \quad E_2 = \frac{1+R_1}{1.5+R_1} \quad \ldots\ldots\ldots(14\text{-}2)$$

Where

S_4 = BOD of settled filter effluent, mg/l;

S_2 = BOD of influent sewage after settling, mg/l;

E_1 = recirculation ratio; and

E_2 = efficiency of the filter.

The equation is applicable only when the organic loading rate on the filter, (recirculation included) is less than 1800 g/d/m³ and hydraulic loading rate (recirculation included) is maintained between 10 to 30 m³/d/m².

When the organic loading ranges between 1800 to 2800 g/d/m³, the equation to be used is

$$S_4 = \frac{S_2}{2.78 + 1.78R_1} \quad \ldots\ldots\ldots(14.3)$$

For all loadings in excess of 2800 g/d/m³, the BOD removal is assumed to be 1800 g/d/m³ only.

The above equations are also applicable to 14-A(c) and the first stage filters of 14-B(a).

In the flow diagram for 14-B (b) and 14-B(d) the effluent from the first stage filter is applied to second stage filter without settling. For the first stage plant effluent in this case,

$$S_4 = \frac{S_2}{2 + R_1} \quad \ldots\ldots(14\text{-}4)$$

$$\text{and} \quad E_3 = \frac{1 + R_1}{2 + R_1} \quad \ldots\ldots(14\text{-}5)$$

the loading limitation being the same as for equation 14-1.

In 14-B(c), the first stage effluent will consist partly of settled sewage which does not pass through the first stage filter. In this case,

$$S_4 = \frac{1.5 + S_2}{2.5 + r} \quad \ldots(14\text{-}6)$$

where r is the assumed recirculation given by

$$\frac{Q^1-Q}{Q}$$

Q being the raw sewage flow to the plant and Q^1 the total flow through the first stage filter.

The BOD loading on the filter is given by the equation

$$La_1 = Q(S_2 + R_1S_4)\,..(14\text{-}7)$$

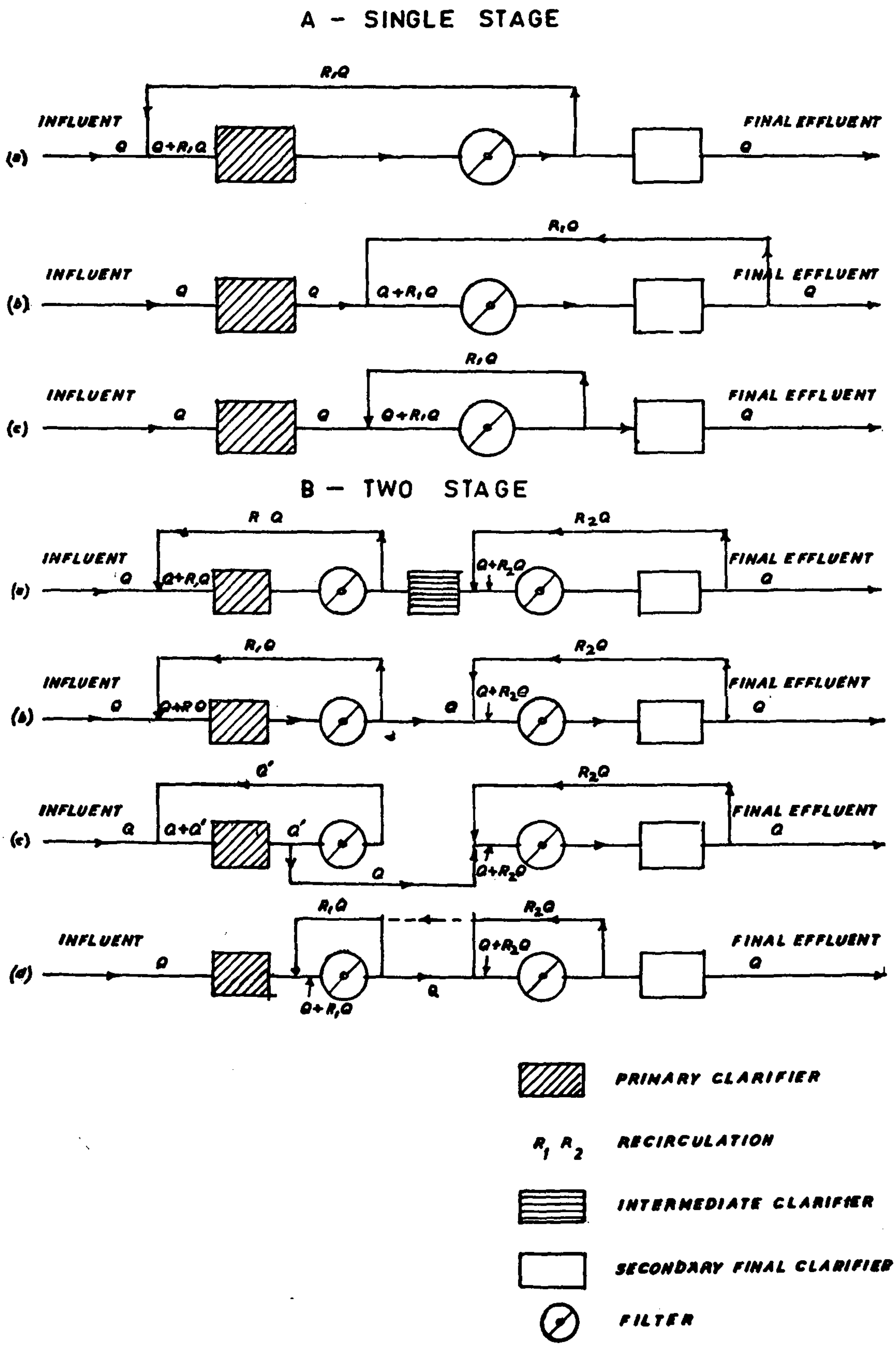
where

La_1 = BOD_5 loading in kg/day; and

Q = Sewage flow in mLd.

FIGURE 14

FLOW DIAGRAMS OF HIGH RATE TRICKLING FILTER PLANTS



The efficiency of removal of BOD for the second stage filter is less than that of the first stage because the amenability to treatment of the applied BOD is affected by previous treatment. Hence for the second stage filters, the Ten State Standards stipulate that the BOD of the sewage applied to the second stage filter (recirculation included) shall not exceed two times the BOD expected in the settled effluent. Rankin's Equations for flow diagrams 14-B(a), (b) and (d) are

$$S_4 + R_2S_6 = 2(1+R_2)S_6 \text{ or } S_6 = \frac{S_4}{2+R_2} \quad \ldots (14.8)$$

$$\text{and } E_3 = \frac{1+R_2}{2+R_2} \quad \ldots (14.9)$$

where

S_6 = BOD of settled effluent from second stage, mg/l;

R_2 = recirculation ratio in the second stage filter; and

E_3 = efficiency of second stage filter.

The equation for 14-B(c) has the form

$$S_6 = \frac{S_4}{R_2+3} \quad \ldots (14\cdot10)$$

because a part of the influent is fresh settled sewage which allows for greater efficiency.

14.2.2 NRC Equations

The NRC equations for trickling filter performance are empirical expressions developed from a study of the operation results of trickling filters serving military installations in U.S.A. These equations are applicable to both low rate and high rate filters.

The efficiency of single stage or first stage of two stage filters E_2 is given by

$$E_2 = \frac{100}{1+0.44\sqrt{\frac{W_1}{V_1F_1}}} \quad \ldots (14\text{-}11)$$

For the second stage, the efficiency

$$E_3 = \frac{100}{1+0.44\sqrt{\frac{W_1(1-E_2)}{V_2F_2(1-E_2)}}}$$

$$= \frac{100}{\frac{1+0.44}{(1-E_2)}\sqrt{\frac{W_2}{V_2F_2}}} \quad \ldots (14\text{-}12)$$

where

E_2 = percentage efficiency in BOD removal of the single stage or first stage of the two stage filter :

E_3 = percentage efficiency of second stage filter;

W_1 = BOD loading of settled raw sewage in the single stage or first stage of the two stage filter in kg/day;

$W_2 = W_1(1-E_2)$ = BOD loading on second stage filter in kg/day;

V_1 = Volume of first stage filter in m^3;

V_2 = Volume of second stage filter in m^3;

F_1 = recirculation factor or number of effective passes for first stage filter = $\frac{1+R_1}{[1+(1-f)R_1]^2}$

R_1 = recirculation ratio for the first stage filter;

f = treatability factor (0.9 for sewage);

F_2 = recirculation factor or number of effective passes for second stage filter= $\frac{1+R_2}{[1+(1\text{-}f)R_2]^2}$; and

R_2 = recirculation ratio for the second stage filter

14.2.3 Other Equations

14.2.3.1 *Velz equation*

While the earlier equations have been derived on the basis of data analysis, velz equation is based on a fundamental law relating the BOD remaining at depth D as follows :

$$\frac{L_D}{L} = 10^{-KD} \quad \ldots (14\text{-}13)$$

Where

L = Total removable BOD, mg/l

L_D = Removable BOD at depth D, mg/l

D = Filter depth, m; and

K = a constant

Removable BOD in the equation is the maximum fraction of applied BOD removed at a specified hydraulic loading range. When recirculation is practised the total applied BOD, La, after dilution by recirculation, R, may be determined as follows;

$$La = \frac{L_i + RL_e}{1+R} \quad \ldots (14\text{-}14)$$

where

L_i = Total influent BOD, not including recirculation, mg/l;

L_e = Total effluent BOD, mg/l.

14.2.3.2 *Eckenfelder equation*

Eckenfelder improved on the Velz equation to include (i) the effect of changes in filter depth on the BOD removal per unit of depth and (ii) the hydraulic loading rate and suggested the following equations ;

$$\frac{L_e}{L_a} = \frac{1}{1 + \dfrac{18.6\, D^{0.67}}{\left(\dfrac{Q}{A}\right)^{0.5}}} \quad \ldots (14\text{--}15)$$

Where

L_a = influent BOD (including recirculation), mg/l

L_e = BOD of unsettled filter effluent, mg/l

D = depth in m

Q = flow in mLd and

A = area in hectares

14.2.3.3 *Galler and Gotaas equation*

Galler and Gotaas, based on a multiple regression analysis of data from existing plants and providing for the effects of recirculation, hydraulic loading, filter depth and temperature of the sewage, developed the following equation.

$$L_e = \frac{0.182\, K(iLi + rLe)^{1.19}}{(1+R)^{0.78}(D+0.305)^{0.67}a^{0.25}} \quad \ldots (14\text{--}16)$$

$$\text{Where } K = \frac{9.731}{i^{0.28}\, T^{0.15}}$$

L_e = Unsettled filter effluent BOD, mg/l

L_i = filter influent BOD, mg/l

D = filter depth, m

i = influent flow, mLd

r = recirculation flow, mLd

a = filter radius, m

T = Sewage temperature, °C

Galler and Gotaas suggested that recirculation improves the performance of the filter but established that a ratio of 4:1 was the practical upper limit for recirculation.

14.2.4 Applicability of the different equations

The NRC and Eckenfelder equations show fairly close agreement for filter volumes when no recirculation is adopted while the Galler and Gotaas equation give volumes significantly different. With recirculation there is good agreement in filter volumes between the Eckenfelder and Galler and Gotaas equations but the NRC equation gives very conservative design volumes. In general the NRC equations would seem to apply when recirculation is not considered, when seasonal variations in temperature are not large and when the sewage load is highly variable and of high strength.

14.3 CONSTRUCTIONAL FEATURES

14.3.1 Shape of Filter

Filters may have rectangular or square shape if fixed nozzles are used for flow distribution. As the rotary distributors are more reliable and easy to maintain and operate current practice is to adopt only circular filters.

14.3.2 Provision for Filter Flooding

Provision for flooding the filters is useful for controlling filter flies and ponding. To enable flooding, the filter walls must be designed for the internal water pressure and the main collecting channel must be placed inside the filter and provided with gate valves. An overflow pipe leading from the filter to the main collecting channel downstream from the gate valve is also necessary.

Provision for filter flooding should always be made in the case of small filters, especially the low rate filters. Such a provision in large filters, would not only increase the cost but also likely to cause hydraulic problems with the sudden discharge of large volumes of sewage when the flooded filter is drained. In such cases alternate methods may be required for controlling filter flies and ponding (14-4).

14.3.3 Filter Walls

Filter walls may be reinforced concrete or brickwork usually built above ground level. For flooding operation reinforced concrete is preferred.

14.3.4 Filter Floor

The filter floor is designed to support the underdrainage system and the superimposed filter media. The usual practice is to provide a nominally reinforced cement concrete slab, 10-15 cm thick, over a proper levelling course. The floor should slope between 0·5 to 5% towards the main collecting channel. The flatter slopes are used in larger filters.

14.3.5 Underdrainage System

The underdrainage system is intended to collect the trickling sewage and sloughed solids and to convey it to the main collecting channel and also to ventilate the filter media. The underdrains cover the entire floor of the filter to form a false bottom and consist of drains with semicircular or equivalent inverts. They will be formed of precast vitrified clay or concrete blocks, complete with perforated cover or they may be formed *in situ* with concrete or brick and covered with perforated precast concrete slabs.

The slope of the underdrains should be the same as that of the floor. The drains shall be so sized that the flow occupies less than 50% of the cross-sectional area with velocities not less than 0·75 mps at peak instantaneous hydraulic loading. The cover over the drains shall be perforated to provide a total area of inlet openings into the drains not less than 15% of the surface area of the filter. Underdrains may be open at both ends so that they may be inspected easily and flushed out if they become clogged.

14.3.6 Main Collecting Channel

The main collecting channel is provided to carry away the flow from the underdrains and to admit air to the filter. In a circular filter, the main channel may be located along a diameter, suitably curved around the central feed well or parallel to the diameter with a slight offset from the centre. Alternatively, the channel may be provided along the outer periphery of the filter. If inside the filter, the channel shall be provided with perforated covers to enable drainage and also ventilation of the filter media above the channel. The channel should be extended outside the filter, both at the upper and lower ends with vented manholes to facilitate ventilation and access for cleaning.

The channels shall have semicircular or other rounded inverts. The velocity in the channels shall not be less than 0·6 mps for the average instantaneous hydraulic loading and the flow shall be only half-depth particularly where recirculation is low. At the peak instantaneous hydraulic loading, the water level in the channel should not rise above the inverts of the underdrains at their junctions with the channel.

14.3.7 **Ventilation**

Adequate natural ventilation can be ensured by proper design of the underdrains and effluent channels. For filters larger than 30 m dia, a peripheral head channel on the inside of the filter with vertical vents is desirable to improve ventilation. 1m^2 of open grating in ventilating manholes and vent stacks should be provided for 250 m^2 of filter area. The vertical vents can also be used for flushing the underdrains.

In extremely deep or heavily loaded filters there may be some advantage in forced ventilation if it is properly designed, installed and operated. Such a design should provide for an air flow of 1m^3/min/m^2 of filter area in either direction.

It may be necessary during periods of extremely low air temperature to restrict the flow of air through the filter to keep it from freezing. However a minimum of 0·1 m^3/min/m^2 of filter area should be provided.

14.3.8 **Filter Media**

The requirements for filter media are high specific surface area, high percent void space, resistance to abrasion or disintegration during placement, insolubility in sewage or other waste water and resistance to spalling and flaking.

The most commonly used filter media is broken stone (trap rock, granite or limestone), slag or gravel of size 25 to 75 mm. The filtering material should be washed before it is placed in position. The Brinnel Hardness number of the material should be 12. Such media should be round or cubical in shape and free of thin, elongated and flat pieces. Not more than 5% of the media (by weight) should have the longest dimension greater than 3 times the smallest dimension.

Stones less than 25 mm dia, do not provide sufficient pore space between them to permit free flow of sewage and sloughted solids and also leads to plugging of media and ponding of filters. Large size stones greater than 100 mm dia, overcomes the plugging problem but due to relatively smaller surface area per unit volume cannot support as large a microbial population as the smaller size stones.

The size of the filer media is of considerable significance as the specific surface area decreases with increase in media size but the percent void space increases as in Table 14.1.

Table 14.1

Media	Size mm	Specific Surface area m^2/m^3	% void space
Granite	25—75	62	46
Granite	100	43	60
Blast furnace slag	30—75	65	49
Plastic	..	100	95

The recent trend is towards the use of larger media especially for high rate filters. The current specification for stone media is that when mechanically graded over vibratory screens :

100% should pass through 110 mm square mesh
95—100% should be retained on 75mm sq. mesh
0·2% alone should pass through 50mm sq. mesh
0·1% alone should pass through 25mm sq. mesh
(% given are by weight)

Media shall be placed and packed by hand for at least a height of 30 cm above the underdrains to avoid damage to the underdrainage system. The remainder of the material may be placed by means of wheel barrows or boxes or by belt conveyors. They should not be dumped or tipped from lorries.

14.3.8.1 *Plastic media*

The above criteria do not apply to plastic synthetic media which have high specific surface area, high void space and low weight. Synthetic filter media have of late been used successfully in superrate filters for the treatment of strong industrial wastes or sewage mixed with strong industrial wastes having hydraulic loading rates in the range of 30—90 m^3/d/m^2 and organic loading rates of 1000 to 2000 g/d/m^3. The media consists of interlocking sheets of plastics which are arranged in a honeycomb fashion to produce a porous and nonclog filter media. The sheets are corrugated so that a strong, lightweight media is obtained. Filters as deep as 10 m have been used with this type of synthetic media.

14.3.9 **Filter Dosing**

In the case of low rate filters, the minimum flow rate of sewage inflow may not be sufficient to rotate the distributor and discharge sewage from all nozzles. Hence, when adequate head is available dosing tank is provided to collect the settled sewage and dose the filter through a siphon intermittently. When head is inadequate, a collection well is provided to store the sewage and a suction level controlled pump, intermittently pumps the sewage to the filter. The design siphons are designed to dose the filters once in about 5 minutes under average flow conditions. In the case of high rate filters, there is no need for the special dosing device since continuous dosing is possible.

14.3.10 Flow Distribution

Fixed nozzle distributors are no longer used because of the elaborate piping requirement and the necessity of dosing tanks, siphons or motor operated valves to obtain variable dosing rates. Among the moving types, the longitudinally travelling distributors are not used in the country because of the long resting period associated with their time of travel from one end of the bed to the other and the need for a reversing gear at each end of the bed to change the direction of motion.

The present practice is to use only reaction type rotary distributors. Rotary distributors are commercially available in the country upto 60 m dia. The piping to the distributor is generally taken below the filter floor and in rare case through the filter media just above the underdrains. The pipe should be designed for a peak velocity of not greater than 2·0 mps and an average velocity not less than 1 mps.

The reaction type rotary distributor consists of a feed column at the centre of the filter, a turntable assembly at the top and two or more hollow radial distributor arms with orifices. The turntable should be provided with antitilt devices and also arrangements for correcting the alignment to obtain balanced rotation. The turntable assembly is provided with a mercury or mechanical water seal at its base. The current trend is to discourage mercury seals because of the chances of causing mercury pollution. Facilities should be available for draining the central column of the flow distributor for attending to repairs and maintenance.

The distributor arms are generally two in number, multiples of two also being adopted. When multiple arms are provided, low flows are distributed through two arms only and as flow increases, it is distributed by the additional arms. This is achieved by overflows from weirs, incorporated in the central column diverting the higher flows into the additional arms. The peak velocities in the distributor arms should not exceed 1·2 mps. The distributor arms are generally fabricated of steel and are liable to rapid corrosion. They should be fabricated and bolted together in such lengths as to facilitate dismantling for periodic repainting of their inside surfaces. The orifices in the distributor arms should be composed with aluminium orifice plates. Spreader plates, preferably of aluminium, should be provided below the orifices to spread out the discharge. The clearance between the distributor pipe and the top of the filter media should be greater than 15 cm.

Distributor arms should have gates at the end for flushing them. Atleast one end plate should have arrangement for a jet impinging on the sidewall to flush out fly larvae. The distributor arms may be of constant cross section for small units but in larger units, they are tapered from the centre towards the end to maintain the minimum velocity required in the arms.

The distribution arrangements should ensure uniform distribution of the sewage over the filter surface for which the size and spacing of the orifices in the distributor arms have to be varied carefully from the centre towards the end. Under average flow conditions, the rate of dosing per unit area at any one point in a filter should be within $\pm 10\%$ of the calculated average dosing rate per unit area for the whole filter. The distributors should also ensure that the entire surface of the filter is wetted and no area is left dry.

Reaction type rotary distributors require adequate hydraulic head for operation. The head required is generally 1 to 1·5 m measured from the centre line of distribution arms to the low water level in the distribution well or in the siphon dosing tank preceding the filter. When hydraulic head is inadequate to provide the reaction drive, the rotary distributor may be driven by an electric motor. The speed of rotation of the distributors shall ensure that the intervals of successive dosings is between 15 and 20 seconds.

14.3.11 Multiple Units

In a single stage plant, it is advisable to split the required filter volume into two or more units so that when one filter is taken out of operation for maintenance or repairs, the entire sewage can be passed through the remaining units, overloading them temporarily.

In a two stage plant, if multiple units are proposed in each stage, the entire sewage may be routed through the remaining units of the stage when one filter in that stage is taken out of operation. However, the recirculation flow is maintained at the original level, operating the stage at a lower recirculation ratio. If, instead, only one filter is proposed for each stage a bypass should be provided for each stage. It is customary in the design of two stage filters to use two filters of equal size.

14.3.12 Plant Hydraulics

The feed pipe to the filter, the distributor, the underdrains and the main collection channel should be designed for the peak instantaneous hydraulic loading on the filter. In low rate filters, the peak loading will be the peak discharging capacity of the dosing siphon or the dosing pump. In the case of high rate filters, the peak loading on the filters will be the sum of the peak rate of sewage flow and the constant recirculation rate.

When multiple units are used for the high rate filters in any stage, the hydraulics of the plant should be checked for peak loading with one filter out of operation, the entire flow routed through the remaining units. A reduced recirculation ratio is adopted for this condition so as to reduce the peak loading and avoid oversizing of the piping.

When multiple units are used care should be taken to ensure that the flow is divided properly between the various filters.

4.3.13 Pumping Arrangements

In a high rate filter, pumping is required for recirculation. Pumping may also be required for lifting the filter effluent to the settling tank or the next stage filter.

Except in the case of small plants, recirculation pumps should be installed in multiple units so that the recirculation rate can be changed as found necessary.

Pumps for lifting the flow-through sewage should have adequate capacity to pump the peak flows through the plant. The pumps should be installed in multiple units to take care of diurnal variations in flow which will approximately be the same as the sewage inflow to the plant. It will further be necessary to provide storage in the suction well equal to about 10 min of discharge capacity of the lowest duty pump. Float control arrangements are desirable in the suction well for controlling the number of pumps in operation.

In all these cases, at least one pump should be provided extra as a standby. Also, in the case of recirculation pumps, flow measuring and recording devices are desirable on the discharge line so that a record can be kept of the recirculation ratio actually employed in the plant.

14.4 OPERATIONAL PROBLEMS

Ponding or clogging of the filter media is one of the important operational problems in trickling filters. Ponding decreases filter ventilation, reduces the effective volume of the filter and lessens filter efficiency. Ponding or clogging is due to excessive organic loading, inadequate hydraulic loading and inadequate size of media. Remedies consist of raking or forking the filter surface, washing the filter by applying a high pressure stream of water at the surface, stopping the distributor to allow continuous heavy point by point dosing or chlorinating the influent with a dose not exceeding 5 kg/100m^2 of filter area.

Filter flies pose another serious operational problem in trickling filters. The problem is more intense in the case of low rate filters. In high rate filters fly breeding occurs mainly on the inside walls of the filter. The problem can be reduced by (a) removing excessive biological growth by the previously discussed methods, (b) flooding the filter for 24 hours at weekly or biweekly intervals, (c) jetting down the inside walls of the filter with a high pressure hose, (d) chlorinating the influent (0·5 to 1·0 mg/l) for several hours at one to two week intervals and (e) applying insecticides. The insecticide should be applied to the filter side walls and surface at intervals of 4—6 weeks. Development of resistant strains should be guarded against.

Filter odours also present a problem in trickling filter operation. Odours are most serious when treating septic effluents in low rate filters. Odours can be controlled by providing recirculation and maintaining a well ventilated filter.

In conditions of extreme cold weather, ice cover may form on the surface of the bed. Reduction of the recirculation flow, adjustment of nozzles or construction of wind breakers are methods used to reduce icing problems.

CHAPTER 15

STABILIZATION PONDS

Stabilization ponds are open, flow-through earthen basins specifically designed and constructed to treat sewage and biodegradable industrial wastes. Stabilization ponds provide comparatively long detention periods extending from a few to several days when the putrescible organic matter in the waste gets stabilized by the action of natural forces.

15·1 CLASSIFICATION

Stabilization ponds are classified as aerobic, anaerobic or facultative depending upon the mechanism of waste purification. The aerobic pond functions aerobically throughout its depth with all the oxygen needs being met by algal photosynthesis. The pond is kept shallow, with depths less than 0·5 m and the contents are stirred occasionally to prevent anaerobic conditions in the settled sludge. In the anaerobic pond, the purification results mainly from methane fermentation owing to the large depths employed. The process is somewhat attended by septic odours and the effluent will be only partially purified. Pond depths usually range from 2·5 to 4 m. This type of pond finds use mainly in the treatment of strong industrial wastes and has limited application for the treatment of sewage. The facultative pond functions aerobically at the surface while anaerobic conditions prevail at the bottom. The aerobic layer acts as a good check against odour evolution from the pond. The treatment effected by this type of pond is comparable to that of conventional secondary treatment processes. The facultative pond is hence best suited and most commonly used for treatment of sewage.

The discussion in this chapter is, therefore, confined to facultative ponds. Aerated lagoons are also discussed in this chapter as a modification of the system employing mechanical aeration devices.

15·2 MECHANISM OF PURIFICATION

In a facultative pond, the influent organic matter is stabilized partly by methane fermentation in the bottom layers and partly by bacterial oxidation in the top layers. When the sewage enters the pond, the suspended organic matter in the influent as well as the bioflocculated colloidal organic matter settle to the bottom of the pond. In the absence of dissolved oxygen at the pond bottom, the settled sludge undergoes anaerobic fermentation with the liberation of methane which represents a BOD removal from the system, 0·25 gramme of methane being liberated for every gramme of ultimate BOD stabilized. In the liquid layers of the pond, algae begins to grow under favourable conditions. The algae utilizes the carbon dioxide in the sewage for photo synthesis during day light hours, liberating oxygen which maintains aerobic conditions in the upper layers of the pond. The aerobic conditions promote the oxidation of organic waste matter by the aerobic bacteria. Thus it is seen that there is an interdependence between algae and bacteria with the algae supplying oxygen required by the bacteria and the bacteria making available the carbon dioxide required by the algae. This interrelationship is referred to as algae-bacteria symbiosis.

15·2·1 **Diurnal Variation**

Both the dissolved oxygen and the pH of the pond are subject to diurnal variation due to photosynthetic activity of algae which is related to incident solar radiation. A high dissolved oxygen concentration upto about 4 times the saturation value may be observed in the afternoon hours. Simultaneously, the pH value may reach a maximum of 9·0 or more due to the conversion of carbon dioxide to oxygen. Towards the evening or in the night, when photosynthetic activity decreases or stops, there is a gradual decrease in both dissolved oxygen and pH. In properly designed ponds, the dissolved oxygen does not completely disappear from the top layers and the pH does not fall below the influent pH.

15·2·2 **Odour Control**

In a facultative pond, the nuisance associated with anaerobic reactions is eliminated due to the presence of oxygen in the top layers. The foul smelling end products of anaerobic degradation which permeate to the top layers are oxidised in an aerobic environment. Furthermore, due to a high pH in top layers, compounds such as organic acids and hydrogen sulphide, which would otherwise volatilise from the surface of the pond and cause odour problems, are ionised and held back in solution.

15·2·3 **Algae**

In stabilization ponds, the significant algae are green algae which include *Chlorella, Scenedesmus, Hydrodictyon Chlamydomonas* and *Ankistrodesmus* and bluegreen algae which include *Oscillatoria, Spirulina, Merismopedia* and *Anacystis. Chlorella, Scendesmus* and *Hydrodictyon* possess relatively high oxygen donation capacity per unit weight. However, it is not practical to promote the growth of any particular type of algae in a pond which will depend on such factors as temperature, characteristics of the waste and intensity of sunlight. Concentration of algae in a stabilization pond is quite high and gives the pond effluent a typical dark green colour. Floating blue-green algal mats may develop in ponds during summer months. They are undesirable since they lead to insect breeding and cut out sunlight.

15·3 DESIGN CONSIDERATIONS

15·3·1 Surface Area

The amount of oxygen that can be produced by photosynthesis and the BOD that can be satisfied per unit area of a facultative pond depends mainly on the quantum of sunlight falling on the pond surface which in turn depends on the latitude of the pond site, its elevation above MSL, time of the year and the sky clearance.

Recommended BOD loadings (IS : 5611) for different latitudes are given in Table 15·1.

Table 15.1

Permissible area BOD loading at Different latitudes

Latitude °N	Areal BOD_5, Loading kg/ha/d
36	150
32	175
28	200
24	225
20	250
16	275
12	300
8	325

The recommended BOD loadings are for municipal sewage and are inclusive of the BOD of the settleable solids in the wastes. The values are applicable to towns at sea levels and where the sky is clear for nearly 75% of the days in a year.

The values may be modified for elevations above sea level by dividing by a factor, $(1+0\cdot003)$ El, where El is the elevation of the pond site above MSL in hundred metres. A further correction in the pond volume has to be made when the sky is clear for less than 75% of the davs at the rate of 3% for a fall of every 10%.

At the recommended BOD loadings, sufficient photosynthetic oxygen will be produced to stabilise about 90% of the influent BOD. Also, aerobic conditions will be maintained in the top layers of the pond at most of the time and there will generally be no evolution of septic odours from the pond.

When the ponds are intended to serve small communities such as institutions or when they are to be located close to residences, it will be prudent to adopt lesser BOD loadings than recommended in Table 15·1 so as to fully ensure the absence of septic odours.

In any system, the individual pond area should not exceed 40 ha. However, it is desirable to have more than one pond in any system requiring 0·5 ha or more pond area.

15·3·2 Detention Period

The detention period of a pond should be adequate for the bacteria to stabilise the BOD to the desired degree. The removal of the BOD in the pond follows a unimolecular pattern

$$Y = L\,(1-10^{-P_T \cdot t})$$

$$\text{or} \quad t = \frac{1}{P_T} \cdot \left(\log \frac{L}{L-Y}\right) \qquad \ldots\ldots\ldots\ldots (15\text{—}1)$$

where

Y = BOD removed;

L = Influent BOD;

P_T = BOD removal rate constant for the pond; and

t = detention period in days.

The rate coefficient at 20°C (P_{20}) is found to be approximately 0·1 per day.

The removal rate constant at other temperatures may be determined by the formula. $P_T = P_{20}\,(1\cdot047^{\,T-20})$ ....(15—2)

15.3.3 Depth

Having determined the surface area and detention capacity or volume of a pond, it becomes necessary to consider the depth of the pond only in regard to its limiting values. Shallow depths in facultative ponds will allow growth of aquatic weeds in the ponds while excessive depths will cause the entire pond contents to turn anaerobic. The optimum range of depth for facultative ponds is 1·0—1·5 m. When depth determined from area and detention period works out lesser than 1·0 m, the depth should be increased to 1·0 m, keeping surface area unchanged. When the depth works out to be greater than 1·5 m, the depth should be restricted to 1·5 m and surface area increased to give the required detention period.

15.3.4 Sludge Accumulation

The rate of sludge accumulation in facultative ponds depends primarily on the suspended solids concentration in the influent wastes. The reported rate of sludge accumulation in ponds treating municipal sewage ranges from 0·05 to 0·10 m^3/capita/year. A value of 0·07 m^3/capita/year forms a reasonable assumption in design. In multiple cell ponds operated in zones, most of the sludge accumulation will be in the primary cells.

Continued sludge accumulation in ponds over many years will cause (i) sludge carryover into the effluent, (ii) development of aquatic weeds on sludge banks, and (iii) reduction in pond efficiency due to reduction in the detention period. Facultative ponds therefore require periodical desludging at intervals ranging from 6 to 12 years.

15.4 CONSTRUCTION DETAILS

15.4.1 Site selection

Facultative pond sites should be located as far away as practicable (at least 200 m) from habitations or from any area likely to be built up within a reasonable future period. If practicable the pond should be loacted such that the direction of prevailing wind is towards uninhabited areas. The pond location should be downhill of ground water supply source to avoid their chemical or bacterial pollution. Special attention is required in this regard in porous soils and in fissured rock formations.

The pond site should not be liable to flooding. The elevation of the site should permit the pond to discharge the effluent by gravity at MWL. The site should preferably allow an unobstructed sweep of wind across the pond, open to the sun and not shaded by trees. Advantage should be taken of natural depressions while locating the ponds.

15.4.2 **Pretreatment**

It is desirable to provide medium screens and grit removal devices before facultative ponds.

15.4.3 **Construction in Stages**

In cases where the design flow will materialise only in the course of time, it is important to design facultative ponds in multiple cells and construct the cells in stages. Otherwise, the small flows in the initial years may not be able to maintain satisfactory water levels in the ponds and objectionable weed growths and mosquito breeding may develop in the installations.

15.4.4 **Multiple Units**

Multiple cells are recommended for all except small installations (0·5 ha or less). Multiple cells in parallel facilitate maintenance as any one unit can be taken out of operation temporarily for desludging or repairs without upsetting the entire treatment process. The parallel system also provides better distribution of settled solids.

Multiple cells in series enable better coliform removal and reduced algal concentration in the effluent. The series system implies a high BOD loading in the primary cells and to avoid anaerobic conditions in these cells they should have 65-70% of the total surface area requirements.

A parallel-series system possesses the advantages of both parallel and series operations. A convenient arrangement for this system consists of three cells of equal area, of which two are in parallel and serve as primary ponds and the third serves as secondary pond in series.

15.4.5 **Pond Shape**

The shape should be such that there are no narrow or elongated portions. Round, square or rectangular ponds with length not exceeding three times the width are acceptable. Maximum basin length of 750 m is generally adopted. The corners should always be rounded to minimise accumulations of floating matter and to avoid dead pockets.

15.4.6 **Balanced Cutting**

Ponds are usually constructed partly in excavation and partly in embankment. The volume of cutting and the volume of embankment should be balanced to the maximum extent possible in order to economise construction costs.

15.4.7 **Embankment**

Embankment materials usually consist of material excavated from the pond site. The material should be fairly impervious and free of vegetation and debris. The embankment should be compacted sufficiently. The top width of the embankment should be atleast 1·5 m in small ponds and 3·0 m in large ponds (length of embankment exceeding 1·0 km) with a jeep track formed on top to facilitate inspection.

The free board should be at least 0·5 m in ponds less than 0·5 ha in area. In larger installations, the free board should be designed for the probable wave heights and should be at least 1·0 m.

Embankment slopes should be designed based on the nature of soil, height of embankment and protection proposed against erosion. Outer slopes are generally 2·0—2·5 horizontal to vertical. Inner slopes are made 1·0—1·5 when the face is fully pitched and flatter, 2·0—3·0, when the face is unprotected. Inner slopes should not exceed 4 as flatter slopes create shallow areas conducive to the growth of aquatic weeds.

The outer faces of the embankments should be protected against erosion by turfing. The inner faces should preferably be completely pitched to eliminate problems of erosion and growth of marginal vegetation. Pitching may be by rough stone revetment or with plain concrete slabs or flat stones with adequate gravel packing. When complete pitching is not possible, at least partial pitching from a height 0·3 m above water line to 0·3 m below water line is necessary and the face above the line of pitching should be turfed to the top of embankment.

15.4.8 **Pond Bottom**

The pond bottom should be level, with finished elevations not more than 0·10 m from the average elevation. The bottom should be cleared of all vegetation and debris. The soil formation of the bottom should be relatively tight to avoid excessive liquid losses due to seepage. Where the soil is loose, it should be well compacted. Gravel and fractured rock areas must be avoided.

15.4.9 **Influent Lines**

When the sewage is to be pumped into the pond, the pumping main may be laid along the bottom of the pond to discharge through an upward inclined 90° bend. In gravity flows, the outfall sewer should terminate at a manhole located as close to the pond as practicable. The invert of the manhole should be atleast 0·2 m above the MWL of the pond. From the terminal manhole, the influent piping may be carried into the pond grade, supported on masonry pillars. A tee or bend should be provided to discharge the wastes at about 0·30 m depth below the liquid surface

A concrete apron of adequate size should be provided below the discharge end of the influent line to prevent erosion of the pond bottom. The influent line itself should be provided with firm supports which will not yield after the pond bottom becomes saturated and cause the influent pipeline to break.

The influent pipeline should extend into the pond atleast 15—20 m from the water edge. Multiple inlets will distribute the settleable solids in the wastes over a wide area and reduce the problem of septicity around the inlets and are desirable except in small ponds.

15.4.10 Pond Outlets

Multiple outlets are desirable except in small ponds and may be provided at the same rate as for inlets, one for every 0·5 ha pond area. The outlets should be so located with reference to the inlets as to avoid short circuiting. The outlet structures may consist either of pipes projecting *into the ponds or weir boxes. In the former* case vertical tees and in the latter case hanging baffles submerged to a depth of 0·25 m below the water surface should be provided to ensure that floating algal scum is not drawn along with the effluent.

When the outlet structure is a weir box, it is desirable to provide adjustable weir plates so that the operating depth in the pond can be altered if required.

15.4.11 Pond Interconnections

Pond interconnections are required when ponds are designed in multiple cells in series. These interconnections should be such that the effluent from one cell withdrawn from the aerobic zone can be introduced at the bottom of the next cell. Simple interconnections may be formed by pipes laid through the separating embankments. At their upstream ends, the interconnecting pipes should be submerged about 0·25 m below the water level. The downstream ends may be provided with a bend, facing downward, to avoid short circuiting by thermal stratification, care being taken to prevent erosion of the embankment or the pond bottom. Where the pond effluent is to be used for farming and involves pumping, the outlet pipe should be led to a sump of adequate capacity (30 minutes at the rate of pumping).

15.4.12 Other Aspects

Provision should be made for flow measurement both at inlet and outlet of the ponds. Wherever practicable, facilities should be available to drain out the pond completely by gravity through a sluice arrangement. The pond site should be fenced to prevent entry of cattle and discourage trespassing. Public warning boards should also be put up near the ponds clearly indicating that the pond is a sewage treatment facility.

15.5 OPERATION AND MAINTENANCE

In commissioning a pond, the sewage should be allowed to fill the pond gradually to a depth of about 30 cm and this level maintained by admitting periodically, a small quantity of raw sewage. Algal growth will establish itself naturally, without requiring any artificial seeding. After the first algal bloom has established itself which may take a week or two, further raw sewage is admitted gradually at a rate not exceeding the design rate until the entire pond is filled up. The pond is then allowed to rest for two to three days to ensure that the algal growth has fully established. The pond at this stage is ready for continuous inflow of sewage.

Once the pond is in normal operation, the effluent and influent pipings should be inspected daily to check that they are not blocked. The pond embankments should be inspected periodically to examine whether there is any damage to them by burrowing animals.

It is very important to ensure that weeds and grass do not grow from the bank into the water. The environment in a facultative pond is not conducive to mosquito breeding; but, if there is vegetation at the water line, sheltered pockets will harbour mosquito larvae and a serious health and nuisance problem may arise. Where the inside slopes of the embankments are fully lined, there will be no growth of marginal vegetation and little attention will be required for its control.

In summer months, blue green algae may grow vigorously in the ponds giving rise to floating mats of algae. The algae in the mats may then die and give rise to odours. The algal mats may also attract flies. The growth of algal mats should therefore be controlled by frequent removal in the case of small ponds and by breaking up the mat from a boat and allowing them to sink in the case of large ponds.

Facultative ponds require desludging at long intervals when sludge has accumulated to an extent affecting pond performance. Desludging may be done by emptying the pond upto the top level of the sludge and allowing the sludge to dry out in the sun. The dried sludge can be removed and sold as fertiliser. Adequate thought should be given even at the time of design and construction of the ponds to the method of desludging that will be adopted. In multicell serial ponds, desludging may be required only in the first pond.

15.6 PERFORMANCE

The high concentrations of algae in the pond effluent will exert high BOD in the standard laboratory BOD test involving darkroom incubation and will also give high SS values. The BOD_5 and SS values may each be in the range of 50-100 mg/l. However, the effluent will not cause nuisance when disposed of on land or discharged into receiving waters because the algal cells do not readily decompose or exert oxygen demand under natural conditions. In fact, the algae increases the oxygen levels in the receiving waters by continued photosynthesis.

Because of the above reasons, the standard BOD and SS tests are not considered useful for evaluating the quality of facultative pond effluents. The quality is usually assessed based on the BOD_5 of the filtered effluent, the assumption being that the suspended solids in the effluent is all algae. The filtration procedure adopted for the test is the same as for the suspended solids test.

Well designed facultative ponds give about 80—90% BOD reduction based on the filtered BOD_5 of the effluent.

Facultative ponds also effect high bacterial reduction, the efficiency being particularly high in multicell ponds operated 'in series'. Coliform and faecal streptococci, removals are as high as 99.99%. Intestinal pathogens belonging to *Salmonella* and *Shigella* groups are reported to be completely eliminated in stabilisation ponds. Cysts of *Entamoeba histolytica* and helminthic larvae are also eliminated.

15.7 APPLICATIONS

The facultative pond is simple and cheap to construct. It does not require skilled operation and is easy to maintain. Properly designed, the pond also gives consistently good performance. The facultative pond has therefore become very popular for municipal and institutional sewage treatment. The method is suited wherever land is cheap and readily available and may be used for treating sewage either for discharge into streams or lakes or for use on land. The method is particularly useful for interim sewage treatment when due to lack of funds or due to meagre flow in the initial stages, it is considered inexpedient to construct initially the conventional treatment plant envisaged ultimately.

The facultative pond is also suited for the treatment of industrial wastes which are biodegradable provided the wastes are not coloured and do not contain substances toxic to algae.

15.8 AERATED LAGOONS

In contrast to waste stabilization ponds where oxygen required for stabilizing the organic matter is furnished by algae, the oxygenation is provided in aerated lagoons by mechanical surface aerators installed on floats or rafts or on fixed platforms. In the aerated lagoon, the depth varies from 2.5 to 4 m, and the land requirements are much less because of the greater depths and smaller detention times needed for the stabilisation of organic matter. The lagoons can be operated either at a low rate of aeration not adequate to keep all the solids in suspension but enough to keep the top layers aerobic or to keep all solids in suspension by a greater amount of aeration. In the former called the facultative lagoon, the sewage solids tend to settle down and anaerobic bottom is established whereas in the latter called aerobic-flow-through type, the entire pond is aerobic.

In the facultative type, SS concentration varies from 30 to 150 mg/l with detention times ranging from 3 to 5 days. Oxygenation requirements are about 0.8 kg per kg of BOD_5 removed and the power requirements vary from 0.8 to 1.0 kw per 1000 m^3. The BOD removals are of the order of 75 to 90%, but there is need for removal of accumulated sludge after some years.

In the aerobic-flow through types, SS concentration may be anywhere between 60 and 300 mg/l with detention times of 2 to 10 days. Oxygenation requirements are of the order of 0.7 to 1.3 kg per kg of BOD_5 removed and the power requirements vary from 0.75 to 2.25 kw per 1000 m^3. The BOD removals are only of the order of 75 to 85%, since the sludge solids are not allowed to accumulate and are carried along with the effluent.

CHAPTER 16

SLUDGE DIGESTION

The principal objective of sludge digestion is to subject the organic matter present in the settled sludge of the primary and final sedimentation tanks to anaerobic or aerobic decomposition so as to make it innocuous and amenable to dewatering on sand beds or mechanical filters before final disposal on land, lagoon or sea. Regardless of the processes used, sludge digestion brings about a reduction in its volume. While anaerobic digestion of sludge produces gas which can be utilised wherever feasible, aerobic digestion does not produce any utilisable byproduct other than well stabilized sludge.

16.1 ANAEROBIC DIGESTION

Anaerobic digestion is the biological decomposition of organic matter in the absence of oxygen. When an organic sludge undergoes, decomposition anaerobically, the organic solids are hydrolysed, liquefied and gassified and as a result, a well mineralised sludge is obtained. Not all the organic matter in sewage sludge is stabilized under this condition, since some of the constituents are so complex that they resist biological decomposition. The objective of sludge digestion is therefore, to stabilize only the biodegradable matter that resist dewatering either on sand beds or in mechanical filters.

There are primarily two sets of reactions in anaerobic digestion, the first being known as acid fermentation and the second as methane fermentation. In the first set of reactions, the organic matter is hydrolysed and liquefied resulting in the production of acetic, propionic, butyric and other volatile acids. In the second set of reactions, the volatile acids produced by the first group of organisms are utilised by the second group of methane fermenters with the formation of methane and carbon dioxide. For proper functioning of the process, it is necessary that a good balance between the two reactions is maintained so that there is no accumulation of excess volatile acids, normal operating values, being 200 to 400 mg/l with a maximum of 2000 mg/l. This sequence of reactions brings about a reduction of the volatile solids present in the sludge thereby permitting easy dewatering on a sand bed or a vacuum filter.

16.2 DIGESTER CAPACITY

Based on the information available on characteristics of sewage of some of the principal cities in the country, the average per capita suspended solids can be taken to be 90 gm/day, of which 60% settles as sludge with volatile solids in the sludge seldom exceeding 70%. The capacity of digesters depends on :

(i) daily volume and moisture content of input sludge and digested sludge;

(ii) temperature of digestion;

(iii) desired degree of destruction of volatile solids; and

(iv) storage capacity for digested sludge.

16.2.1 Volume of Sludge

The volume of daily sludge varies depending upon the degree of removal of suspended solids in primary and final settling tanks, moisture content and specific gravity of the sludge. In the case of sludge continuously desludged from primary sedimentation tanks, solids do not generally exceed 4 to 5% while in secondary tanks, they range between 0.5 and 1%. When the secondary sludge is combined with the primary sludge the solids in the mixed sludge range between 2.5 to 5%. The solids content of digested sludge is usually in the range of 6 to 13% when reduction in bulk takes place due to separation of supernatant.

Typical figures of solids in sludges are given in Table 16.1.:—

Table 16.1

Solids in sludges by various treatment processes

(Per capita per day)

Treatment Process	Total (gms)	volatile (gms)	Non volatile or fixed solids	Sp.gr of dry solids	% Solid in wet sludge
(1)	(2)	(3)	(4)	(5)	(6)
Total S S in Sewage	90	63	27	1·22	
Settleable solids (60%)	54	38	16	1·22	
Non-Settleable solids (40%)	36	25	11	1·22	

(1)	(2)	(3)	(4)	(5)	(6)
I. *Primary Sedimentation & Digestion*					
1. Solids removed as fresh sludge	54	38	16	1·22	4 to 5%
2. Solids digested*	..	—25	+6		
3. Solids remaining in Pr. digested sludge	35	13	22	1·60	10 to 13%
II. *Activated Sludge Process*					
Non settleable solids affected	36	25	11	1·22	
Solids digested during activation (10%)	..	—2·5	+0·5		
Solids to be removed	34	22·5	11·5		
Solids removed as fresh activated sludge (90%)	31	20	11	1·25	
Combined Primary (I) & Secondary Excess A.S.(II)	54+31 =85	38+20 =58	16+11 =27	1·22	2·5 to 4%
Solids digested*	..	—38	+10		
Solids remaining in digested combined Pr. & Excess AS	57	20	37	1·65	6 to 7%
III. *T. F. Process*					
Non-settleable solids	36	25	11	1·22	
Solids digested during filtration (10%)	..	—2·5	+0·5		
Solids to be removed in Sed. tank	34	22·5	11·5		
Solids removed in S.S. tank (40%)	14	9	5		
Combined Pr. (I) and T. F. Humus (III)	68	47	21	1·22	4 to 5%
Solids digested in digester*	..	—32	+8		
Solids remaining in digester Pr. & T. F. Humus	44	15	29	1·66	8 to 10%

* It is assumed that 2/3 of the volatile matter is destroyed during digestion and one fourth of the digested matter is converted to non-volatile residual sludge.

Percentage volatile and non-volatile matter is assumed to be 70% and 30% respectively.

Sp. gravity of volatile and non-volatile matter is assumed as 1.0 & 2.5 respectively.

The Specific gravity of dry solids Sd, is given by

$$\frac{100}{S_d} = \frac{\%\ \text{organic or volatile matter}}{\text{Sp. gr. of organic matter}} + \frac{\%\ \text{mineral matter}}{\text{sp. gr. of mineral matter}} \quad \ldots(16\text{-}1)$$

Generally the Sp. gr. of organic matter varies from 1.0 to 1.2 and that of mineral matter from 2.4 to 2.65.

With the known percentage solids or % moisture in the wet sludge, the Sp. gr. of wet sludge S_w could be worked out from the relationship.

$$\frac{100}{S_w} = \frac{\%\ \text{moisture}}{\text{Sp. gr. or water}} + \frac{\%\ \text{of solids}}{S_d} \quad ..(16.2)$$

Once the sp. gr. of wet sludge and % of solids in wet sludge are known, volume of wet sludge could be determined both for raw and digested sludges.

16.2.2 Temperature

Digestion of sludge is temperature dependent, the time of digestion varying with temperature as shown in Table 16.2.

Table 16.2

Temperature in °C	10	15	20	25	30	35	40	45	50	55	60
Digestion period (days)	75	56	42	33	27	24	25	23	16	14	18
	Mesophilic						Thermophilic				

Upto a temperature of 40°C, the digestion is brought about by a particular type of organisms while a totally different type of organisms establish in the digester at temperatures higher than 45°C. The former range is called the mesophilic range and the latter thermophilic. Normally digesters are not operated in the thermophilic range as it would warrant special heating and also is beset with operational and maintenance problems. In the mesophilic zone, the range of 25° to 40° gives reasonably short detention periods.

The ambient temperature in the country is generally favourable for operation under mesophilic conditions throughout the year. But in special conditions, where extremely low temperatures are likely to be encountered, it may be necessary to heat the digesters for specific periods in the year to maintain the digesters at favourable temperatures.

Heating of digester contents as a routine is not practised in the country because of the fact, that except in a few places, the ambient temperature is suitable to maintain a reasonable rate of anaerobic activity. Further, the digesters are generally designed with extra capacity for storage of digested sludge in monsoon seasons where sand drying is adopted, which serves as a buffer capacity for winter season by providing increased detention periods. Considering these factors it seems reasonable to adopt a temperature range between 25 and 30° C for design purposes.

The percent volatile matter destroyed is dependent upon the percentage volatile matter in raw sludge. The detention periods given in Table 16.2 would accomplish about two-thirds destruction of volatile solids for a 70% volatile solids normally encountered in the raw sludge. The different values of volatile solids destruction when digestion is considered satisfactory are given in Fig. 16.1.

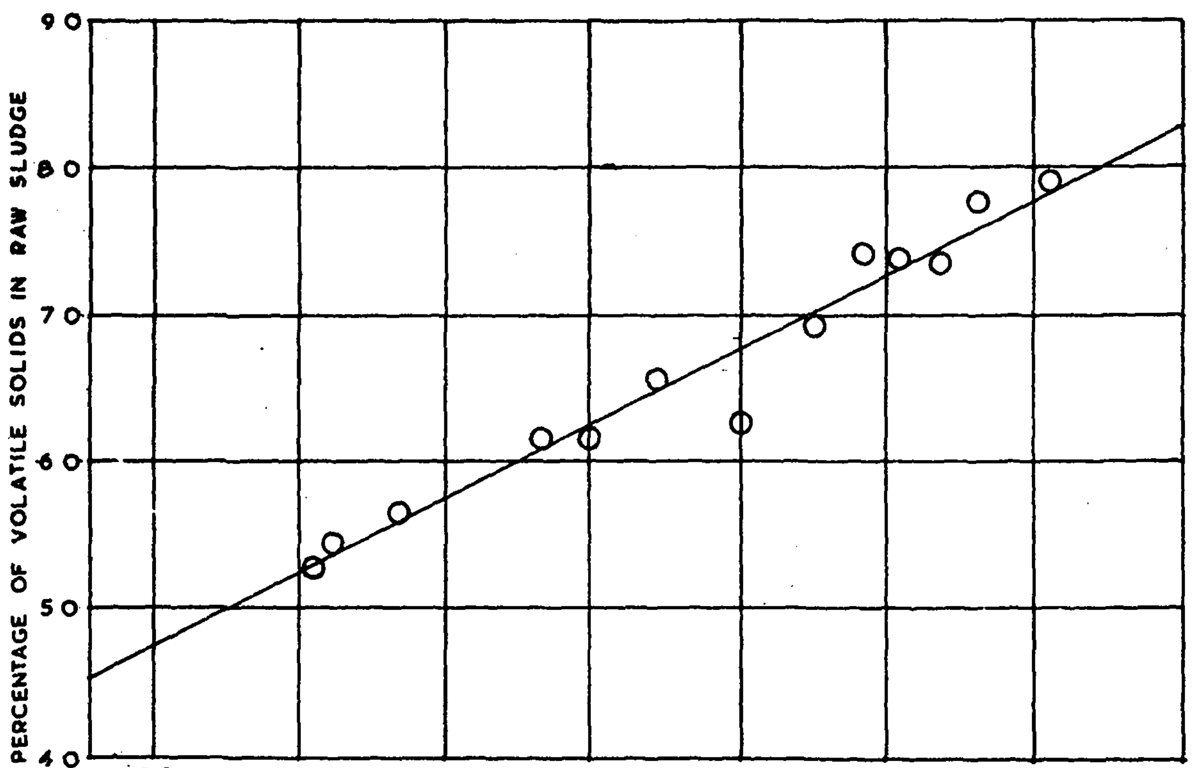


FIG.16.1 REDUCTION IN VOLATILE MATTER BY DIGESTION

16.2.3 Storage for Digested Sludge

Storage capacity for digested sludge is required in places where digested sludge is applied to drying beds for dewatering as loading on such beds is interrupted during peak monsoon periods. The longest monsoon period which may be anticipated in this country is about 8 weeks. However, where meteorological data is available, such data should be used to determine the capacity needed for storage.

16.2.4 Conventional Digestion

Lack of proper mixing in the conventional digesters leads to stratification, giving rise to distinct layers of scum, supernatant, actively digesting sludge and digested sludge. The supernatant is withdrawn periodically and returned to the influent of the treatment plant while the sludge is added at mid-depth and withdrawn from the bottom.

In this type of digesters, much of the digester volume is wasted and sometimes acidification takes place in the top and middle layers while methane fermentation is confined to the lower layers. This can lead to areas of low and high pH in the system, which restrict optimum biological activity. Also chemicals added for pH control are not dispersed throughout the tank and their effectiveness is limited. Grease breakdown is poor because the grease tends to float to the top of the digester, while the methane bacteria are confined to the lower levels. Methane bacteria are removed with the digested sludge and are not recycled to the top, where they are required. During progression from top to bottom of the digestion tank, the sludge is compressed and gradually dewatered.

Since the progress of digestion is parabolic the capacity of the digester is given by the expression :—

$$V=[V_f-\tfrac{2}{3}(V_f-V_d)]\,T_1+V_dT_2 \quad (16\text{-}3)$$

Where

V=Volume of digester;

V_f=Volume of fresh sludge added per day;

V_d=Volume of digested sludge withdrawn per day;

T_1=Digestion time in days; and

T_2=Monsoon storage in days.

The digestion can be in a single unit or in two units the primary and the secondary, the former being provided with the needed time for digestion and the latter to meet the requirements of monsoon storage.

Useful checks of the digester capacities can be made based on the following parameters:

(i) per capita capacity—

(a) primary sludge ... 0·05—0·075 m^3

(b) combined sludge... 0.10—0.15 m^3

(ii) Loading factor—

(a) primary or combined sludge 0.3 to 0.75kg/m^3/day

Typical problem relating to digestion of combined sludge is given in Appendix 18.

16.2.5 High Rate Digestion

In high rate digestion, the sludge is more or less continuously added and vigorously mixed either mechanically or by recirculating a portion of the digestion gases through a compressor. The digester is also heated to maintain maximum activity in the mesophilic region. High rate digestion is particularly useful for upgrading conventional digester units to accommodate additional loads where land is scarce. Because of good mixing, there is no stratification and hence loss of capacity does not arise due to supernatant or scum or dead pockets. By adopting more or less continuous addition of raw sludge and resorting to prethickening of the raw sludge to a solid content of 6%, the detention times could be reduced to 10 to 15 days. Thickening beyond 6% affects the fluidity of the sludge hampering its movement in pipes and mixing in the digester. There is no supernatant separation in the high rate digester and only about 50% destruction of VS is achieved. In case the sludge has to be applied to drying beds, a second stage digester is provided where separation of supernatant accompanied with a reduction in volume of sludge due to gravity thickening takes place and digestion is completed. Additional storage capacity needed for the monsoon period has also be provided in the second stage digester. Capacities for high rate digestion system may be determined by :

$$V' = V_fT_h \ldots\ldots (16\text{-}4)$$

$$V''=[V_f+\tfrac{2}{3}(V_f-V_d)]\,T+V_dT_2 \ldots\ldots (16\text{-}5)$$

where

V'=Volume of first stage digester,

V''=Volume of second stage digester,

T_h=detention time in the high rate digester in days and

T=detention time in the second stage digester which is of the order of 10 days.

16.3 DESIGN OF DIGESTER ELEMENTS

16.3.1 Number of Units

Conventional digesters are designed as single units for plants treating upto 4 mLd. For larger plants, units are provided in multiples of two, the individual capacity not exceeding 3 mLd.

High rate digesters are designed comprising primary and secondary digestion tanks, each unit generally capable of handling sludge from treatment plants upto 20 mLd.

16.3.2 Tank Shape and Size

Circular tanks are most common for sludge digestion and they should preferably be not less than 6 m nor more than 55 m in dia.

16.3.3 Water Depth and Free Board

Side water depth of a digester may be kept between 4.5 to 6.0 m but should not exceed 9 m even for very large tanks. The free board is dependent upon the type of cover and the maximum gas pressure. For fixed dome or conical roofs, free-board between the liquid level and the tope of the digester wall should not be less than 0.4 m. For floating covers, the free board between the water level and the top of the tank wall should preferably be not below 0.6 m. For fixed slab roofs, a free board of 0.8 m is recommended.

Depth of sludge in a digester has to be carefully worked out since too deep a digester causes excessive foaming which may result in chokage of the gas mains, building up high pressures in the digester causing a hazard. When gas production reaches a figure of about 9m^3/day/m^2 of top surface of sludge, foaming becomes noticeable.

Therefore, before the tank depth and surface area of a digester are worked out, maximum gas production rate should be determined. On an average, about 0.9 m^3 of gas is produced per kg. of volatile matter destroyed. The optimum diameter or depth of digester is calculated such that at twice the average rate of daily gas production, the value of $9 m^3/m^2$ of tank area is not exceeded.

16.3.4 Bottom Characteristics

The bottom slope should not be less than 1 in 12 to facilitate easy withdrawal of sludge. The tank floor should be designed for uplift pressure due to the subsoil water or suitably protected either by anchoring or by pressure release valves.

16.3.5 Roofing

Sludge digesters can have either fixed or floating roofs. Reinforced concrete domes, conical or flat slabs are used for fixed roof and steel domes are used for floating cover. Steel floating covers may either rest on the liquid or act as gas holders in the digesters themselves. If a floating cover is used for gas holder in a digestion tank, an effective vertical travel of 1.2 to 2 m should be provided.

Gas domes are provided in the roof at or near the centre of circular tanks. A gas take off point in the fixed or floating roof should be located at least 1 m above the maximum liquid level in the tanks.

At least two manholes on the digester roof should be provided. The size of each manhole should preferably be not less than 0.75 m to facilitate repair and maintenance through them. For tanks greater than 25 m dia, at least four manholes should be provided for removal of excessive scum blanket and other matter which are difficult to remove through sludge drawoff pipes. These also serve to give adequate ventilation of the tank at the time of repair or maintenance.

Digester covers should have pressure and vacuum release valves and flame traps. At least two access manholes with gastight and watertight covers are also to be provided in the operating gallery near the bottom of the side wall of the digesters. These manholes are necessary to conveniently enter into the digesters, for maintenance of equipment and for removal of excessive accumulation of grit and other heavy material from the digesters. While the primary digesters are always constructed with covers with gas collection arrangements, the secondary digesters are often kept open. This practice may not be objectionable since the maximum gas production and digestion take place in the primary stage itself. However, in areas where the intensity of rainfall is very high, both the primary and secondary digesters should be covered.

Access ladders or stairs to the roofs of the digesters with proper guard rail on the roof should be provided. Fixed or movable stairs should also be provided for easy entry into the digesters.

16.3.6 Digester Control Room

Normally a control room is provided near the digesters to house the piping and the process control equipment, which are principally the sludge heating units, sludge transfer and recirculation pumps, sludge sampling sinks, thermometers, blowers for ventilation and electrical control equipment. Where heating of sludge digesters is practised, the operations could be managed by locating conveniently, the necessary valves for supernatant and sludge withdrawal, in the digester wall itself. However, in sewage treatment plants having more than four digesters, it is advisable to have a separate operation control room to house the necessary control equipments for convenient operation.

16.3.7 Mixing of Digester Contents

Mixing of the digester content is an important requirement in sludge digestion. Mixing brings about intimate contact between the raw sludge fed and the digesting sludge securing advantages of seeding and buffering thereby maintaining a rapid and uniform rate of digesting in the tanks. It also minimises scum formation, temperature stratification and dead zones in the digesters.

Conventional digesters are mechanically stirred with revolving arms dipping a little below the scum level equipped with vertical fingers or pickets extending downwards in the scum stratum which pass between fixed pickets, breaking up the scum. Alternatively, draft tubes extending from below the liquid level to the lower level of the tank with an enclosed high capacity propeller to force the circulation are also used which minimise the scum formation apart from providing mixing and seeding of the sludge. Another practice of compressing the sludge gas, and discharging at three or more points in the to supernatant zone below the scum zone is also adopted for minimising the scum. Scum can also be reduced by spraying tank liquor or water when the scum is brought into the active digesting zone.

Agitation of the tank contents can be achieved by (i) withdrawing the boom sludge continuously and pumping it to the digesting sludge zone such that the entire content of the digester is turned over in a period of at least 4 hours, (ii) providing mechanical agitation, (iii) recirculating the gas continuously through perforated or diffusion pipes located at the bottom of the digesters.

16.3.8 Piping

CI is commonly used for pipelines carrying sludge including fittings and joints. Pipes should be well supported and be capable of being drained. Vents should be provided at high points in order that the gas generated by the digesting sludge does not accumulate in these pipelines. Adequate number of flanges and flexible couplings should be provided on exposed sludge lines to facilitate dismantling and insertion of cleaning equipment whenever necessary. In long pipe runs, tees with flanges equipped with 40 to 60 mm hose connections should be provided for easy cleaning and flushing of the pipe.

Flushing is an important requirement and adequate arrangements should be provided for flushing the sludge lines with water or clarified sewage.

A minimum dia of 200 mm should be used for the sludge pipelines for both gravity withdrawal and suction to pumps. Velocities of 1.5 to 2·4 mps should preferably be maintained to prevent solids deposition and accumulation of grease which ultimately clog sludge piping.

Primary and digested sludge have different hydraulic characteristics from those of water, though the secondary sludge is almost similar to water in its characteristics. The head loss in sludge pipes increases with the increase in percentage of solids and as such 'C' values of 40 to 50 in Hazen William formula should be used for designing the pipelines.

For gas lines CI or GI is commonly used. Galvanised steel may also be used for exposed gas piping. Flanged joints may be provided for exposed gas piping of sizes 100 mm and above in dia while screw or welded type joints are recommended for pipe less than 100 mm. Mechanical joints should be used for underground piping. It is necessary that all gas piping be located at a level that will allow proper draining of the condensate. It is desirable to maintain a gas pipe slope of 1 in 50 with a minimum slope of 1 in 100 for adequate drainage. Gas pipes should preferably be painted with bituminous coating. For dia of 100 mm and above, cast iron with flanged gasketed joints or flexible mechanical joints may be used. Adequate pipe supports should be provided to prevent breakage. It is desirable to provide a flanged pipe bypass before a gas meter. A firm foundation should also be laid below the pipe and caution must be exercised during back filling to prevent any disturbance of the alignment and grade. In highly acidic or alkaline soils, the pipe must be wrapped with either asbestos or some other protective material. Coal tar enamel may also be used in some cases. Cathodic protection is not generally needed on gas lines. Adequate number of drip traps must be provided in gas pipelines, especially at the downward bends. Suitable number of tees should also be provided with removable screwed plugs or flanges for cleaning purposes. A drip trap of 1 litre capacity would be satisfactory. Trap outlets should run to floor drains wherever convenient. It is preferable to use positive type traps which prevent the gas from escaping while emptying the condensate.

16.3.9 Sampling Sinks and Valves

A sink should be provided for each digester unit for drawing the supernatant liquor and sludge from various levels in the digester. Sinks should either be of white enamelled cast iron or of stainless steel. They should be made at least 30 cm deep. The supply of adequate water for flushing the sinks should also be provided.

The sludge sampling pipes usually of GI should be short and between 40 to 50 mm in dia. These pipes may be arranged so as to draw samples from at least three levels in the digester at 0.6 m intervals. Sink valves should be either brass plug type or CI flanged type.

16.3.10 Liquid Level Indicator

The digester may be designed for a fixed liquid level. Alternatively, a liquid level indicator with gauge board or any other positive level measuring device may be used for each digester.

16.3.11 Gas Collection

Sludge gas is normally composed of about 60 to 70% methane and 25 to 35% carbon dioxide by volume, with smaller quantities of other gases like hydrogen sulphide, hydrogen, nitrogen and oxygen. The combustible constituent in the gas is primarily the methane. Depending upon the sulphate content of the sewage and the sludge, the concentration of hydrogen sulphide in the gas varies. Hydrogen sulphide in addition to its corrosive properties imposes a limit on the utilisability or causes nuisance during the burning of the gas, Sludge gas containing 70% methane has a fuel value of about 5,800 k cal/m^3. In terms of solids digested, the average gas production is about 0.9 m^3/kg of volatile solids destroyed at a normal operating pressure of 150 to 200 mm of water.

Minimum and maximum rates of gas production will however depend upon the mode of feeding of raw sludge into the digester. Where batch feeding is practised, the minimum and maximum gas production rates may vary from 45% to more than 200%. In the continuous feeding system, the difference between the maximum and the minimum is considerably reduced. Intermittent mixing of digester contents is also responsible for wide fluctuations in gas production rates.

It is, therefore, desirable to feed the high rate digesters with raw sludge and run the mixing device as continuously as possible to obtain not only a uniform rate of digestion but also uniform production of gas.

Sludge gas should be collected under positive pressure to prevent its mixing with air and causing explosion. The explosive range of sludge gas is between 5 to 15% by volume of gas with air. The gas may be collected directly from under a floating cover on the digester or from the fixed cover by maintaining a constant water level. Where primary and secondary units are provided to operate in series with the primary having a fixed cover and the secondary with a gas holder or floating cover, the gas piping from each digester should be interconnected. A separate gas holder may be provided to collect the gas from the primary unit where the secondary unit is kept open.

A gas dome above the digester roof should be used for gas take off. The velocity in sludge gas piping should not exceed 3.5 mps to prevent carry over of the condensate from the condensate traps and avoid high pressure loss and damage to meters or flame traps and other appurtenances of the system.

An integrating meter made of corrosion resistant material should be used to measure gas production from the digesters. Removal of condensate from the meter is also desirable. Pressure release valves are provided for controlling the gas in the digester by releasing gas pressure exceeding 200 to 300 mm of water and also preventing partial vacuum and possible cover collapse during rapid withdrawal of sludge or gas.

A distance of at least 30 m should be kept between a waste gas burner and a digestion tank or gas holder to avoid the possibility of igniting the gas mixture. Waste gas burners should be located in the open for easy observation. A pilot device should also be provided with the waste gas burners. Condensate traps, pressure release valves and flame traps should also be provided ahead of waste gas burners. Manometers indicating the gas pressure in cm of water may be used on the main gas line from the digester or ahead of the gas utilization device. A common open end U tube manometer should not be used for such purposes as it may be hazardous.

16.3.12 Gas Holder

The primary purpose of a gas holder is to adjust the difference in the rate of gas production and consumption as well as to maintain a uniform pressure at the burner. When gas holders are also used for storage of gas for utilisation, a storage capacity of at least 25% of the total daily gas production should be provided.

The gas holders may be of the following types : —

(i) A bell shaped cylindrical tank submerged in water installed either on the top of a digester or as a separate unit. The structure holding the water may be made of RCC. As the gas enters or leaves, the holder rises or falls.

(ii) A pontoon cover type which floats on the liquid content of the digester consisting of steel ceiling, skirt plates, a gas dome and steel trusses.

(iii) Dry type gas holder consisting of a cylindrical steel tank in which a disc-shaped piston makes contact at its periphery with the inside of the tank. The gas enters the holder from beneath the piston which floats on the gas. Leakage of gas is prevented by either tar or a felt seal around the edge of the piston. A suitable roof should be provided if this type of dry gas holder is installed.

(iv) A high pressure holder either cylindrical or spherical in shape and made of either welded or rivetted steel construction, for storing the gas under high pressure. This type of gas holder is seldom used for sewage treatment plants unless the gas has to be utilised for special purposes.

The appurtenances for gas holders include ladders, condensate drains, pressure gauges and safety valves.

16.3.13 Performance of Digesters

The following parameters are indicative of good digestion :—

(a) % of volatile solids destroyed as given in Fig. 16.1	
(b) per capita gas production	$0{\cdot}025 m^3$/day
(c) gas production per kg. of volatile matter added.	$0{\cdot}4 m^3$
(d) gas production per kg. of volatile matter destroyed.	$0{\cdot}9 m^3$
(e) pH of the digesting sludge	7 to 8
(f) methane content of gas produced.	60 to 70% with an average of 65%
(g) dry solids in digested sludge—	
Primary	10—15%
Mixed	6—10%
(h) volatile acids	200 to 400 mg/l
(j) grease	practically absent
(k) colour	black
(l) drainability	quickly drainable
(m) odour	inoffensive

16.4 AEROBIC DIGESTION

Aerobic digestion is also a useful method of stabilising sewage sludge. It can be used for secondary tank humus or for a mixture of primary and secondary sludges but not for primary sludges alone. The major advantage of aerobic digestion over anaerobic digestion are :—

(i) lower BOD concentration in digester supernatant;

(ii) production of odourless and easily dewaterable biologically stable digested sludge;

(iii) recovery of more basic fertiliser value in the digested sludge;

(iv) lower capital cost; and

(v) fewer operational problems.

Because of these advantages, aerobic digesters are being increasingly used particularly for small treatment plants. However, running cost in terms of the power cost, is much higher than for anaerobic digesters.

The factors that should be considered in designing an aerobic digester include detention time, loading criteria, oxygen requirement, mixing and process operation. The volatile solids destroyed in aerobic digestion in about 10 to 12 days time, at a temperature of 20°C would be 35 to 45%. Higher temperature will result in reduction in the period of digestion. Oxygen requirements normally vary between 1.7 to 1.9 gm/gm of volatile solids destroyed. It is also desirable to maintain the dissolved oxygen between 1 to 2 mg/l in the system. Operational difficulties may be expected if compressed aeration is practised. Extended aeration system including oxidation ditches are examples of aerobic digestion.

CHAPTER 17

SLUDGE THICKENING, DEWATERING & DISPOSAL

Sludge thickening or dewatering is adopted for reducing the volume of sludge or increasing the solids concentration to (a) permit increased loadings to sludge digesters; (b) increase feed solids concentration to vacuum filters; (c) economise on transport costs as in ocean barging in case of raw sludges; (d) minimise the land requirements as well as handling costs when digested sludge has to be transported to disposal sites; and (e) save on the auxiliary fuel that may otherwise be needed when incineration of sludge is practised.

17.1 SLUDGE THICKENING

This practice is adopted for the separation of greater amount of water from sludge solids than can be attained in settling tanks. Thickening produces a saving in unit costs compared to sludge digestion and dewatering processes.

Three types of thickening are commonly practised viz., (a) gravity thickening, (b) air floatation, and (c) centrifugation.

17.1.1 Gravity Thickening

Gravity thickening is the most common practice for concentration of sludges. This is adopted for primary sludge or combined primary and activated sludge but is not successful in dealing with activated sludge independently. Gravity thickening of combined sludge is not effective when the activated sludge exceeds 40% of the total sludge weight, and other methods of thickening of activated sludge have to be considered.

Gravity thickeners are either continuous flow or fill and draw type, with or without addition of chemicals. Use of slowly revolving stirrers improves the efficiency. Continuous flow tanks are deep circular tanks with central feed and overflow at the periphery. They are designed for a hydraulic loading of 20,000 to 25,000 lpd/m^2. Loading rates lesser than 12,000 lpd/m^2 are likely to give rise to odour problems. The normal sludges contain too much solids to permit this loading and hence it is necessary to dilute the sludge with the plant effluent. The recommended solids loading for primary, mixture of primary and trickling filter and mixture of primary and activated sludge (60:40 weight ratio) are 100,60,40 kg/day/m^2 respectively. Underflow solids concentrations that can be expected with these loadings are about 8 to 10% with primary, 6 to 8% with primary and trickling filter and 2 to 6% with primary and activated sludges. Better efficiencies can be obtained by providing slow revolving stirrers, particularly with gassy sludges.

Continuous thickeners are mostly circular with a side water depth of about 3 m. Concentration of the underflow solids is governed by the depth of sludge blanket upto 1 m beyond which there is very little influence of the blanket. Underflow aolids concentration is increased with increased sludge detention time, 24 hours being required to achieve maximum compaction. Sludge blanket depths may be varied with fluctuation in solids production to achieve good compaction. During peak conditions, lesser detention times will have to be adopted to keep the sludge blanket depth sufficiently below the overflow weirs to prevent excessive solids carryover.

It is necessary to ensure provisions for (a) regulating the quantity of dilution water needed; (b) adequate sludge pumping capacity to maintain any desired solids concentration, continuous feed and underflow pumping; (c) protection against torque overload and (d) sludge blanket detection.

17.1.2 Air Floatation

Air floatation units employ floatation of sludge by air under pressure or vacuum and are normally used for thickening of waste activated sludge. These units involve additional equipment, higher operating costs, higher power requirements, more skilled maintenance and operation. However, removal of grease and oil, solids, grit and other material as also odour control are distinct advantages.

In the pressure type floatation units, a portion of the subnatant is pressurised from 3 to 5 kg per cm^2 and then saturated with air in the pressure tank. The effluent from the pressure tank is mixed with influent sludge immediately before it is released into the floatation tank. Excess dissolved air then rises up in the form of bubbles at atmospheric pressure attaching themselves to particles which form the sludge blanket. Thickened blanket is skimmed off while the unrecycled subnatant is returned to the plant.

The vacuum type employs the addition of air to saturation and applying vacuum to the unit to release the air bubbles which float the solids to the surface.

The efficiency of air floatation units is increased by the addition of chemicals like alum and polyelectrolytes. The addition of polyelectrolytes does not increase the solids concentration but improves the solids capture from 90 to 98%.

17.1.3 Centrifugation

Thickening by centrifugation is resorted to only when the space limitations or sludge characteristics will not permit the adoption of the other two methods. This method involves high maintenance and power costs. Centrifuges employed are of either disc or solid bowl type. Disc centrifuges are prone to colgging while the latter type gives poorer quality effluent.

17.2 SLUDGE DEWATERING

Most of the digested primary or mixed sludge can be compacted to a water content of about 90% in the digester itself by gravity but mechanical dewatering with or without coagulant aids or prolonged drying on open sludge drying beds may be required to reduce the water content further. The dewatering of digested sludge is usually accomplished on sludge drying beds which can reduce the moisture content to below 70%. But excess oil or grease in the sludge will interfere with the process. Where the required space for sludge drying beds is not available, sludge conditioning followed by mechanical dewatering on vacuum filters, filter presses or centrifugation followed by heat drying or incineration could be adopted.

In most parts of the country, the climate is favourable for open sludge drying beds which is economical and easy to manage.

17.2.1 Sludge Drying Beds

This method can be used in all places where adequate land is available and dried sludge can be used for soil conditioning. Where digested slugde is deposited on a well drained bed of sand and gravel, the dissolved gases tend to buoy up and float the solids leaving a clear liquid at the bottom which drains through the sand rapidly. The major portion of the liquid drains off in a few hours-after which drying commences by evaporation. The sludge cake shrinks producing cracks which accelerates evaporation from the sludge surface. In areas having greater sunshine, lower rainfall and lesser relative humidity, the drying time may be about two weeks while in other areas, it could be four weeks or more. Covered beds are not generally necessary.

17.2.1.1 *Design criteria*

(a) AREA OF BEDS

The area needed for dewatering digested sludge its dependent on total volume of sludge, climate, temperature and location. Areas required for drying beds range from 0.1 to 0.15 m^2/ capita with dry solids loading of 80 to 120kg/m^2 of bed per year for digested primary sludge and from 0.175 to 0.25 m^2/capita with dry solids loading of 60 to 120kg/m^2/year for digested mixed sludge, for bed specifications indicated subsequently, A typical worked out example may be seen at Appendix-20,

(b) BED SPECIFICATIONS

A sludge drying bed usually consists of a bottom layer of gravel of uniform size over which is laid a bed of clean sand. Open jointed tile underdrains are laid in the gravel layer to provide positive drainage as the liquid passes through the sand and gravel.

(i) Gravel

Graded gravel is placed around the underdrains in layers upto 30 cm with a minimum of 15 cm above the top of the under drains. At least 8 cm of the top layer shall consist of gravel of 3 to 6 mm size.

(ii) Sand

Clean sand of effective size of of 0.5 to 0.75 mm and uniform coefficient not greater than 4.0 is used. The depth of sand may vary from 15 to 30 cm. The finished sand surface shall be level.

(iii) Underdrains

Underdrains are made of vitrified clay pipes or tiles of at least 10 cm dia laid with open joints. Underdrains shall be placed not more than 6 m apart.

(iv) Walls

Walls shall preferably be of masonry and extend atleast 40 cm above and 15 cm below sand surface. Outer walls should be kerbed to prevent washing outside soil on to beds.

(v) Dimensions

Drying beds are commonly 6 to 8 m wide and 30 to 45m long. A length of 30 m away from the inlet should not be exceeded with a single point of wet sludge discharge, when the bed slope is about 0.5%. Multiple discharge points may be used with large sludge beds to reduce the length of wet sludge travel.

(vi) Number of beds

Beds should be at least two in number since multiple beds afford operating flexibility. The size of individual beds is often determined by the discharge pressure of the digester, which is generally 0.3 to 0.6 m.

(vii) Sludge inlet

All sludge pipes and sludge inlets are so arranged to easily drain and have a minimum of 200 mm dia (terminating at least 30 cm above the sand surface. Splash plates should be provided at discharge points to spread the sludge uniformly) over the bed and to prevent erosion of the sand.

(viii) Drainage

Drainage from beds should be returned to the primary settling units if it cannot be satisfactorily disposed of otherwise.

(c) PREPARATION OF BED

Sludge drying beds should be prepared well in advance of the time of application of a fresh batch of sludge. All dewatered sludge which has formed a cake should be removed by rakes and shovels or scrapers, care being taken not to pick up sand with the sludge. After the complete removal of sludge cake, the surface of the bed is cleaned, weeds and vegetation removed, the sand levelled and finally the surface properly raked before adding the sludge. The raking reduces the compaction of the sand on the surface and improves the filterability of the bed.

Only properly digested sludge should be applied to the drying beds. Poorly digested sludge will take a much longer time for dewatering. Sludges containing oils, grease and floating matter clog the sand and interfere with percolation. Samples of sludge from the digester should be examined for the physical and chemical characteristics to ensure that it is ready for withdrawal.

(d) WITHDRAWAL OF SLUDGE

Sludge should be withdrawn from the digester at a sufficiently high rate to clear the pipeline. Rodding and back-flushing of the inlet pipe may sometimes become necessary to make the material flow easily. Valves must be opened fully to start with and later adjusted to maintain regular flow. The flow may be regulated to keep the pipe inlet from being submerged. Naked flames should be prohibited while opening sludge valves and exposed discharge channels.

(e) DEPTH OF SLUDGE

Sludge should be deposited evenly to a depth of not greater than 20 cm. With good drying conditions, the sludge will dewater satisfactorily and become fit for removal in about 2 to 3 weeks producing a volume reduction of 20 to 40%.

(f) REMOVAL OF SLUDGE CAKE

Dried sludge cake can be removed by shovel or forks when the moisture content is less than 70%. When the moisture content reaches 40% the cake becomes lighter and suitable for grinding. Some sand always clings to the bottom of the sludge cake and results in loss of sand thus reducing the depth of the bed. When the depth of the bed is reduced to 10 cm, clean coarse sand which matches the original sand, should be used for replenishment to the original depth of the bed.

(g) HAULING AND STORAGE OF SLUDGE CAKES

Wheel barrows or pick up trucks are used for hauling of sludge cakes. In large plants mechanical loaders and conveyors may be required to handle large quantities of dried sludge. Sludge removed from the bed may be disposed of directly or stored to make it friable, thereby improving its suitability for application to soil.

17.2.2 Mechanical Methods

Vacuum filtration is the most common mechanical method of dewatering, filter presses and centrifugation being the other methods. Chemical conditioning is normally required prior to mechanical methods of dewatering. Mechanical methods may be used to dewater raw or digested sludges preparatory to heat treatment or before burial or land fill. Raw sludge is more amenable to dewatering by vacuum filtration because the coarse solids are rendered fine during digestion. Hence filtration of raw primary or a mixture of primary and secondary sludges permits slightly better yields, lower chemical requirement and lower cake moisture contents than filtration of digested sludges. When the ratio of secondary to primary sludges increases, it becomes more and more difficult to dewater in the filter. The feed solids concentration has a great influence, the optimum being 8 to 10%. Beyond 10%, the sludge becomes too difficult to pump and lower solids concentration would demand unduly large filter surface. In this method, conditioned sludge is spread out in a thin layer on the filtering medium, the water portion being separated due to the vacuum and the moisture content is reduced quickly.

17.2.2.1 *Sludge conditioning*

Prior conditioning of sludge before application of dewatering methods renders it more amenable to dewatering. Chemical conditioning and heat treatment are the two processes normally employed.

(i) CHEMICAL CONDITIONING

Chemical conditoning is the process of adding certain chemicals to enable coalescence of sludge particles facilitating easy extraction of moisture. The chemicals used are ferric and aluminium salts and lime, the more common being ferric chloride with or without lime. Digested sludge because of its high alkalinity exerts a huge chemical demand and therefore the alkalinity has to be reduced to effect a saving on the chemicals. This can be accomplished by elutriation. Polyelectrolytes show promise for sludges with finely dispersed solids. The choice of chemical depends on pH, ash content of sludge, temperature and other factors. Optimum pH values and chemical dosage for different sludges has to be based on standard laboratory tests. The dosage of ferric chloride and alum for elutriated digested sludge are of the order of 1.0 kg/m^3 of sludge. Alum when vigorously mixed with the sludge, reacts with the carbonate salts and releases CO_2 which causes the sludge to separate and the water drains out more easily. Hence for effective results, alum must be mixed quickly and thoroughly. The alum floc, however, is very fragile and its usefulness has to be evaluated vis-a-vis ferric chloride before resorting to its application.

Feeding devices are necessary for applying chemicals. Mixing of chemicals with sludge should be gentle but thorough, taking not more than 20 to 30 seconds. Mixing tanks are generally of the vertical type for the small plants and of the horizontal type for large plants. They are provided with mechanical agitators rotated at 20-60 rpm.

(a) Elutriation

The purpose of elutriation of sludge is to reduce the coagulant demand exerted by the alkalinity of the digested sludge, by dilution with water of lower alkalinity followed by sedimentation and decantation. Some end products of digestion such as ammonium bicarbonate which exert increased demand of chemicals in conditioning are removed in the process. There are three methods of elutriation, viz., single stage, multi-stage and countercurrent washing, the water requirement being dependent upon the method used. For a given alkalinity reduction, single stage elutriation requires 2·5 times as much water as the two stage and 5 times as much water as countercurrent washing. Hence single stage washing is used only in small plants. Countercurrent washing, although higher in initial cost, is adopted in all large plants. Water requirement also depends on alkalinity of dilution water, alkalinity of sludge and desired alkalinity of elutriated sludge. Sludge and water are mixed in a chamber with mechanical mixing arrangement, the detention period being about 20 secs. The sludge is then settled in settling tanks and excess water decanted. A maximum surface loading on settling tank of about 40 m^3/m^2/day and a detention period of about 4 hours are adopted.

Countercurrent elutriation is generally carried out in twin tanks similar to sedimentation tanks, in which sludge and wash water enter at opposite ends. Piping and channels are so arranged that wash water entering the second stage tank comes first in contact with sludge already washed in the first stage tank. The volume of wash water required is roughly 2 to 3 times the volume of sludge elutriated.

The dosage of chemicals, detention period and flow of conditioned sludge to mechanical dewatering units are automatically controlled by float switches so that these variables are adjusted on the basis of performance and the quality of sludge cake coming out.

(ii) HEAT TREATMENT (PORTEUS PROCESS)

In this process, sludge is heated for short periods of time under pressure.

Sludge is preheated in a heat exchanger before it enters a reactor vessel where steam is injected to bring the temperature to 145° to 200°C under pressures of 10 to 15 kg/cm^2. After a 30 min contact time, the sludge is discharged through the heat exchanger to a sludge separation tank. The sludge can be filtered through a vacuum filter to a solid content of 40 to 50% with filter yields of 100 kg/m^2/hr.

17.2.2.2 *Equipment*

(a) VACUUM FILTERS

The vacuum filter consists of a cylindrical drum over which is laid a filtering medium of wool, cloth or felt, synthetic fibre or plastic or stainless steel mesh or coil springs. The drum is suspended horizontally so that one quarter of its diameter is submerged in a tank containing sludge. Valves and piping are arranged to apply a vacuum on the inner side of the filter medium as the drum rotates slowly in the sludge. The vacuum holds the sludge against the drum as it continues to be applied as the drum rotates out of the sludge tank. This pulls water away from the sludge leaving a moist cake mat on the outer surface. The sludge cake on the filter medium is scraped from the drum just before it enters the sludge tank again. Vacuum pumps, moisture traps, filtrate pumps, filtrate receivers , conveyors and pipes and valves are necessary adjuncts to the filter. Operating costs of vacuum filters are usually higher than for sludge drying beds. However, they require less area since dewatering is rapid. The operation is independent of weather conditions and it can be used for dewatering even raw or partially digested sludges requiring drying or incineration. The capacity of the filter varies with the type of sludge being filtered. In calculating the size of filter the desired moisture content of the filter cake is a factor. If wetter cake is acceptable, higher filtration rates and lower coagulant dosage can be used. The filtration rate is expressed in kg of dry solids per square metre of medium per hour. It varies from 10 kg/m^2/hr for activated sludge alone to 50 kg/m^2/hr for primary sludges. A design rate of 15 kg/m^2/hr is a conservative figure that can be used when the quality of the sludge and the type of filter to be used are not known. Filter drums are rotated at a speed of 7 to 40 rph with a vacuum range of 500 to 650 mm of mercury. The filter run does not exceed 30 hrs per week in small plants to allow time for conditioning, clean up and delays. At larger plants, it may work for 20 hrs a day. The moisture of the filtered cake varies normally from 80% in case of raw activated sludge to 70% for digested primary sludges. Filters should be operated to produce a cake of 60 to 70% moisture if it is to be heat dried or incinerated. At the end of each filter run, the filter fabric is cleaned to remove sticking sludge. A high pressure stream of water is used to clean the filter cloth. The filters are usually located in a separate room or building with adequate light and ventilation.

(b) FILTER PRESSES

Filter presses consist of a series of rectangular movable plates recessed on both sides and fitted with filter cloth. Sludge is pumped into the space between the plates and a pressure of 4 to 12 kg/cm^2 is maintained for a period of 1 to 3 hrs forcing the liquid through the filter cloth and plate outlet ports. After filtration, the cakes, 25 to 40 mm thick with a moisture content of 55 to 70% are allowed to drop from the press by separating the plates. The method permits the direct and practically complete removal of suspended solids from the crude sludge.

(c) CENTRIFUGATION

The process of high speed centrifuging has been found useful to reduce the moisture in sludge to around 60%. Usually the liquor from the centrifuge has a high solids content than filtrate from sand drying beds. Return of this liquor to the treatment plant may result in a large recirculated load of these fine solids to the primary settling and sludge system and also in reduced effluent quality. (Refer 17. 1.3. also).

17.2.3 **Heat Drying**

The purpose of heat drying is to reduce further the moisture content and volume of dewatered sludge, so that it can be used after drying without causing offensive odours or risk to public health. Several methods such as sludge drying under controlled heat, flash drying, rotary kiln, multiple-hearth furnaces, etc., have been used in combination with incineration devices. Drying is brought about by directing a stream of heated air or other gases at about 350° C. The hot gases, dust and ash released during combustion are to be removed by suitable control mechanisms to minimise air pollution. The dried sludge removed from the kilns is granular and clinker-like which may be pulverised before use as soil conditioner.

17.2.4 Incineration

The purpose of incineration is to destroy the organic material, the residual ash being generally used as landfill. During the process all the gases released from the sludge are burnt off and all the organisms are destroyed. Dewatered or digested sludge is subjected to temperatures between 650°C to 750° C. Cyclone or multiple hearth and flash type furnaces are used with proper heating arrangements with temperature control and drying mechanisms. Dust, flyash and soot are collected for use as landfill.

It has the advantages of economy, freedom from odours, and a great reduction in volume and weight of materials to be disposed of finally. But the process requires high capital and recurring costs, installation of machinery and skilled operation. Controlled drying and partial incineration have also been employed for dewatering of sludges before being put on conventional drying beds.

17.3 SLUDGE DISPOSAL

Sludge is usually disposed of on land as manure to soil, or as a soil conditioner, or barged into sea. Burial is generally resorted to for small quantities of putrescible sludge. The most common method is to utilize it as a fertiliser. Ash from incinerated sludge is used as a landfill. In some cases wet sludge, raw or digested, as well as supernatant from digester can be lagooned as a temporary measure but such practice may create problems like odour nuisance, ground water pollution and other public health hazards. Wet or digested sludge can be used as sanitary landfil or for mechanised composting with city refuse.

17.3.1 Sludge as Fertilizer

The use of raw sludge as a fertilizer directly on land for raising crops as a means of disposal is not desirable since it is fraught with health hazards. Application of sewage sludges to soils should take into consideration the following guiding principles :

(i) Sludge from open air drying beds should not be used on soils where it is likely to come into direct contact with the vegetables and fruits grown.

(ii) Sludge from drying beds should be ploughed into the soil before raising crops. Top dressing of soil with sludge should be prohibited.

(iii) Dried sludge may be used for lawns and for growing deep rooted cash crops and fodder grasses where direct contact of edible part is minimum.

(iv) Heat dried sludge is the safest from public health point of view. Though deficient in humus, it is convenient for handling and distribution. It should be used along with farmyard manure.

(v) Liquid sludge either raw or digested is unsafe to use. It is unsatisfactory as fertilizer or soil conditioner. If used, it must be thoroughly incorporated into the soil and the land should be given rest, so that biological transformation of organic material takes place. It should be used in such a way as to avoid all possible direct human contact.

In general, digested sludges are of moderate but definite value as a source of slowly available nitrogen and some phosphate. They are comparable to farmyard manure except for deficiency in potash. They also contain many essential elements to plant life and minor nutrients, in the form of trace metals. The sludge humus also increases the water holding capacity of soil and reduce soil erosion making an excellent soil conditioner specially in arid regions by making available needed humus content which results in greater fertility.

17.3.2 Sludge Lagooning

Use of sludge lagoons for storage, digestion, dewatering and final disposal of dried sludge may be adopted in isolated locations where the soil is fairly porous and when there is no chance of ground water contamination. Drainage water should not be allowed to enter the lagoon. The depth of lagoon and its area should be about twice that required for sand drying under comparable conditions. Depth may range from 0.5 to 1·5 m. Lagoons have been used for regular drying of sludge on a fill and draw basis or allowed to fill dry and then levelled out and used as lawns. Lagoons have also been employed as emergency storage when digesters have to be emptied for repairs. As they are less expensive to build and operate, they have been resorted to, particularly for digested sludge in areas where large open land suitably located is available. Use of lagoons is not generally desirable, as they present an ugly sight and cause odour and mosquito breeding.

17.3.3 Disposal in Water or Sea

This is not a common method of disposal because it is contingent on the availability of a large body of water adequate to permit dilution. At some sea coast sites, the sludge either raw or digested may be barged to sea far enough to make available the required dilution and dispersion. The method requires careful consideration of all factors for proper design and siting of outfall to prevent any coastal pollution or interference with navigation.

CHAPTER 18

SLUDGE PUMPING

In sewage treatment plants, sludge produced in the different units has to be necessarily moved from point to point by pumping watery or thick sludge or occasionally scum. Pumping may be intermittent as in the case of raw sludge from settling tank to the digester or continuous as in the return of the activated sludge to the aeration tank. Efficiency of sludge pumps is therefore important in the proper operation of a sewage treatment plant. The efficiency of a sludge pump is considered subordinate to dependable, satisfactory and trouble free operation. Sludge pumps have to be resistant to abrasion as sludges too often contain sand and grit. Design of pumps will have to provide for sufficient clearance, as close clearances though necessary for higher efficiencies, lead to frequent stoppages and excessive wear.

18.1 SLUDGE PUMPS

Generally, sludge pumps are of the reciprocating (plunger) or centrifugal type. Diaphragm pumps, airlift pumps and ejectors are also used. Reciprocating pumps of the simplex, duplex or triplex models with capacities of 150 to 250 lpm per plunger are available. They are suitable for suction lifts upto 3 m and are self priming. The pump speeds should be between 40 and 50 rpm and designed for a minimum head of 25m, since accumulations of grease in sludge piping would cause progressive increase in head with use.

Centrifugal pumps will have to be equipped with variable speed drives, to suit variable capacities since throttling the discharge to reduce the capacity with a constant drive results in frequent stoppages. The design should also consider the need for pumping sludge solids without clogging and yet avoid pumping large quantities of overlying sewage.

There are various modifications of centrifugal pumps presently in the market. The centrifugal pumps with nonclogging impellers are confined to bigger sizes. The scrupeller pump, a modification of screw-feed impeller pump is applicable specially to the pumping of sludge. It is less prone to clogging than the ordinary centrifugal pumps and eliminates some of the objections inherent in reciprocating pumps.

18.2 APPLICATION OF PUMPS

Types of sludges required to be pumped are primary, secondary, return, elutriated, thickened and concentrated. In addition scum also needs pumping.

18.2.1 Pumps for Primary Sludge

Plunger and centrifugal pumps of the screw type have been widely used for transporting primary sludge, plunger pumps being preferred for this duty, for the following reasons:

(i) pulsating action tends to concentrate the sludge in the hoppers;

(ii) suction lifts can be accommodated ;

(iii) low pumping rates are possible with large port openings;

(iv) positive delivery is generally assured;

(v) constant but adjustable capacity, regardless of large variations in pumping head is obtainable;

(vi) large discharge heads are possible;

(vii) clogging is easily corrected; and

(viii) heavy concentration of solids can be pumped readily.

18.2.2 Pumps for Secondary Sludge

Nonclogging centrifugal and plunger pumps as well as airlifts and ejectors find use for secondary sludge, but the centrifugal pump is preferred because of the advantages like greater efficiency, capacity for handling light solids concentration, uniform and smooth delivery of solids, mixing of the mass which is being pumped, quiet and cleaner operation than the plunger pump and lower maintenance costs for continuous operation.

18.2.3 Pumps for Recirculation and Transfer of Sludge

Selection of the type is largely dependant on location of the pump. Where priming is positive and no suction lift is involved, large, centrifugal pump with variable-speed drive is a good choice. The nature of solids to be handled by these pumps being similar or less concentrated than the primary sludge and since suction lifts are low, centrifugal pumps give successful results. Where a suction lift is involved, the plunger pump may be selected. Often it may be possible to combine this duty with primary or secondary sludge pumping by proper arrangement of units. Plunger pumps have an advantage over centrifugal pumps when it becomes necessary to dewater a digester completely.

18.2.4 Pumps for Elutriation of Sludge

Pumps that are useful for primary sludges find application for elutriation purposes also. In small plants, plunger pumps are preferred as they may often be combined with the primary sludge pumps in a common location. They are generally equipped with counters to measure the quantity of sludge pumped. In larger plants, the centrifugal pump is chosen since it is capable of smoother, quieter and cleaner operation, as well as for delivery of larger volumes of elutriated sludge. Suitable sludge meters used in conjunction with centrifugal pumps employed in elutriation permit the measuring of sludge quantities.

Adjustment of washing ratios and variation of the rate of delivery to the vacuum-filters corresponding to various dewatering rates can be accomplished by variable-speed drives.

18.2.5 **Pumps for Thickened and Concentrated Sludge**

Pumps for this duty will have to accommodate higher friction losses in discharge lines. Plunger pumps serve this purpose better.

18.2.6 **Scum Pumping**

Scum is usually pumped by screw-feed pumps, plunger pumps and pneumatic ejectors.

18.3 STANDBY UNITS REQUIRED

Several factors like the particular function involved, size of plant and arrangement of units determine the need for standby units.

Since primary and secondary sludge pumping are important functions, standby pumps are generally provided, in actual numbers or by such arrangements that dual duty is possible. Scum is usually mixed with the primary sludge and pumped.

18.4 SLUDGE CONVEYING PIPING

After determining the characteristics of sludge and the proper types of pumps for pumping the same, the next important thing is to design the pipelines to and from the pumps. There are many factors which need consideration in the design of pipes, such as the rate and velocity of flow of sludge and arrangements of piping.

Sewage sludge flows like a thin plastic material and hence the formulae for the flow of water in pipes are not applicable. The velocity of flow should be in the critical range, above the upper limit of the laminar flow and below the lower limit of turbulent flow, in order to avoid clogging and deposition of grease so that the application of hydraulic formulae for flow of water become permissible. In general, velocities between 1·5 and 2·5 mps are satisfactory. The friction head loss in the sludge pipe can be estimated by the application of Hazen Williams formula adopting 'C' values of 40 to 50 depending on the material to be used for conveying sludge, and lower values being adopted for high solids content of the sludge.

Pipes less than 200 mm dia should not be used for gravity withdrawal or for suction lines to pumps. In order to take care of thin sludge to flow by gravity for short distances within the treatment plant, a 3% or greater slope should be adopted.

Suction and discharge pipes should be arranged in such a way that their lengths are short, straight and with minimum bends. Large radius elbows and sweep tees are usually adopted for change in direction. High points should be avoided, as far as possible, to prevent gas pockets. Suitable recess and sleeves are usually provided for all pipes passing through masonry. Double flanged pipes are usually adopted for sludge line providing valves at selected locations to clean the lines.

18.5 PUMP APPURTENANCES

In order to assist the operator to perform his job efficiently and easily, several appurtenances such as air chamber, revolution counter, gland seals, valves, gauges, sampling devices, washouts and drains, time clocks and measuring devices are necessary.

18.5.1 **Air Chamber**

Air chamber of adequate size is necessary for all plunger type sludge pumps on the discharge side of the pump. It may also be provided on the suction side of the pump particularly where positive suction head exists. Such chambers absorb the shock of plunger pump pulsations.

18.5.2 **Revolution Counter**

Plunger sludge pumps shall be equipped with revolution counters or integrating recorders to help the operator to determine the quantity of sludge pumped. In duplicate pump installations, these aid in equalising the service and wear of each pump.

18.5.3 **Gland Seals**

Stuffing boxes are normally provided for external sealing in centrifugal pumps to ensure against the leakage of air around the shaft. Pump manufacturers provide grease seals or clear water sealing. Water seals not only provide air tightness but also help to wash the grit and dirt away from the packing. Potable water is never connected with sludge pumps directly.

18.5.4 **Valves**

Swing check or ball check valves are generally used for sludge pump duty as they provide full opening without obstruction. Water hammer could be avoided by weighted or spring loaded checks. Pumps with dual check valves are sometimes used to give more consistent operation and to aid in the use of pumps as metering devices. In larger size plants where the valves are not in frequent use, gate valves are generally used in sludge lines. Single disc valves are also used. For interior installation, the rising stem valves offer the advantage of visual determination of the valve positions. For exterior underground locations, gate valves are generally used. Underground sludge valves should be avoided as far as possible.

18.5.5 **Gauges**

Pressure gauges shall be installed on both delivery and suction sides. Gauges provided with a cast iron bowl and an oil-resistant rubber diaphragm which will keep sludge out of the finer working parts of the gauges should be used.

18.5.6 **Sampling Devices**

Sampling cocks of 35 mm dia are usually provided in all sludge pumps either within themselves or in the immediate piping adjacent to the pump. They are provided with plug valves which are simple and easy to operate for taking samples.

18.5.7 Washouts and Drains

Washout arrangements are provided for sludge pumps to effect easy and rapid clearance. Drains on the pump body should be of ample size to assure liquid drainage and release of pressure and should be connected to adjacent floor drain for cleaning.

18.5.8 Time Clocks

Time clocks, wired across the magnetic starters or motor leads of sludge pumps can be of valuable help to the operators. They help to keep an accurate record of hours of run of the pumps for attending to lubrication, equalisation of wear and tear as part of preventive maintenance.

18.5.9 Measuring Devices

While time clocks and counters are adequate for small plants, additional venturimeters and flow tubes with flushing provisions are used in larger plants for recording and measuring the quantities of sludge handled. Magnetic meters are more suitable for sludge metering.

18.6 PUMP DRIVE EQUIPMENT

The drive equipment used to operate pumps consists of gasoline or gas engines and electric motors. Gasoline engines are used mainly for standby service in case of electric power failure. They are either directly connected to pumps or operate electric generators which in turn furnish the power for driving pumps. Gasoline engines are costly to run and need strict supervision. Gas engines using sludge gas as fuel are used to drive centrifugal pumps.

Electric motors are nowadays exclusively used to operate sewage and sludge pumps. The type generally found are synchronous and induction motors. The synchronous motors which operate at constant speed, are mainly used in very large plants while induction motors find common application in sewage works. Induction motors can be either wound-rotor or squirrel cage type. The wound rotor motor is costlier and at the same time requires sligtly more maintenance. Explosive proof motors are currently used for transmission of power to pumps, i.e. to couple the prime movers to the pumps through chain drives, belt drives (conventional or V-type belts), varispeed drives, directly coupled drives and gear drives. Magnetic starters remotely located to avoid moisture and dampness are used for control of sludge pumps since start-stop buttons have the disadvantage of manual operation.

CHAPTER 19

CHLORINATION OF SEWAGE

Chlorination of sewage can be used to prevent septicity in sewers, overcoming difficulties during treatment and disinfection of plant effluents prior to disposal. Regular chlorination of effluents has not been practised so far in the country except for sewage from infectious diseases hospitals, T.B. sanatoria, etc.

19.1 PURPOSE OF CHLORINATION

Chlorination of sewage is used for

(i) control of odours within the collection system and plant;

(ii) aiding grease removal;

(iii) inhibiting formation of hydrogen sulphide, a contributory factor to corrosion;

(iv) reduction of BOD of raw and treated sewage and digester supernatant;

(v) prevention of sludge bulking in activated sludge plants;

(vi) keeping activated sludge aerobic during thickening;

(vii) checking of foaming in anaerobic digesters;

(viii) disinfection of raw sewage and plant effluents;

(ix) control of filter-fly nuisance; and

(x) overcoming ponding due to clogging of trickling filters.

The oxidising power of chlorine is responsible for these effects which are brought about by reaction of chlorine with unstable organic matter and odorous compounds and inactivation of micro and macro life present in sewage. Factors influencing the effects of chlorination have been discussed in the companion volume—Manual on Water Supply and Treatment. The dose of chlorine for giving a definite residual is dependent on the concentration of the sewage as well as its septicity. Moreover, any residual effect is only temporary.

19.2 DOSAGE

The dose of chlorine varies not only according to the purpose for which it is used but also varies widely from plant to plant and therefore, it is difficult to prescribe definite limits for the dosages In most cases, it ranges between 2 to 25 mg/l. However, it is advisable to determine the actual dosages by experimentation on the basis of residual chlorine or number of coliforms in the treated effluent.

Application of chlorine can bring about 15 to 25% BOD reduction of raw sewage, approximately 0·5 mg of chlorine being required for each mg reduction of BOD. For oxidation of digester supernatant, the dose may range between 20 and 140 mg. The control of filter flies requires sufficient dose to produce a residual chlorine of 0·1 to 0·5 mg/l at filter nozzles. For control of ponding in trickling filters, a residual of 1 to 10 mg/l has to be maintained at filter nozzles. Chlorine dose for disinfection of raw sewage and primary effluents ranges between 10 and 25 mg/l and that of secondary effluents between 2 and 15 mg/l. Sewage solids can, however, shelter organisms, particularly virus and the safety obtained is not complete.

19.3 CHLORINE CONTACT CHAMBERS

Chlorine contact chambers with contact times from 15 to 30 min at peak flow should be provided when effluents are to be disinfected. In order to achieve the requisite contact time, plug flow type of tanks having round-the-end baffles are recommended. Provision should be made for rapid mixing of the added chlorine through hydraulic turbulence for at least 30 seconds. The other important considerations in the design are maintenance of self cleansing velocity and provision for bypassing portions of the chamber for cleaning purposes. Where the length of the outfall is sufficient to ensure the required contact time, the contact chamber may be eliminated.

19.4 CHLORINE STORAGE, HANDLING AND FEEDING

Chlorine may be applied either in the form of its compounds or as chlorine gas. The equipment for feeding and other considerations are the same as those required in water treatment practice as discussed in the Manual on Water Supply and Treatment.

CHAPTER 20

EFFLUENT DISPOSAL AND UTILISATION

The effluents from sewage treatment units may be discharged into lakes, streams, rivers, estuaries, oceans or on land. The nature and degree of treatment given to the sewage is dependent upon the requirements imposed by the regulatory authorities. It is the large water portion along with a small residual organics after treatment that has to be disposed of while the major portion of the organics is handled within the treatment plant itself. The water content of the sewage effluent along with the fertility value of the organics serves to make it useful for irrigation and pisciculture; the effluent is also put to use for industrial purposes where quality is not very important or for artificial recharging of aquifers in areas of rapid depletion of underground water sources.

20.1 DISPOSAL ON LAND

The potential manurial and irrigational values of the sewage effluent permit its utilization in sewage farming. However, to safeguard the public from health hazards involved, certain restrictions have necessarily to be imposed as detailed in chapter 21.

20.2 DISPOSAL BY DILUTION

Where utilisation for sewage farming is not found suitable, the treated effluent may be disposed of into a stream course or into sea or a stagnant body of water. The quality and quantity and use of the water into which the effluent is discharged decides the degree of treatment required for the sewage.

20.2.1 Basic Information

The basic information to be collected for planning effluent outfall works should consist of

(i) studies of the characteristics of the sewage to be treated particularly with respect to toxic metals and other materials likely to be admitted into the sewers and the degree of treatment necessitated;

(ii) hydrographic surveys or examination of available hydrographic records including :

(a) run off records, and characteristics of flow both at and below the point of disposal during the worst periods in the case of streams;

(b) observations on currents and effects of winds, seiches and temperature stratification upon the dispersion of the sewage, in the case of lakes; and

(c) tides, the effects of winds, salinity and temperature stratification upon the movement of the sewage, in the case of tidal estuaries;

(iii) studies of possible locations for and forms of sewer outfall in its relation to hydrographic conditions particularly in the case of lakes and ocean outfall; and

(iv) studies of the various uses of the water receiving the sewage effluent, giving due consideration to the protection of water supplies and the relative value and economy of sewage treatment and water purification, the safeguarding of the bathing beaches and other recreational facilities, conservation and protection of useful aquatic life and its commercial value in relation to sewage treatment, the avoidance of unsightly or offensive conditions created by the sewage solids on or in the waters or along the shores, the prevention of sludge bank formation and of the resulting encroachment on waterways.

20.2.2 Standards

It is necessary to adhere to the standards laid down by the Indian standards Institution as embodied in IS:4764—1973, while disposing of sewage or sewage effluent in a body of water. However where the dilution is low, these standards may have to be made stringent to ensure that the receiving water conforms to the Standards IS:2296—1974. Where Statutory Water Pollution Prevention and Control Boards exist, this will be governed by the consent conditions issued by the Boards.

20.3 RECLAMATION OF WASTEWATERS

Complete reclamation of sewage effluent is not generally adopted, this being only supplementary to other methods of disposal. Reclamation is restricted to meet the needs depending upon the availability and cost of fresh water, transportation and treatment costs and the water quality standards and the impending uses like watering of lawns and grass lands; cooling, boiler-feed and process water; forming artificial lakes; wetting of refuse for compaction and composting; and raising agricultural crops. Some of these uses may need tertiary treatment like sand filtration, microstraining or fish ponds.

20.4 PISCICULTURE

If local conditions are suitable, partially purified sewage effluent may be used for fish culture without further dilution. Raw sewage cannot directly be used for fish culture without dilution as it does not contain sufficient dissolved oxygen for the survival and growth of fish. The waste stabilization pond effluent and the percoloted effluent from sewage farms have also been successfully used in pisciculture.

20.5 ARTIFICIAL RECHARGE OF AQUIFERS

Artificial recharge of groundwater aquifers is one of the most common methods for combining effluent dispoal with water reuse. Replenishment of groundwater sources has been done on a practical scale. Treated effluent has been used to arrest salt water intrusion which may take place due to the lowering of ground water table bv the rapid pumping to meet large water demands. Another possible use of recharge is the application of effluent for flooding thereby increasing the yield of oil bearing strata. Recharging may be done through injection or diffusion wells. Treatment of effluent must include chlorination to prevent algae and slime formation which clog the pores of the soil.

When points of effluent discharge are well arranged and effluent quantities are limited, there is no serious threat to ground water quality. However, in many unsewered residential areas, particularly suburban developments, domestic wastes are disposed of through closley spaced individual sewage disposal units somtimes interspersed with water wells. Adequate precautions should be taken to ensure that the water sources are not contaminated by the improper location of cesspools. septic tanks and subsurface dispersion systems, Synthetic detergents, unlike most other chemicals, are not usually removed by passage though the soil mantle nor do they readily disintegrate. When they spread through the ground water reservoir they create unpleasant taste and frothing.

In the present day when conservation, reclamation and reuse of water are receiving increasing emphasis. sewage effluent constitutes a valuable source for recharging ground water. When used as an irrigant, much lower spreading rates have to be adopted with sewage effluents than fresh water because of the higher concentrations of suspended matter and bacteria.

CHAPTER 21

SEWAGE FARMING

The nutrients in sewage like nitrogen, phosphorus and potassium along with the micronutrients as well as organic matter present in it could be advantageously employed for sewage farming to add to the fertility of the soil, along with the irrigation potential of the water content. As very few cities in the country are sewered and that too partially, the nightsoil and sullage from the unsewered areas is used for farming which is fraught with public health dangers. Sometimes in sewered areas where no treatment facilities are available, sewage farming with raw sewage is practised which is again a grave public health hazard. Even application of treated effluent to land has to be carried out with certain precautions as it is not completely free from this risk. A good sewage farm should be run on scientific lines with efficient supervision with the primary objective of disposal of sewage combined with its utilisation to the possible extent in a sanitary manner without polluting the soil, open water courses or artesian waters or contaminating crops raised on the sewage farm; or impairing the productivity of the soil. It should also provide for hygienic safety of the staff to protect them against the infection by pathogenic organisms and helminths.

Though sewage after primary treatment can be applied to the farms, the temptation of providing only primary treatment and eliminating secondary treatment merely on cost considerations should be resisted. Under no conditions, application of raw sewage on sewage farms should be permitted.

21.1 WATER QUALITY CONSIDERATIONS FOR IRRIGATION WATERS

The quality of water for irrigation is determined by the effects of its constituents both on the crop and the soil. The deleterious effects of the constituents of the irrigant on plant growth can result from (i) direct osmotic effects of salts in preventing water uptake by plants, (ii) direct chemical effects upon the metabolic reactions in the plants (toxic effect), and (iii) any indirect effect through changes in soil structure, permeability and aeration.

The suitability of an irrigant is judged on the basis of soil properties, quality of irrigation water and the salt tolerance behaviour of the crop grown in a particular climate. The water quality ratings along with the specific soil conditions recommended for the country are shown in Table 21.1.

These limits apply to the situations where the ground water table at no time of the year is within 1.5 m from the surface. The values have to be reduced by half if the water table comes up to the root zone.

Table 21.1

Water quality Ratings

Nature of soil	Crop to be grown	Permissible limit of Electrical Conductivity of water for safe irrigation (micromhos/cm)
Deep black soils and alluvial soils having a clay content more than 30% : Soils fairly to moderately well drained.	Semi tolerant	1500
	Tolerant	2000
Heavy textured soils having a clay content of 20—30%. Soils well drained internally and having a good surface drainage system.	Semi-tolerant	2000
	Tolerant	4000
Medium textured soils having a clay content of 10—20% : Soils very well drained internally and having a good surface drainage system.	Semi-tolerant	4000
	Tolerant	6000
Light textured soils having a clay content of less than 10% : Soils having excellent internal and surface drainage.	Semi-tolerant	6000
	Tolerant	8000

If the soils have impeded internal drainage either on account of presence of hard stratum unusually high amounts of clay or other morphologic reasons, advisedly, the limit of water quality should again be reduced to half. If supplemetary canal irrigation is available, waters of higher electrical conductivity could be used profitably, in lean periods.

21.1.1 Osmotic Effects

When water is applied for cultivation on land, some of it may run off as surface flow or be lost by direct surface evaporation, while the remainder infiltrates into the soil. Of the infiltrated water, a part is used consumptively, a part is held by the soil, for subsequent evapotranspiration and the remaining surplus percolates or moves laterally through the soil. The water retained in the soil is known as the 'soil solution' and tends to become more concentrated with dissolved consituents as plants take relatively purer water. An excessive concentration of salts in the soil solution prevents water uptake by plants. Table 21.1 shows permissible levels of electrical conductivity (EC) and hence total salts of water for safe irrigation in the four types of soils. It may be pointed out that good drainage of the soil may be a more important factor for crop growth than the EC of the irrigant as leaching of soils results in maintaining a low level of salt in soil solution in the root zone.

21.1.2 Toxic Effects

Individual ions in irrigation water may have toxic effects on plant growth. Table 21.2 lists some of the known toxic elements and their permissible concentration in irrigation waters when continuously applied on all soils and also when used on fine texture soils for short terms. Many of these are also essential elements for plant growth.

Table 21.2

Maximum Permissile concentration of Trace elements in Irrigation Waters

Element		Maximum permissible concentration mg/l for water used continuously on all soils	For short term use on fine texture soils
Aluminium	Al	1·0	20·0
Arsenic	As	1·0	10·0
Beryllium	Be	0·50	1·0
Boron	B	0·75	2·0
Cadmium	Cd	0·005	0·05
Chromium	Cr	5·0	20·0
Cobalt	Co	0·2	10·0
Copper	Cu	0·2	5·0
Fluorine	F	..	10·0
Lead	Pb	5·0	20·0
Lithium	Li	5·0	5·0
Manganese	Mn	2·0	20·0
Molybdenum	Mo	0·005	0·05
Nickel	Ni	0·5	2·0
Selenium	Se	0·05	2·0
Vanadium	V	10·0	10·0
Zinc	Zn	5·0	10·0

21.1.3 Impairment of Soil Quality

21.1.3.1 *Sodium hazard*

In most normal soils, calcium and magnesium are the principal cations held by the soil in replaceable or exchangeable form. Sodium tends to replace calcium and magnesium when continuously applied through irrigation waters. An increase of exchangeable sodium in the soil causes deflocculation of soil particles and promotes compaction, thereby impairing soil porosity and the water and air relations of plants. The sodium hazard of irrigation water is commonly expressed either in terms of percent soluble sodium (PSS) or sodium adsorption ratio (SAR) where

$$PSS = \frac{100 \times Na^{+}}{(Na^{+} + Ca^{++} + Mg^{++} + K^{+})}$$

$$\text{or } \frac{100 \times Na^{+}}{(\text{Total cations})} \quad \ldots\ldots(21\text{—}1)$$

$$\text{and } SAR = \frac{Na^{+}}{\left\{\frac{Ca^{++} + Mg^{++}}{2}\right\}^{\frac{1}{2}}} \quad \ldots\ldots(21\text{-}2)$$

and the cations are expressed as meq/l. Generally the sodium hazard of soil increases with the increase of PSS or SAR of irrigation water and exchangeable sodium percentage of the soil. The maximum permissible value of PSS in irrigation water is 60. Where waters with higher PSS values are used, gypsum should be added to the soil occasionally for soil amendment.

Hazardous effect of sodium is also increased if the water contains bicarbonate and carbonate ions in excess of the calcium and magnesium as there is a tendency for calcium and magnesium to precipitate as carbonates from the soil solution and thereby increasing the relative proportion of exchangeable sodium. Values of residual sodium bicarbonate less than 1.25 meq/l are considered safe and above 2.5 meq/l as unsuitable.

The effect of potassium on soil is similar to that of sodium but since the concentration of potassium is generally quite small in irrigation waters, it is often omitted from consideration.

21.1.3.2 *Organic solids*

While stable organic matter improves porosity of soil, thereby facilitating aeration, an excessive application of unstable organic matter would lead to oxygen depletion in the soil. Deposition of sediments especially when they consist primarily of clays or collodial material may cause crust formations which impede emergence of seedlings. In addition, these crusts reduce infiltration with the consequent reduction of irrigation efficiency and less leaching of saline soils.

21.1.4 Other Considerations

Soils are usually well buffered systems. Their pH is not significantly affected by application of irrigation water. However, extreme values below 5.5 and above 9.0 will cause soil deterioration. Development of low pH values in soils promotes dissolution of elements such as iron, aluminium or manganese in concentrations large enough to be toxic to plant growth. Similarly, waters having high pH values may contain high concentrations of sodium, carbonates and bicarbonates, the effect of which have been discussed earlier.

Chlorides and sulphates are toxic to most crops in high concentrations. Ordinarily, the detrimental effects of salinity on crop growth become perceptible first.

Excessively high or low temperatures in irrigation waters may affect crop growth and yields. A desirable range of water temperature is from 12 to 20°C.

21.2 PUBLIC HEALTH ASPECTS

Presence of pathogenic microorganisms in treated sewage and raw sewage used as irrigants is of major importance. Raw sewage always contains a large variety of intestinal pathogens like helminthic eggs, bacteria and viruses. The public health aspects of sewage farming should be considered from the view points of exposure of farm workers to sewage and that of the consumers of the farm products. It has been established through surveys carried out in different sewage farms of the country that labourers working in sewage farms suffer from a number of ailments directly attributed to handling of sewage.

21.3 DESIGN AND MANAGEMENT OF SEWAGE FARMS

Optimum utilisation of sewage in agriculture means the complete and judicious use of its three main components viz., water, plant nutrients and organic matter on the farms in such a way that (a) the pathogenic infection is neither spread among the farm workers nor the consumers of sewage farm products, (b) the ground water is not contaminated, (c) there is maximum outturn per unit volume of sewage (d) there is no deterioration of the soil properties and (e) none of the three components are wasted.

21.3.1 Management of Water in Sewage Farming

The principle to be borne in mind in irrigation management is to irrigate only when it is required and only to the extent it is required by the crop. The water requirement depends on the soil type, the crop and the climate. The water requirement (cm) of main soil types to be wetted to a depth of 30 cms required by most of the crops is given in Table 21.3

Table 21.3

Water requirements to wet different soils to a Depth of 30 cm

Sand	1·25
Sandy loam	2·50
Loam	5·00
Clay loam	6·25
Clay	7·50

Water requirement of crops vary with the duration of their growing season and the amount of growth in unit time. Details for some of the Indian crops which can be grown on sewage farm are given in Table 21.4.

Table 21.4

Water requirements of Crops

Crops	Growing period (days)	Total water requirements (cm)	Optimum pH range
1. Soyabean	110—120	37·50	6·0—8·5
2. Mustard	120—140	37·50—55·00	6·0—9·5
3. Sunflower (Kharif)	100—110	37·50	6·0—8·5
4. Sunflower (Rabi)	110—120	87·50	6·0—8·5
5. Barley	88	35·25	6·5—8·0
6. Cotton	202	105·50	5·0—6·0
7. Jowar	124	64·25	5·5—7·5
8. Maize	100	44·50	5·5—7·5
9. Linseed	88	31·75	5·0—6·5
10. Rice	98	104·25	5·0—6·0
11. Milling varieties of Sugarcane.	365	237·50	6·0—8·0
12. Wheat	88	37.0	5.5—7.5

The irrigation requirement of any crop is not uniform throughout its growing period. It varies with the stage of growth. For example grain crops require maximum irrigation during the time of ear-head and grain formation. Sugarcane requires more frequent irrigation from about the sixth or the seventh month onwards. In case of fruit trees the irrigation has to be stopped during their resting period. If the irrigation is not given at critical growth stages of the crop, it results in lower yields.

Water requirement of crop at different stages of growth can be determined either directly (gravimetrically) or indirectly by use of Tensioneters or Irrometers or gypsum blocks. Normally, when there is about 50 % depletion of available moisture in the soil, irrigation is recommended. The crop plants themselves show signs of moisture stress. One has to be always on the look out for such first symptoms to determine the need for irrigation. Some plants like sunflower also serve as good indicators of stress symptoms. Sunken screen pan evoporimeter could also be used for estimating use of water by crop plant and scheduling irrigation.

The extent of irrigation depends on the depth of irrigation to be given and volume of water required to wet the soil to the required depth. If tensiometers or gypsum blocks are embedded at the required depths, they would indicate the stage when the soil at that depth is saturated. Nearly about 70 to 80% roots of most crops are found in the first 30cm of the soil. Some may go deeper on the next 30 cm. Normally, in irrigating medium type of soil it is wetted to about 30cm depth or a little more.

If the figures for water requirements for crop as mentioned in Table 21-4 are to be satisfied much higher hydraulic loadings will have to be applied since a portion of sewage after its passage through the soil is carried away by the underdrainage system. The applicable hydraulic loadings of settled sewage are therefore dependent upon the type of soil and the recommended rates are given in Table 21-5.

Table 21. 5

Recommended Hydraulic Loadings

Type of soil	*Hydraulic loading (m^3/hectare/day)*
(i) Sandy	200—250
(ii) Sandyloam	150—200
(iii) Loam	100—150
(iv) Clay loam	50—100
(v) Clayey	30—50

Sewage conforming to the norms specified in 21.3.3 should be applied to the soil by strip, basin or furrow irrigation. Wild flooding or sprinkler irrigation should not be adopted.

The distribution channels should be properly graded to avoid ponding and silting. It is advisable that the main distributary is lined.

21.3.2 Management of Soil

It is necessary that the soil is given rest for about 3 to 4 months every alternate or third year preferably in summer months. This can be achieved if the farm is designed on the basis of water requirement in the winter season. After the harvest of the winter crop, say in March, the soil may be opened up by deep ploughing and cultivated sufficiently to make it as porous and permeable as possible before the next crop is raised.

Maintenance of soil oxygen level is very important as it is required for root respiration and a number of biological processes in the soil. Refilling of oxygen in the pores in the surface layers of soil depends upon the reestablishment of contact of the soil atmosphere with the environment. This process can be accelerated by suitable cultural practices and by providing sufficient irrigation intervals. It is, therefore, desirable that an intercultural operation is followed as soon as the soil condition allows working after every irrigation. It should always be seen that the soils of sewage farm should have a surplus of oxygen than that normally required in the ordinary farm because the soil oxygen has to perform an additional job of satisfying the BOD of sewage. The intercultural operation following every one or two irrigations is all the more necessary in the case of clayey soil. In the areas where rainfall is low, it is desirable to flood the soils with primary water at least once a year to leach down the salts accumulated in the soil. If the soil salinity and alkalinity pose a serious problem, amendment of soil with the required quantity of gypsum should be carried out. Subsoil drainage is very important. Poor drainage should be improved by installing underground tile drains.

The sewage distribution system must, as a rule, be a pipe network. Only under most favourable sanitary conditions or on a temporary basis can an open permanent sewage network be allowed with the permission of local public health engineering authority provided the channels are regularly and thoroughly cleaned.

Crop rotation fields on agricultural sewage farms must be laid out in accordance with the natural slope of the terrain to eliminate the irregularities of distribution.

On agricultural sewage farms no sewage should be allowed to be held beyond the farm boundaries. With this in view, protection banks are arranged along the lowest lying boundaries of each crop rotation field. In addition, buffer (reserve) filtration fields (plots) should be provided to handle sewage during unfavourable seasons or rest periods, harvest time and in other cases when sewage cannot be used for irrigation purposes.

Where the ground water table is less than 2m below the surface, sewage farming should be avoided.

21.3.3 Utilisation of Plant Nutrients

Sewage contains 26-70 mg/l of nitrogen (N), 9-30 mg/l of phosphate (P_2O_5) and 12-40 mg/l or even more of Potash (K_2O). The recommended dosages for N,P and K for majority of field crops are in the ratio of 5:3:2 or 3 respectively. The figures for N, P and K contents of sewage on the other hand show that sewage is relatively poor in phosphates. Excess potash is not of significance but a relative excess of nitrogen affects crop growth and development. Crops receiving excessive dosages of nitrogen show superfluous vegetative growth and decrease in grain or fruit yield. The phosphate deficit of sewage, therefore, should be made good by supplementing with phosphate fertilisers, the extent of phosphate fortification depending upon the nature of crop and its phosphate requirements. As the availability of phosphate is low, it would be desirable to apply the required quantity of phosphatic fertiliser at the time or even before (about a fortnight) the sowing or planting of the crop.

Even when sewage nutrients are balanced by fortification, irrigation with such sewage may supply excessive amounts of nutrients resulting in waste or unbalanced growth of plants with adverse effects on yields. It may therefore be necessary to dilute the sewage. Dilution also helps in reducing the concentration of dissolved salts and decomposable organic matter in the sewage thus decreasing hazards to the fertility of the soil. It is desirable to limit the BOD and total suspended solids of sewage to be disposed on land for irrigation to 180 and 250mg/l respectively, which can be achieved by primary sedimentation.

21.3.4 Protection against Health Hazards

Sewage farms should not be located within 1 km of sources of centralised water supply or mineral springs; in the vicinity where water bearing layers prevail; or on areas with ground water levels less than 2 m below the surface. Measures should be taken to prevent pollution of artesian water. Sewage farms must be separted from residential areas by at least 300m.

Evidence is on the increase to show that labourers working on the sewage farms suffer from a number of ailments directly attributed to handling of sewage. In view of this it is highly desirable to mechanise sewage farm operation to the extent possible.

Sewage or wastewaters of individual enterprises engaged in the processing of raw material of animal origin, of hospitals, biofactories and slaughter houses should be disinfected before they are taken to the sewage farms.

Disinfection and agricultural utilization of sewage containing radioactive substances are carried out in accordance with special instructions.

The staff of sewage farms must be well versed in the sanitary rules on the utilization of sewage for irrigation as well as with personal hygiene.

All persons working in sewage farms must undergo preventive vaccination against enteric infections and annual examination for helminthoses and provided treatment if necessary.

Sewage farms should be provided with adequate space for canteens with proper sanitation, wash-stands and lockers for irrigation implements and protective clothing; besides, safe drinking water must be provided for the farm workers and the population residing within the effective range of the sewage farms.

All the farm workers should be provided with gum boots and rubber gloves which must compulsorily be worn while at work. They should be forced to observe personal hygiene such as washing after work as well as washing before taking food. The use of antiseptics in the water used for washing should be emphasized. The farm workers should be examined medically at regular intervals and necessary curative measures enforced.

Cultivation of crops like lettuce, tomato, onion, carrot, chillies. parsley, turnip, radish, groundnut, celery, cucumber, melon, strawberry, garden berry, cabbage, etc. should be banned as they are eaten raw. Cultivation of paddy in bunded fields is likely to give rise to sanitation problems and hence undesirable. Growing of non-edible commercial crops like cotton, jute, fodder, milling varieties of sugarcane and cigarette tobacco would be suitable. Cultivation of grasses and fodder legumes, medicinal and essential oil yielding plants like mentha and Citronella may be allowed. Cultivation of cereals, pulses, potatoes and other crops which are cooked before consumption may be permitted, if sewage is treated and care is taken in handling the harvests to ensure that they are not contaminated. Cultivation of crop exclusively under seed multiplication programmes would be advantageous as these are not consumed. As an additional safeguard, sewage irrigation should be discontinued at least two months in advance of harvesting for fruits and berries, one month for all kinds of vegetables and a fortnight for all other crops. Direct grazing on sewage farms should be prohibited.

21.4 DISPOSAL OF SLUDGE

In case of farms using settled sewage, the sludge from the sedimentation tanks should be mixed with manure or peat and composted for which adequate land should be provided. The duration of composting is so chosen as to ensure complete dehelminthization of the sludge. The composted material is applied on fallow fields or in the farm itself.

Chapter 22

SEPTIC TANKS

In rural areas and the fringe areas of suburban towns and also in case of isolated buildings and institutions, such as hotels, hospitals, schools, small residential colonies, underground sewerage system with complete treatment of sewage may be neither feasible nor economical. In such cases, the generally accepted method of treatment and final disposal of domestic sewage without giving rise to public health hezard or nuisance is by the use of septic tanks followed by subsurface disposal of effluent. In areas with porous soil, this method gives satisfactory results. Septic tanks should be located as far away as possible from the exterior of any building and should not be located in swampy areas or areas prone to flooding. In clayey, nonporous soils or where houses are closely spaced, suitably designed leaching pits may have to be used if septic tanks cannot be avoided. Leading the septic effluents into open drainage system is not at all satisfactory as it would cause health hazards, nuisance and mosquito breeding. When facilities for connection to a sewer are opened up, the effluent from the septic tanks must be connected to the sewers or preferably, direct connections may be taken since the latter would obviate the need for separate arrangements of sludge removal.

A septic tank is a combined sedimentation and digestion tank where sewage is held for some period when the suspended solids settle down to the bottom. This is accompanied by anaerobic digestion of sludge and liquid, resulting in appreciable reduction in the volume of sludge and release of gases like carbon dioxide, methane and hydrogen sulphide. The effluent although clarified to some extent will still contain considerable amount of dissolved and suspended putrescible organic solids and viable pathogens and therefore the disposal of effluent merits careful consideration. Because of the unsatisfactory quality of the effluent and also difficulty of providing a proper disposal system for the effluent, septic tanks are recommended only for individual homes and small communities and institutions whose contributory population does not exceed 300. For larger communities, provision of septic tanks should be avoided as far as possible but may be extended to a population of 500 in undulating topography. For the septic tanks to function satisfactorily, a fairly adequate water supply is a prerequisite. Wastes containing excessive detergents and disinfectants are not suited for treatment in septic tanks as they adversely affect the anaerobic decomposition.

22.1 DESIGN CRITERIA AND CONSTRUCTION DETAILS

Rational design of a septic tank should be based upon the functions it is expected to perform, viz., (i) sedimentation to remove the maximum possible amounts of suspended solids from sewage, (ii) digestion of the settled sludge resulting in a much reduced volume of dense, digested sludge and (iii) storage of sludge and scum accumulating in between successive cleanings thereby preventing their escape. Thus the tank should have an effective capacity large enough to provide for the above three requirements.

22.1.1 Sewage Flow

The maximum flow to the tank is based on the number of plumbing fixtures discharging simultaneously rather than the number of users and per capita waste water flow expected to reach the tank. For this purpose various sanitation facilities such as water closets, wash basins, baths, etc. are equated in terms of fixture units (table 22.1). A fixture unit is a standard receptacle which gives a discharge of 10 lpm when flushed.

Table 22·1

Fixture Equivalents

Facility	*Equivalent Fixture Unit*
Water Closet . . .	1
Bath	1/2
Wash basin/kitchen sink .	1/2
Urinal (with auto flush) .	1
Urinal (without auto flush) .	1/2
Slop sink . . .	1
Laboratory sink . . .	2
Combination fiixture . .	1
Shower bath . . .	1
Bath tub	2
Drinking fountain . .	1/2
Ablution tap . . .	1/2
Dish washer . . .	1/2

The estimated number of fixture units and the number of fixture units that contribute to the peak discharge in small installations serving upto 50 persons, for residential housing colonies upto 300 persons and for eating establishments and boarding schools are given in tables 22.2, 22.3 and 22.4.

Table 22.2

Estimated peak discharge for small tanks upto 50 users

Number of users	Number of fixture units	Probable no. of fixture units discharging simultaneously	Probable peak discharge (lpm)
5	1	1	10
10	2	2	20
15	3	2	20
20	4	3	30
25	5	4	40
30	6	4	40
35	7	5	50
40	8	6	60
45	9	6	60
50	10	7	70

Table 22.3

Estimated peak discharge for residential housing colonies

Number of users	Number of Households	Number of fixture units	Probable peak discharge (lpm) (based on 60% fixture units discharging simultaneously)
100	20	40	240
150	30	60	360
200	40	80	480
300	60	120	720

Table 22.4

Estimate peak discharge for eating establishments, Boarding Schools and Similar Establishments

Number of Users	W.C.	Bath	Wash basin/ kitchen sink	No. of fixture units	Probable peak discharge (lpm)]' (based on 70% of fixture units discharging simultaneously)
50	6	6	6	12	84
100	12	12	12	24	168
150	19	19	19	38	266
200	25	25	25	50	350
300	37	37	37	74	518

22.1.2 Tank Dimensions

22.1.2.1 *Sedimentation*

Both surface area and detention or depth are important factors in the settling of flocculant particles such as sewage solids. For average Indian conditions at a temperature of 25° C, the surface area required will be 0·92 m^2 for every 10 lpm peak flow rate. This is based on 75% removal of sewage particles of size 0·05 mm and above with a specific gravity of 1·2. A minimum depth of sedimentation of 25—30 cm is necessary. The length is maintained 2—4 times the breadth.

22.1.2.2 *Sludge digestion*

The fresh sludge must stay in the tank long enough to undergo satisfactory anaerobic digestion so that as much of the organic matter as possible may be destroyed and the sludge may become innocuous and suitable for dewatering or drying.

The time required for digestion is temperature dependent and as seen from figure 22.1, the detention time in a septic tank can be computed on the basis of annual average temperature.

The per capita suspended solids entering the septic tank may be taken as 70 gm/day. Assuming that 60% of the solids is removed along with fresh sludge, of which 70% is volatile, with a solids content of 5% or moisture 95%, the volume of fresh sludge works out to 0·00083 m^3/cap/day. Considering that 2/3 of the volatile matter is destroyed of which 1/4 is mineralised during digestion and solids content of 13% in the digested sludge, the volume of the digested sludge works out

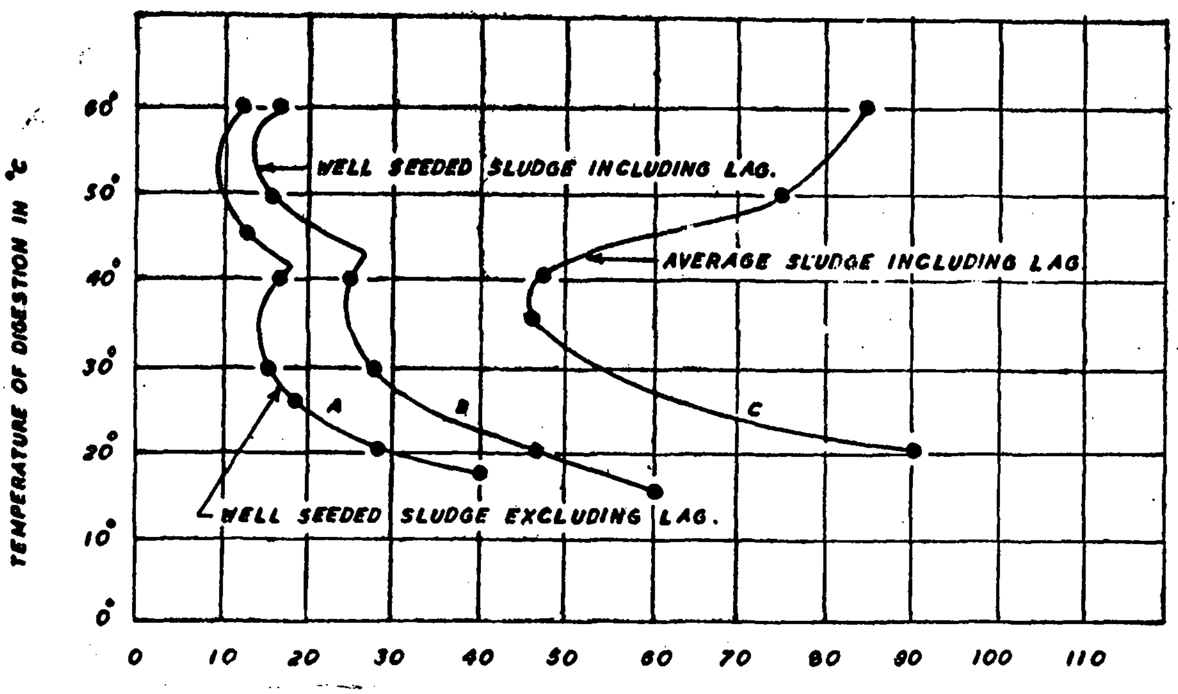


FIG 22 1 TIME FOR ANAEROBIC DIGESTION OF 10% OF WELL SEEDED SEWAGE SLUDGE to 0·0002 m³/cap/day. The digestion zone contains both fresh and digesting sludge and hence the digestion space should provide for the average volume of the mixture of fresh and digested sludge which works out to 0·000515 m³/cap/day. Based on the period of digestion, the capacity needed for the digestion zone could be determined. From Figure 22-1 at 25°C the capacity required for sludge digestion works out to $63 \times 0 \cdot 000515 = 0 \cdot 032$m³/capita.

22.1.2.3 *Sludge and scum storage*

Adequate provision should be made for the storage of digested sludge and scum in the tank as otherwise their accumulation interferes with the efficiency of the tank by encroaching upon the space provided for sedimentation and digestion. A sludge storage capacity of $0 \cdot 0002 \times 365 \times 100 = 7 \cdot 30$m³/100 persons for an interval of cleaning of one year is provided below the sedimentation zone.

22.1.2.4 *Total capacity*

The tank should also provide for a free board of not less than 30 cm. which should be sufficient to include the depth of scum above the liquid surface. No additional capacity for seed—sludge need be provided, if care is taken during desludging to leave about 5 to 10% digested sludge or a minimum of 25 to 50 mm depth of sludge on the tank bottom. When the cleaning is annual, the total tank capacity at 25°C for 10 persons will be about 2·15 m³ based on different requirements as follows :

(i) Sedimentation—Probable peak flow 20 lpm—Area required at 0.92 m²/10 lpm = 1.84 m² Providing a depth of 30 cm, volume=1.84×0.3	0·55 m³
(ii) digestion	0.32 m³
(iii) Sludge storage	0.73 m³
(iv) free board including 0.25 m³ for seed sludge (1.84×0.3)	0.55 m³
TOTAL .	2.15 m³

A septic tank designed on the criteria discussed normally provides a detention period of 24 to 48 hours based on an average daily flow of sewage. However, since the average daily flow varies so widely from one installation to another, detention period should not be considered as an important criteria for design of septic tanks.

22.1.3 **Construction Details**

The inlet and outlet should not be located at such levels where the sludge or scum is formed as otherwise, the force of water entering or leaving the tank will unduly disturb the sludge or scum. Further, to avoid short circuiting, the inlet and outlet should be located as far away as possible from each other and at different levels. Baffles are generally provided at both inlet and outlet and should dip 25 to 30 cm, into and project 15 cm above the liquid. The baffles should be placed at a distance of one fifth of the tank length from the mouth of the straight inlet pipe. The invert of outlet pipe should be placed at a level 5 to 7 cm below the invert level of inlet pipe. Baffled inlet will distribute the flow more evenly along the width of tank and similarly a baffled outlet pipe will serve better than a tee-pipe

For larger capacities, a two-compartment tank constructed with the partition wall at a distance of about two-thirds the length from the inlet gives a better performance than a single compartment tank. The two compartments should be interconnected above the sludge storage level by means of pipes or square openings of dia or side length respectively of not less than 75 mm.

Every septic tank shall be provided with ventilation pipes, the top being covered with a suitable cage of mosquito proof wire mesh. The height of the pipe should extend at least 2m above the top of the highest building within a radius of 20 m.

Septic tanks may be constructed by brick work, stone masonry or concrete *cast in situ* or precast materials. Precast household tanks made of materials such as asbestos cement could also be used, provided they are watertight and possess adequate strength in handling and installing and bear the static earth and superimposed loads.

All septic tanks shall be provided with watertight covers of adequate strength. Access manholes of adequate size shall also be provided for purposes of inspection and desludging of tanks.

The floor of the tank should be of cement concrete and sloped towards the sludge outlet. Both the floor and side wall shall be plastered with cement mortar to render the surfaces smooth and to make them watertight.

22.1.4 **Sludge Withdrawal and Disposal**

The digested sludge shall be withdrawn through a dip pipe of not less than 15 cm dia under a hydrostatic pressure of at least 45 cm. The sludge pipe shall deliver the sludge to the sump and be provided with a delivery valve to draw the sludge as required. The sludge may be disposed of in covered pits or into a suitable vehicle for removal from the site. Spreading of sludge on the ground in the vicinity should not be allowed. Portable pumps may also be used for desludging in which case there will be no need for sludge pipe or sludge sump. As far as practicable manual handling of sludge should be avoided.

Half yearly or yearly desludging of septic tank is desirable. But if it is not feasible or economical and if there is difficulty to find labour for desludging, small domestic tanks should be cleaned at least once in 2 to 3 years, provided the tank is not overloaded due to use by more than the number for which it is designed. A portion of the sludge not less than 25 mm in depth should be left behind in the tank bottom which acts as the seeding material for fresh deposits.

22.2 SECONDARY TREATMENT AND DISPOSAL OF EFFLUENT

Although sewage undergoes treatment in a septic tank, the effluent may still contain pathogenic organisms and hence septic tank effluents cannot be considered safe. The effluent coming out of the tank will be septic and malodorous and hence more objectionable than the liquid that goes into the tank. This however, does not mean that the tank has no value. Its primary purpose is to condition the sewage so that it will cause less clogging of the dispersal field or make it more amenable to other treatment. Final purification of the effluent and the removal and death of pathogans is effected by percolation through the soil or other media.

In general, the disposal of effluent may be either underground or overground. Normally underground disposal either in the form of soak pits or dispersion trenches is practised. Both these methods are designed to achieve subsurface percolation or seepage into the soil. Satisfactory disposal therefore depends, to a great extent, on porosity and percolation characteristics of the soil. In addition, other factors, such as, level of subsoil water table, the climatic conditions, presence of vegetation, aeration of soil and concentration of suspended solids in the effluent also influence the application of these methods. Soakpits or dispersion trenches can be adopted in all porous soils where percolation rate (as discussed in the Appendix—21) is below 25 minutes per cm and the depth of water table is 180 cm or more from the ground level. Dispersion trenches should be preferred in soils with percolation rates between 12 and 25 minutes if adequate land is available. In areas with higher water table, dispersion trenches should be located partly or fully above ground level, in a mound.

The subsoil dispersion system shall be atleast 20 m away from any source of drinking water. It should also be as far away as possible from the nearest dwellings but not closer than 6 m to avoid any corrosive effect due to tank gases vented into atmosphere. Subsoil dispersion system is not recommended in limestone or crevice rock formations where there may be solution cavities which may convey the pollution to long distances and pollute water resources. In impervious soils such as dense clays and rocks, where percolation rates exceed 25 minutes, adoption of upflow or reverse filters, trickling filters, subsurface sand filters or open sand filters followed by chlorination should be considered, particularly for larger installations.

In the absence of information relating to ground water or subsoil, subsurface explorations are necessary. Percolation tests as described in Appendix—22 determine the acceptability of the site and serve as the basis of design for liquid absorption. The total subsurface soil area required for soak pits or dispersion trenches is given by the empirical relation :

$$Q=130/\sqrt{t} \qquad \ldots\ldots(22\text{—}1)$$

where

Q=maximum rate of effluent application in lpd/m^2 of leaching surface; and

t=standard percolation rate for the soil in minutes.

In calculating the effective leaching area required, only area of trench bottom in case of dispersion trenches and effective side wall area below the inlet level for soak pits should be taken into account.

22.2.1 Soak Pits

Soak pits or seepage pits are cheap to construct and are extensively used. They need no media when lined or filled with rubble or brickbats. The pits may be of any regular shape, circular or square being more common. When water table is sufficiently below ground level, soak pits should be preferred only when land is limited or when a porous layer underlies an impervious layer at the top, which permits easier vertical downward flow than horizontal spread out as in the case of dispersion trenches. Minimum horizontal dimension of soak pit should be 1 m, the depth below the invert level of inlet pipe being at least 1 m. The pit should be covered and the top raised above the adjacent ground to prevent damage by flooding.

22.2.2 Dispersion Trenches

Dispersion trenches consist of relatively narrow and shallow trenches about 0·5 to 1 m deep and 0·3 to 1 m wide excavated to a slight gradient of about 0·25%. Open jointed earthenware or concrete pipes of 70 to 100 mm size are laid in the trenches over a bed of 15 to 25 cm of washed gravel or crushed stone. The top of pipes shall be covered by coarse gravel and crushed stone to a minimum depth of 15 cm and the balance depth of trench filled with excavated earth and finished with a mound above the ground level to prevent direct flooding of trench during rains. The effluent from the septic tank is led into a small distribution box from which several such trenches could radiate out. The total length of trench required shall be calculated from the equation (22—1) and the number of trenches worked out on the basis of a maximum length of 30 m for each trench and spaced not closer than 2 m apart. Parallel distribution should be such that a distribution box should be provided for 3 to 4 trenches.

22.2.3 Upflow Filter

The upflow filter can be successfully used for secondary treatment of septic tank effluent in areas where dense soil conditions, high water table and limited availability of land preclude soil absorption or the leaching system for effluent disposal. It is a submerged filter with stone media and the septic tank effluent is introduced from the bottom. The microbial growth is retained on the stone media making possible higher loading rates and efficient digestion. The capacity of the unit is 0·04 to 0·05 m^3 per capita or 1/3 to 1/2 the liquid capacity of the septic tank it serves. BOD removals of 70 % can be expected. The effluent is clear and free from odour and nuisance. This unit has several advantages viz. (a) a high degree of stabilisation; (b) little sludge production; (c) low capital and operating cost; (d) low loss of head in the filter (10 to 15 cms) in normal operation.

CHAPTER 23

PLANT OPERATION AND MAINTENANCE

Maintenance comprises those operations which serve to keep equipments and processes functioning properly without interruption. It can be classified as (a) preventive maintenance which constitutes works and precautions to be taken to prevent breakdown and (b) corrective maintenance which involves carrying out repairs after breakdown. Preventive maintenance is more economical than corrective maintenance and provides uninterrupted service which is essential to achieve the basic objectives, of treatment, *viz.*, protection of health of the community and prevention of nuisance.

The primary aim of sewage treatment plant operation is the maintenance and running of the plant efficiently and economically, so that the effluent from the plant meets the regulatory standards and could be discharged safely on land or into water bodies.

The basic requirements of successful operation and maintenance of sewage treatment plants are :

(i) a thorough knowledge of the processes and equipments;

(ii) proper and adequate tools;

(iii) adequate stock of spare parts and chemicals;

(iv) assignment of specific maintenance responsibilities to operating staff;

(v) systematic and periodic inspection and strict adherence to servicing schedules;

(vi) training of all operating staff in proper operating procedures and maintenance practices;

(vii) overall supervision of operation and maintenance schedules; and

(viii) good house keeping.

The main units of the plant are designed for maximum efficiency within a certain flow range and sewage quality. Close control and coordination of operation of different units are therefore required within the limits of design. Hence accurate measurements of flow of raw sewage, settled sewage, air, mixed-liquor, sludge and final effluent are required. For this purpose flow measuring devices and meters, preferably of the indicating and recording types, are provided to guide the operator in his supervision and to obtain data for progressive improvement. For quality control, analysis of sewage, sludges, etc., as they pass through different units of the treatment plant and of the effluent should be carried out on a regular basis. Proper recording of data is essential for an accurate assessment of efficiency of operation. On the chemical side, dosages must be closely and accurately proportioned to the varying rates of flow of sewage and sludge.

Better plant operation is possible only when the operator is fully conversant with the characteristics and composition of sewage handled and the results achieved during each stage or unit of the treatment process.

Operation and preventive maintenance of several treatment units and the frequency of cleaning, lubrication of mechanical equipments etc., are to be strictly adhered to if optimum results are to be expected.

23.1 TREATMENT UNITS

23.1.1 Screens

Hand cleaned screens should be cleaned as often as required to prevent backing up of sewage in the inlet sewer.

Mechanical screens should be kept properly lubricated as per instructions of the manufacturers. The entire mechanism should be painted at least once a year. Slack in chains should be promptly replaced with spare links. Periodic inspection of mechanical and automatic screens is essential to ensure that the equipment is functioning properly.

Screen chambers should be hosed at least once a day to keep clean and the walls should be scrubbed at least once a week.

Prompt and hygienic disposal of screenings is necessary. Mechanical screens may discharge screenings into wheel-barrows or closed cans. Burial is the most common method of disposal. Composting with city refuse, trenching under earth cover and incineration can also be adopted. Where shredders are used, screenings should be washed to remove grit to prevent wearing of the cutting edges of shredders.

Daily record of operations should be maintained to show frequency of cleaning, volume of wet screenings removed and daily power consumption for mechanically operated screens. Besides, record should also be made of time-settings between strokes for mechanically operated screens.

23.1.2 Grit Chamber

The frequency of grit removal should be adjusted such that the storage compartment is never more than about halffull at any time. Cleaning of grit chamber becomes essential after a heavy storm, particularly when sewage is received from a combined sewerage system.

In manual cleaning, the flow is shut off, the chamber emptied by gravity or pumping and the grit hauled by using long handle shovels, pail buckets and wheel-barrows. The operator must always use gumboots and hand gloves. At least two men should be present during cleaning operations.

Inspection of mechanically cleaned grit chambers consists of disposal of washed grit, lubrication of mechanical equipments as per manufacturers' schedule and routine inspection.

Grit should be disposed of by burial an should not be thrown on ground.

The record of operation should show the dates of cleaning, amount of grit removed, flow through the chamber between cleanings and method of disposal of the grit.

23.1.3 **Sedimentation Tanks**

23.1.3.1 *Sludge*

Sludge removal should be sufficiently frequent to avoid development of septic conditions.

Sludge is removed continuously in some plants and 2 to 4 times a day in others. Continuous removal reduces floating scum and results in a thick sludge.

Sludge from the primary sedimentation tank is drawn from the sludge sump by means of a pump discharging into digesters. Sludge from secondary settling tanks is pumped partially to aeration tanks and partially to primary sedimentation tanks. While drawing the sludge, the operator should take samples and adjust the pump capacity according to quantities required for feeding the digester or returned to aeration or primary sedimentation tanks. Excessive sludge pumping and withdrawal of watery sludge should be avoided. When the sludge is drawn by hydrostatic pressure, the valve on the pipe is opened partly and the sludge allowed to flow out. When the sludge becomes thick the valve should be closed. Most plants utilizing this method are so designed that the operator can see the sludge as it is being drawn and judge when the valve should be closed.

In sedimentation tanks provided with mechanical sludge scrapers, sludge may be withdrawn continuously or at predetermined intervals by automatic starting and stoppage of pumps. The setting of this equipment should be periodically checked.

23.1.3.2 *Skimmings*

Floating materials collecting on the surface of primary sedimentation tanks are removed by skimming devices operated mechanically. Where such mechanical skimmers are not provided, manual removal by a scum shovel at least once a day is recommended. In mechanical skimming device, the skimmer brush trips the scum into a scum trough discharging into a sludge sump, from where it is pumped along with the sludge. The skimmer device should be inspected periodically and moving parts lubricated.

23.1.3.3 *Structures*

The sidewalls of the settling tanks should be such that collection of solids, grease, oil and aquatic growths are avoided. Collections, if any, should be removed periodically by brushing and hosing them down without disturbing the tank contents. Dark floating matter and rising bubbles on the surface indicate improper cleaning and inadequate sludge removal.

Inlet and outlet channels should be kept clean and hosed at least once a week. All baffles should be cleaned of any sticky materials and stringy growths on the surface and edges.

The bearings, transmission gears, traction rollers, etc., should all be properly lubricated as per the lubricating schedule suggested by the manufacturers of skimmer and scraper mechanisms.

In addition, it is good practice to dewater each clarifier at least once a year to inspect the submerged portions of the mechanism such as flight scrapers, squeeges, etc., repair or replace the worn-out parts, check all nuts and bolts for tightness and repaint, if necessary. Motors should be checked periodically for overload conditions and electric wirings for proper insulation.

23.1.3.4 *Records*

The daily operation records should show frequency and method of cleaning, flow through time, volume of sludge and scum removed and percentage moisture in sludge, temperature and pH of sludge and settleable solids both in sewage and in effluent from sedimentation tanks. The suspended solids, BOD and COD of both influent and effluent should be reported at least once a week.

23.1.4 **Aeration Tanks**

The operational variables in an activated sludge plant include rate of flow of sewage, air supply, MLSS, aeration period, DO in aeration and settling tanks, rate of sludge return and sludge condition. The operator should possess a thorough knowlege of the type of system adopted viz., conventional, high rate, extended aeration or contact stabilization, so that effective control of the variables can be exercised to achieve the desired efficiency of the plant.

23.1.4.1 *Sewage flow*

Since the activated sludge treatment is biochemical in nature, conditions in the aeration tank should be maintained uniform at all times. Sudden increases in rate of flow or slugs of flow of activated sludge should be avoided. Supernatants from digester containing more than 3000 mg/l of SS if taken into the settling tank, should be pretreated as otherwise a heavy load will be imposed on the activated sludge system. Weekly or preferably daily measurements of sewage flow and the BOD applied to the aeration tank should be made.

23.1.4.2 *Air supply*

Frequent checks of DO at various points in the tank and at the outlet end which should not be less than 1 mg/l, will help in determining the adequacy of the air supply. The uniformity of air distribution can be easily checked by observing bubbling of the air at the surface, which should be even over the entire surface area of the tank. If the bubbling looks uneven, clogging of diffusers is indicated. Clogging is also confirmed by the increase of 0.1 to 0.15 kg/cm² in the pressure gauge reading. Adding chlorine gas to air may help in removing clogging of diffusers on air side if it is due to organic matter. Other methods of cleaning will have to be resorted to, if this procedure does not clear up the clogging. Air flow meters should be checked periodically for accuracy and hourly and daily air supply and air pressures should be recorded to avoid over-aeration or under-aeration. Mechanical or surface aerators should be kept free from fungus or algal growths by cleaning them periodically.

23.1.4.3 *Mixed liquor suspended solids*

Control of the concentration of solids in the mixed liquor of the aeration tank is an important operating factor. Although it is most desirable to hold the MLSS constant at the suggested rates, it is not practicable as it involves frequent measurements of SS likely to be affected by the varying hourly rates of return sludge. The test for MLSS should be done at least once a day and the hour of test should be so chosen that the sewage flow for the previous several hours is more or less of the same pattern each day. Usually 8 to 9 A.M. will be the most satisfactory hour. As the MLSS will be minimum when the peak flow starts coming in and will be maximum in the night hours when the flow drops, the operating MLSS value would be the average hourly value in a day which should be verified atleast once a month. In case of very large plants regular hourly check is desirable.

23.1.4.4 *Return sludge*

The return sludge pumps provided in multiple units should be operated according to the increase or decrease in return sludge rate of flow required to maintain the necessary MLSS in aeration unit, based on the sludge volume index. The sludge volume index should be determined daily to know the condition of the sludge. A value of over 200 definitely indicates sludge bulking.

When sludge bulking occurs, the suggested remedies are : (i) reduction in rate of sewage flow into aeration tanks; (ii) reduction in ratio of return sludge; (iii) increase in air supply or (iv) dilution of incoming sewage. Chemicals that may be used to reduce bulking are chlorine, lime (raising pH to 8.6 to 8.8) or chlorinated coppers. These are added to the return sludge in small doses to ensure that they do not become toxic to micro-organisms.

A good operation calls for prompt removal of excess sludge from the secondary tanks to ensure that the sludge is fully aerobic. This should be measured daily and recorded. The excess sludge is usually taken into primary clarifier for further treatment in digesters or disposed of otherwise.

23.1.4.5 *Foaming*

Foaming or frothing is sometimes encountered in activated sludge plants when the sewage contains materials which reduce the surface tension, the synthetic detergents being the major offender. Froth, besides being unsightly, is easily blown away by wind and contaminates all the surfaces it comes into contact. It is a hazard to workmen because it creates a slippery surface even after it collapses. Foam problems can be overcome by the application of a spray of screened effluent or clear water, increasing MLSS concentration, decreasing air supply or application of a spray of kerosene or addition of other special anti-foam agents. The presence of synthetic anionic detergents in sewage also interferes with the oxygen transfer and hence reduces aeration efficiency.

23.1.4.6 *Microscopic examination*

Routine microscopic examination of solids in aeration tank and return sludge to identify the biological flora and fauna present, will enable a good biological control of the aeration tanks.

23.1.4.7 *Records*

Activated sludge operation should include recording of flow rates of sewage and return sludge, DO, MLSS and biological life, sludge age, air supply rate and aeration period in hours, SS BOD, COD and nitrates in both influent and effluent.

23.1.5 **Trickling Filters**

23.1.5.1 *Distributors*

All clogged spray nozzles or orifices in the revolving distributors should be cleaned as soon as clogging is noticed. Dosing tanks should be kept free from accumulation of deposits. Placing of fine screens in the discharge channel of the sedimentation tank prevents the entry of coarse solids into the filter.

All parts of the filter bed should receive equal loading which should be tested periodically by using watertight pans of standard size 90 cm × 120 cm set flush with the top of filter media, end to end, along the radius. The media surface shall be divided into two concentric circles with the area of the inner being 10% of the total area covered by the distributor. The sewage collection in the pan for 10 revolutions of the distributor when the air is still, is measured. The rate of distribution should not vary more than ± 5% from the mean rate of distribution in the outer 90% area and ± 10% in the inner 10% area.

Rotary distributors should be inspected daily for the purpose of keeping all nozzles clean and in operating condition, keeping the arms level by adjusting buckles on the guy rods or cables and keeping the guide rollers in proper adjustment. It is advisable to take out each rotary distributor once a year to paint all surfaces with an anticorrosive black paint.

In cold climates, spray nozzles should be kept free from freezing by operating the drain valve at the terminal end of the distribution system.

Where dosing tanks with siphons are installed, adequate head should be maintained in dosing tanks. Siphons should be checked for air leaks. Grease and solids accumulate in the dosing tank during high water level and pass into the siphon and clog nozzles. Hence, periodic cleaning of dosing tanks is necessary.

23.1.5.2 *Ponding*

Pools or ponds sometimes form on the surface of the filter. This is due to organic growth or retained organic matter from poorly settled sewage. Sometimes, this is due to careless dumping of fine material in one place, at the time of placing media in the filter box. In many cases the trouble lies in the top layer of the media and forking or raking the media to a depth of 20 to 30 cm will effectively remove ponding. Washing the filter media with a jet of water or giving rest to the filter for 2 to 3 days may also be effective. Prechlorination of sewage or application of caustic soda upto 10 mg/l has also been tried with success to eliminate clogging and ponding problems. When using chemicals, treatment may be given for 8 hour periods on alternate days.

23.1.5.3 *Underdrains*

Filter underdrains should be inspected frequently for clogging. If clogging is evidenced by reduced flow from any drain, this should be flushed and cleaned with sewer rods.

23.1.5.4 *Odour*

Odours from septic sewage must be controlled by eliminating the causes before sewage reaches the spray nozzles, prechlorination being probably the most effective in controlling odour.

23.1.5.5. *Filter flies*

Psychoda filter flies sometimes infest the filters and cause not only nuisance to the workers but also clog the beds. Flooding the bed for 24 hrs at intervals of 9 to 10 days, application of chlorine at a rate of 3 to 5 mg/l or gammexane at a rate of 180 g/ha or D.D.T. at a rate of 3-10 kg/ha of wall surface once a week are the methods available for flushing of the larvae. Adult flies are controlled by pyrethrum spray. Allowing the bed to dry is not a good practice as it will inactivate the microorganisms.

23.1.5.6 *Records*

Operating records should show the units of filter in service each day, the number of nozzles cleaned, the dates of cleaning the distributors and underdrains and the rates at which the filters were operated. In the case of high rate filters, recirculation pumps should be operated according to the time schedule. Quantity recirculated and hours of recirculation should be properly recorded. Dates on which measures were taken for correction of pooding and psychoda control should also be recorded.

23.1.6 **Sludge Digestion Tanks**

Sludge should be added according to a set schedule preferably spread over as long a period as possible. Sludge withdrawals should not be excessive since this may lead to retardation of digestion.

23.1.6.1 *Digester operation*

For start up, digester tanks with fixed covers should be filled initially with water, sludge or sewage to expel air. In tanks with floating cover, the cover should be brought down to the lowest point before filling of the tank is commenced. In order to reduce initial lag period, raw sludge mixed with digested sludge in the ratio of 2:1 to 4:1 may be pumped to the digester so that alkaline digestion starts within a few days after loading. The addition of fresh sludge should commence only after this stage. If digested sludge is not available, raw sludge alone is pumped and kept for 2 to 3 weeks before the digester can be loaded. Open digesters can be charged directly.

The raw sludge feeding rate should be such that the volatile solids in the digester should not exceed 3 to 5% so that digestion is not inhibited. Generally a loading rate of 1 to 2 kg of fresh solids to every 40 to 50 kg of digesting volatile solids should be the ratio to maintain a uniform digestion rate.

Where digesters are equipped with mixing devices, they should be operated in accordance with the manufacturer's instructions. Where facilities for recirculation by pumping exists, they should be used for mixing digester contents, breaking down scum, mixing lime with sludge for pH adjustment etc.; where there is no mixing and recirculation facility, the operator has to rely upon natural mixing of raw and digested sludge in the digestion unit.

Generally heating of digesters by providing internal or external heat exchange units is not required in many parts of the country. But where they are installed, the temperature of the hot water pumped through, should not be above 55°C to prevent sludge caking on the outer surface of coils causing loss in heat transfer efficiency. Digestion is generally carried out in the mesophilic range and the temperature of sludge generally varies from 25°C to 35°C. Thermometers to record temperature of sludge should be kept in order and reading noted twice or thrice a day.

Sludge should be withdrawn from the digesters only when it is fully digested, judged by the dark greyish brown colour without visible raw sewage sludge solids. Sludge should be sampled and tested to find out the condition before withdrawing. Generally not more than 10% of the capacity of digester should be drawn at a time, sludge withdrawn being limited by the capacity of the sludge drying beds.

Frequent pH tests of the sludge should be made and this should be correlated with the alkalinity of the supernatant of the sludge which may range from 1500 to 3000 mg/l. This affords and excellent check on operation. Digestion proceeds most favourably at pH values of 7.0 to 7.6 preferably above 7.2. If the pH is below 7.0, it is usually desirable to raise the pH by adding lime to the sludge as it enters the digester. The alkalinity of the supernatant is a useful guide to control the dosage. A start may be made using 20 to 40 kg of lime per m^3 of sludge, with more added if the pH value or alkalinity does not rise appreciably in a few days. The lime can be added at the sludge pump or in the recirculation pump as recirculation is helpful in bringing about mixing. In many plants, the first year of operation is generally troublesome.

Difficulties in the digestion tanks such as foaming due to overloading or accumulation of acid sludge or excessive formation of H_2S have to be corrected by neutralization and adjustment of pH. H_2S in moist gas leads to corrosion of meters, piping and flame trap through which the digester gas is drawn. This can be overcome by the removal of the H_2S by passing the gas through iron oxide or other scrubbers or by heating the gas to a high temperature to eliminate moisture in it.

Where digester gas is utilised for heating digesters or operating gas engines, etc., the equipment supplied for handling the gas should be installed and operated strictly according to manufacturer's instructions. As the gas is highly explosive, ordinary plumbers should not be engaged in correcting any defects in the gas collection system.

Gas pipes should be kept free from sediments, gas meters being periodically lubricated and fusible plugs in the flame traps frequently checked.

If the pumping of the supernatant liquor, having very high BOD and SS, into sedimentation tank adversely affects it, the liquor may be treated on sand beds or discharged separately.

23.1.6.2 *Records*

Records of the pump capacity and the pumping hours of the sludge from settling tanks to the digester should be maintained. The temperature and pH values of sludges should be recorded daily. The dates of withdrawal of sludge, amount drawn, amount of sludge loaded on drying beds and the depth of loading should be recorded. Records of daily estimation of per cent dry solids in raw sludge, total and volatile solids in the digesting and digested sludges, volatile acids in the digesting sludge and BOD and SS of supernatant should also be maintained. Daily records of the gas production as measured through gas meters and weekly records of gas analysis for percentage composition of methane should be maintained particularly where gas is utilised to produce energy.

23.1.7 **Sludge Drying Beds**

Sludge that is drawn to the beds should contain 4—10 % solids depending upon the type of sludge. Excess water must be removed by sludge thickeners, if necessary.

Wet sludge should be applied to the beds to a depth of 20 to 30 cm. After each layer of dried sludge has been removed, the bed should be raked and levelled. Sludge should never be discharged on a bed containing dried or partially dried sludge. It is preferable to apply the sludge at least a day or two after the sludge cakes are removed.

Removal of dried sludge from bed surfaces should be done with a fork, taking care that as little as possible of the sand is removed. When the sand layer is reduced to as low as 8 to 10 cm, it should be examined for clogging by organic matter and if found so, the entire sand should be removed and the bed resanded to the original depth of 20 to 30 cm.

The dried sludge cakes should be ground, sieved and bagged for sale as fertilizer. Some part of the sludge should be used in the plant itself for gardening, lawns, etc. to demonstrate its fertilizer value and to develop a market value for the digested and dried sludge.

Records of operation of sludge drying beds should show the time and quantity of sludge drawn to each bed, the depth of loading, the depth of sludge after drying time and the quantity of dried sludge removed. The solids content of wet digested sludge, its volatile portion and pH should be determined and recorded. Likewise the moisture content and fertilizer value in terms of NPK of dried sludge should also be analysed and recorded.

23.1.8 **Stabilization Ponds**

As stabilization ponds require comparatively less operation and maintenance, they are often neglected with the result that several failures have been reported. Hence proper supervision and good housekeeping are needed to ensure that the ponds are well preserved and the expected performance is attained.

23.1.8.1 *Operation*

Before the pond is put into operation the bottom is cleared of vegetation and debris. Raw sewage is then admitted to stand to a depth of 15 to 30 cm, small quantities of sewage being added each day to maintain this depth till algae establishes itself naturally in a week or two. After ensuring the establishment of algae, the pond is gradually loaded to raise the water level by 15 cm each day till the entire pond is filled. The pond is then given rest for 2 to 3 days to ensure optimum growth of algae before loading the pond for continuous operation at the designed flow.

If necessary and if algae is readily available the pond may be seeded by spreading a bucketful of algal culture all over the pond.

Ponds treating industrial wastes or a combination of sewage and industrial waste, may need acclimatization of algal species and addition of nutrients if found deficient.

The bunds should be inspected for the condition of the berms, for any burrows by rodents, condition of pitching and erosion due to wind, wave action and rain. Any defects noticed should be promptly set right.

Ponds should be inspected for characteristic changes in colour and odour. A change in colour or odour in all probability forecasts major change in the performance of the pond system. Odour may be caused by the setting in of anaerobic conditions or entry of industrial waste or overloading. Colour changes may be caused by change in volume of inflows, organic loads, temperature, transparency of liquid or light intensity. An aerobic or facultative pond functioning properly will look green on the surface. If the colour changes from green to black accompanied by floating matter, it may be due to too rapid fermentation of bottom sludge, frequent changes in characteristics of incoming waste or overloading. Anaerobic lagoons look greyish pink or light pink. If the ponds turn deep red, an invasion by sulphur bacteria is indicated and the sulphate contents of the incoming wastes should be controlled.

The pond should be regularly cleared of floating mats of algae at the corners and sides. All marginal growth of weeds and vegetation should be removed by suitable implements. Herbicides should be used only when the growth is unwieldy and cannot be effectively cleared by manual or mechanical means. The overgrown and dead grass on slopes should be periodically cut and removed.

Access roads, fencing, etc. should be inspected regularly and repairs, if needed, should be attended to promptly.

Maximum, average and minimum daily flows into the pond over the weir or parshall flume installed at the entry to the ponds are measured and recorded periodically.

In small installations, where there is no facility for analysis, samples of influent, and effluent should be collected at least once a month or when the pond condition appears to deteriorate visibly and got analysed at the nearest laboratory.

In large plants, where a laboratory is always attached, daily analysis should be carried out. The tests to be conducted may include all or a few of the following depending on the nature of the waste, size of the plant and the quality of effluent required: (1) BOD, COD, MPN of coliform organisms total, suspended and volatile solids for both influent and effluent; (2) diurnal variation of pH and DO in the pond; (3) total organic nitrogen, ammonia, nitrates and phosphates of effluent if used as irrigants and (4) on anaerobic ponds, oxidation reduction potential (ORP).

Overloading of ponds leads to anaerobic conditions. When a pond becomes anaerobic due to overloading, measures should be taken to rectify it immediately by adding sodium nitrate to supplement oxygen, agitating the surface, recirculating the pond effluent or bypassing a portion of the flow.

Mosquito breeding in the pond should be prevented by removing all weed growth and marginal vegetation, using larvicidal measures only as a last resort. Seeding the pond with sufficient number of water minnows, such as gambusia, which feed on larvae and eggs of mosquitoes may keep the mosquito population under control.

Fly breeding may be another problem in badly maintained ponds. Good house keeping and proper operation is essential to avoid fly breeding. Floating matter and scum should be removed daily, or broken down and drowned by a water spray.

In general, a well maintained pond with clean surroundings, free of water collections, debris, etc., and a little bit of gardening and landscaping will present aesthetic sight.

23.1.8.2 *Records*

Operation records should include daily inflow rates, daily or weekly or monthly analysis of influent and effluent, diurnal variations of the temperature, pH and DO in the ponds, dates and nature of maintenance repairs, dates of clearing of weeds, vegetation, etc.

23.2 BUILDINGS AND EQUIPMENT

23.2.1 Buildings and Other Structures

All buildings and structures should be well ventilated and illuminated, They should be maintained and kept in good repair, white or colour washed with metallic parts being painted annually. The effect of corrosive gases like H_2S could be minimised by proper ventilation, proper collection and disposal of corrosive gases and painting the structures which are prone to be attacked by the gas, with anticorrosive paints. Dampness inside buildings could be reduced by proper ventilation.

23.2.2 Equipment

The Operator should maintain a book of catalogues supplied by manufacturers containing instruction sheets of all equipments. In addition, printed or written operating and maintenance schedules should be posted near each equipment in the language understood by all operating staff.

Lubricating schedules, cleaning and painting schedules, checks for efficiency, leaks and wear and tear and testing of safety devices, should be followed strictly according to manufacturers instructions.

All metering devices such as weirs and float gauges should be maintained in proper working condition. Charts should be changed at the same hour every day. Records maintained should show total, maximum and minimum rates of flow.

Operating, lubricating and maintanance instructions for all pumps should be strictly followed. Special attention should be given to maintaining centrifugal pumps in an efficient operating condition, free from clogging, excessive friction or entrance losses and abnormal power consumption due to worn out or improperly designed impellers.

Wet wells should be drawn to the minimum elevation only and all accumulations of grease and other deposits removed promptly.

Floats and sequence switches controlling the pumping cycles should be examined at the beginning of each shift. All pumps including standby pumps should be operated in rotation so that the wear and tear is distributed evenly.

All bearings, motors and electrical control equipment should be inspected daily for any overheating. The manufacturer's directions for operation and lubrication should be strictly followed. Packing glands should be checked for overtightening.

Pumps may sometimes be affected by operating them for too brief intervals of less than a minute or two. A reversing switch must be installed for dislodging the clogging materials.

Chlorination equipments should be properly housed and reserve supply of cylinders, valves, gaskets, etc., should always be available. Valves and pipings should be regularly checked for leaks. Leaks should be attended to as per the instructions in the manufacturer's catalogues. Chlorine cylinders should be kept on scales and the weight read each day as a check for the amount of chlorine used. Gas masks must be used while attending to chlorine leaks. Operation records should show the volume of sewage chlorinated, rate of application of chlorine, residual chlorine in the plant effluent and the amount of chlorine consumed each day.

23.3 SAFETY IN THE PLANT

The work of an operator in a sewage treatment plant presents many hazards that must be guarded against. Common type of accidents are injuries from falls and deaths from drowning. Narrow walks or steps over tanks (particularly in darkness, rains and wind), ladders and spiral staircases are potential danger spots where the operator should be alert; overexertion during operation of valves, moving weights and performing other arduous tasks should be avoided. All open tanks should be provided with guard rails to prevent accidental falls. Glass parts as well as moving parts should be screened or guarded.

Gas poisoning, asphyxiation and gas explosion are other hazards. Hence smoking or carrying open flames in and around digesters should be prohibited. Covered tanks or pits should be well ventilated for several hours by keeping them open or by blowing in air, as they present problems of asphyxiation. Entry into them should be permitted only after ensuring the safety.

Gas masks should be stored in locations where no possibility of contamination by gas exists and should be easily accessible. A first aid kit should be available readily at hand. Fire extinguishers of the proper type should be located at strategic points and maintained in good operating condition. All staff should be trained in rendering first aid and operating fire extinguishing equipment.

Adequate number of toilets and bathing facilities, drinking water facilities and lockers should be provided for the convenience of operating staff and protection from risk of infection. Eating facilities and canteens should be provided with proper sanitation where the number of workers per shift is considerable. Suitable signboards at appropriate places should be displayed drawing attention to the potential danger spots.

All workers should be compelled to observe personal hygiene such as washing after work as well as washing before taking food. The use of antiseptics along with washing should be emphasized. They should be examined medically at regular intervals and necessary curative measures provided.

23.4 TRAINING OF PERSONNEL

The number and categories of operating staff required for a sewage treatment plant varies from plant to plant depending on the type of treatment facilities, the capacity of the plant, the degree of treatment required and the nature of effluent disposal. The personnel required may consist of a plant superintendent, chemists, plant operators, mechanics, welders, helpers and unskilled labourers. All operating staff engaged in technical and skilled work should be trained. Large plants should be headed by a plant superintendent who should have the necessary academic training in public health engineering with considerable experience in sewage treatment. Other plants must be placed in charge of a superintendent, who should be an engineer with orientation in public health engineering with experience in operation and maintenance of sewage treatment plants. All junior, operating staff should receive inservice training. It is desirable that all sewage treatment plants are run and maintained by operators who hold certificates of competency.

23.5 RECORDING AND REPORTING

All operating records of the various treatment units in a plant should be properly compiled on a day to day basis and daily, monthly and yearly reports prepared, maintained and periodically reviewed. These reports will form a valuable guide to better operation and serve as an important document in the event of a legal suit resulting from nuisance or danger attributed to the plant or for convincing the statutory authorities about the satisfactory performance of the plant.

23.6 CHECK LIST

Appendix-23 gives a check list of the operation troubles normally encountered in the sewage treatment plants along with the possible remedial measures.

CHAPTER 24

PLANT CONTROL LABORATORY

A well designed and adequately equipped laboratory under a competent analyst is essential in all sewage treatment plants. Very small size plants such as stabilization ponds need not have their own laboratories if the facilities of a nearby laboratory are available. The results of the laboratory analysis will aid in the characterisation of any waste water, pinpoint, difficulties in the operation and indicate improvement measures, evaluate the composition of effluents and thus estimate the efficiency of operation and also measure the probable pollutional effects of the discharge of such effluents upon the receiving water courses. The analytical data accumulated over a period of time is an important document in safeguarding the treatment plant from allegations of faulty operation. The laboratory should also engage in research and special studies for evolving improvements and innovations in the plant operation. The laboratory, therefore must form an integral part of the treatment plant.

24.1 PLANNING OF LABORATORY FACILITIES

24.1.1 PHYSICAL FACILITIES

The actual design of the laboratory depends on the size and type of treatment plants and type and volume of analytical work required to be carried out. Due consideration, therefore, should be given to the space requirement for permanent installed equipments and smooth performance of analytical work by the personnel. Necessary provision for future expansions should also be incorporated in the laboratory design.

24.1.1.1 *Size of the laboratory*

The size and equipments needed for the laboratory depends on the magnitude of the treatment plant. Even the smallest plant shall be provided with a laboratory, where atleast a few simple analyses such as SS, pH, BOD, and residual chlorine can be made. On the other hand large plants providing complete treatment may require a well planned laboratory building with facilities for physical, chemical, biological and bacteriological work.

24.1.1.2 *Location*

Laboratory should be easily accesible from any unit of the plant and so located as to provide adequate natural lighting (preferably north light) and ventilation. It should be away from pumps and other heavy operating machinery. It should be housed in a separate building close to the administration building.

24.1.1.3 *Floor space*

The minimum floor space required is 25 m². More space is required to accommodate additional equipment necessary to be installed in the room and to avoid interference in the work. The width of walkways between rows of tables or equipments should be not less than 1.0 m preferably 1.2 m. Total floor space requirement of any work room should be arrived at by accounting for space requirements for all equipments and their placement and the number of staff utilising the room.

24.1.1.4 *Walls*

Wall should be finished smooth in bright colours and to be of adequate thickness to provide for built in cabinets. The wall space and offsets should be convenient to locate cabinets benches, hoods, incubators, alongside, without any loss of floor space.

24.1.1.5 *Lighting*

All work rooms in a laboratory including stairways and passages should be well lighted. The window areas in terms of floor area should not be less than 20% and all windows fitted with transparent glass panels. Long windows should be preferred to broad windows for greater depth of penetration of light into work rooms. North-south facing should be preferred, for prevention of glare on work tables and benches. There must be adequate artificial lighting to supplement daylight, well distributed to provide uniform general lighting with minimum shadow effects. Spot lights should be provided for specific equipments and instruments such as weighing balances, hoods, etc. Adequate number of plug points should be provided for extra lighting when required.

24.1.1.6 *Power supply*

Adequate electric power supply for at least 200 amps at L.T. voltage is required. Many laboratory equipments require higher voltage and provision for such exigencies should be made. It is also desirable to provide suitable voltage stabilizers to protect sophisticated equipments from damage due to wide fluctuations in the line voltage. This may require consideration in terms of individual units or for the whole laboratory.

24.1.1.7 *Floor*

Floors should be of smooth finish but not slippery and should be easy to wash and keep clean. Concrete flooring with terrazo finish and dadooing upto window sill level is recommended.

24.1.1.8 *Work tables and benches*

A provision of 10m² space of work tables and benches per worker should be sufficient. These tables should be preferably located along the walls. Tables located in any other position should have a clear gangway of width not less than 1 m between adjacent rows. Wall-side tables are generally kept 60 to 75 cm wide and centre tables are designed 140 cm wide to allow work space on both sides. Height of tables should be 90 to 95 cm for working in a standing posture and 75 to 80 cm for working in a sitting posture. Table tops should be finished smooth and coated with acid resistant black paints or covered with black acid resistant glossy sheets. A separate table of size 120 cm × 60 cm with a low stool should be provided generally for analytical balance. Adequate numbers of high stools are provided along with work tables and benches. Drains connected to table sinks should also be resistant to attack from corrosive substances.

24.1.1.9 *Reagent cabinets and cupboards*

These should be provided in adequate numbers and size for storing chemicals and reagents and stock solutions, etc. in a systematic order. Sliding glass panelled shutters should be preferred to hinged shutters to these cabinets. The laboratory tables could be provided with cupboards and open glass shelves on the top to provide additional space for storage of chemicals and stock solutions.

24.1.1.10 *Sinks*

Both table sinks and separate sinks with adequate water supply shall be provided. Table sinks are fitted with gooseneck taps extending high enough above the table to permit washing of litre cylinders. Separate sinks of sufficient size and depth, located at suitable points shall also be provided for washing the glassware. Plumbing to sinks and wash basins shall be of proper design and of non-corrodible materials like PVC particularly for wastewater lines.

24.1.1.11 *Fume hoods and chambers*

Fume hoods and chambers are necessary to prevent spreading of toxic and irritant fumes and odours into other parts of the laboratory and also to prevent condensation on walls, windows and other fixtures causing corrosion. Some analytical work need isolated fume chambers while others could be carried out under an exhaust hood. Positive ventilation with exhaust fans are generally provided for this purpose. Hoods are designed as per standard practice to provide a minimum air velocity of 30 linear m/min.

24.1.1.12 *Gas supply*

Generally sewage treatment plants are located too far away from the town to avail of the city gas supply even if there is any. The plant should provide its own gas supply to the laboratory by installing a gas plant. Efforts should be made to use digester gas if sludge digesters are installed. Gas should be piped to principal work tables and hoods with appropriate fixture outlets.

24.1.1.13 *Balance room*

The analytical balance mounted on a small table to be used in sitting position shall be provided in a separate cubicle or enclosure in bigger laboratories.

24.1.1.14 *Constant temperature room*

In large plants, provision is sometimes made for constant temperature rooms held at 20°C for performance of BOD tests. If this is not available commercial type 20°C BOD incubator may be used.

24.1.1.15 *Sample preparation room*

In large plants employing both primary and secondary processes where number of samples handled daily is large, a separate sample preparation room is very useful. Such rooms, should have refrigerators of suitable capacities. In addition, an attached cold room with storage facilities may also be necessary particularly where bacteriological work is done.

24.1.1.16 *Media preparation and sterilization rooms*

In large plants where continuous bacteriological analysis is being done, additional facilities for media preparation, centrifuging, sterilization by autoclaves, etc. are necessary and additional rooms for accommodating these facilities should also be included. Such rooms are usually attached to the laboratory and are located within easy reach of the analysts.

24.1.1.17 *Record rooms*

Record rooms for keeping laboratory and plant records should be provided in the laboratory office or in the plant administrative block.

24.1.2 **Equipment and Chemicals**

The types of equipment required for sewage treatment plant laboratory depends on the type of plant, the type of analytical work to be carried out and the frequency of each test to be performed. It is advisable to make initial decisions on the specific analysis to be undertaken, the number of samples, the frequency of sampling and the staff requirement to carry out these analyses, so as to avoid unnecessary purchases and keeping of equipments idle for an indefinite period. Equipments that are not used and are kept idle are often neglected and fall into disuse. Hence selection of equipments for the plant laboratory requires most careful planning, so that each equipment bought is specifically on the basis of anticipated function and operating staff.

A list of important equipments required for carrying out several analytical works in a laboratory is given in Appendix-24. The list is not exhaustive but covers most of the requirements. The quantities required has to be decided as suggested above.

Refrigerators provided for preserving samples should be adequate in capacity and numbers and should maintain a temperature of 0 to 6 °C at a maximum ambient temperature of 40 °C. Normally a 250 litre capacity should suffice for a plant having only primary treatment. Two 250 litre capacity or one large 380 litre capacity should suffice the needs of a medium size plant employing both primary and secondary treatment. A large plant may require three or more of different capacities.

All equipment need a certain amount of maintenance care, particularly those that are electrically operated. Periodic servicing of equipment and checking for their efficiency will save the loss of equipment and prevent faulty analyses leading to wrong interpretations.

Estimates of essential consumable articles such as chemicals, glassware, etc. and recurring replacement in the succeeding years of operation must be worked out with utmost care on the basis of the particular treatment processes for each plant. A list of important tests is givenin Appendix 25, which serves as a guideline for choosing the required glassware and chemicals for a particular plant.

All glassware should be stored in an orderly way and used with care to minimize loss due to handling and breakages. Glassware should be cleaned thoroughly after their use and dried before placing in the cupboards and lockers.

Chemicals should be stored in proper shelves and lockers. Toxic chemicals such as arsenic, cyanide, etc., should be kept under lock and key and should be under the direct charge of a senior analyst who issues and accounts for them. Acids, bulky glassware, etc. which can cause accidents and burns by dropping on the floor should not be stored on high shelves, which need ladders or high stools to reach them.

Chemicals that have a limited life should be bought in such quantities as can be used before their potency is lost.

A stock register for all equipments, chemicals and glassware should be maintained in all laboratories and kept uptodate.

24.2 SAMPLING OF SEWAGE AND WASTE-WATER

Laboratory analyses have little value if representative sampling is not done. Sampling points must be located where homogeneity of the sewage or wastewater with good mixing of the material is available. Careless collection of samples give data which may lead to wrong conclusions.

24.2.1 Methods of Sampling

In all cases of sampling, procedures described in "Manual of Methods for the Examination of Water, Sewage and Industrial Wastes" ICMR should be followed. Care should be taken to avoid entry of extraneous materials such as silt, scum and floating matters into sampling bottles. This is very important while sampling below weirs, channels and directly from tanks.

24.2.1.1 *Grab samples*

Grab samples are collected when frequent changes in character and concentrations are likely to occur and influence the treatment, undesirable constituents are suspected, the quality is not expected to vary or when samples require on-the-spot analysis for parameters such as DO, pH and residual chlorine. Representative samples should be taken with good judgement and should be analysed within 2 to 3 hours of sampling. An enamelled bucket or small pail may be suitable for grab sampling.

24.2.1.2 *Composite or integrated samples*

Composite samples are required for several analyses such as BOD, SS, nitrites, etc., over a period of 12 to 24 hours. The need for the continuous attendance of a person in manual sampling is eliminated in automatic samplers.

24.2.2 Sample Volume

1 to 2 litres of grab sample would be a large enough sample to perform all the tests and repeat some tests if required.

For composite samples, a total quantity of 1 to 2 litres collected over a 24 hour period is adequate. Fractional sample at intervals of 1, 2 or 3 hours should be collected in suitable containers each sample being well mixed and a measured portion proportional to the flow transferred by means of a pipette, measuring cylinder or flask and integrated to form a 1 to 2 litre sample. Hourly records of flows normally available with the Plant Superintendent would facilitate taking of representative sample.

All samples should be immediately transported to the laboratory for analysis. In case there is any delay in transportation, adequate precautions should be taken for fixing the constituents on the spot or preserving the sample in ice.

24.2.3 Selection of Sampling Points

Raw sewage sample should be collected after screens or grit chambers.

Samples of effluent from primary sedimentation or secondary sedimentation tanks should be taken from the effluent trough or pipe or ahead of discharge weirs.

Influent to trickling filter should be collected below the distribution arm and the effluent from the filter from the outlet chamber or at the inlet to secondary tedimentation tank.

A point where there is good mixing should be selected for sampling of mixed liquor in aeration tanks in the activated sludge process.

Influent samples of septic tanks, imhoff tanks, clari-digesters and other sole treatment units such as oxidation ponds, oxidation ditches and aerated lagoons, should be collected ahead of these tanks, in inlet chambers or channels leading to these units. Effluent samples should be collected outside the units in receiving wells or channels or chambers. Sampling within these tanks should be specified in terms of depth or distance or both.

Samples of raw sludge should be taken from sludge sumps or from the delivery side of the sludge pumps through sampling cocks.

Return sludge sample in activated sludge plant is collected at the point of discharge into primary units or aeration tank.

Samples from mixed primary and secondary sludge should be collected at the point of delivery to the digester.

Digested sludge samples may be drawn from the sampling points in the digester or from the discharge end of the delivery pipe leading to drying beds.

Digester supernatant could be drawn from sampling cocks provided for this purpose or through sampling wells on digester dome.

24.3 TESTS PERFORMED IN THE LABORATORY

Routine tests are performed to control the operation of different treatment units. The procedure suggested in "Manual of methods for the examination of water sewage & Industrial Wastes" should be followed.

24.3.1 Raw Sewage

Physical tests consist of temperature, odour, turbidity and colour. Chemical tests consist of alkalinity, suspended and dissolved solids, fixed and volatile solids, BOD, COD, pH, nitrogen and its forms and relative stability (methylene blue test).

24.3.2 Sedimentation Tanks

Influents and effluents are analysed for SS, Settleable solids, BOD and COD to assess the efficiency. Occasionally volumetric efficiency of tanks is tested by using a dye or tracer, if there is any reason to suspect short circuiting in the tanks. Primary sludge from the tank is analysed for per cent solids, organic content and specific gravity if digestion is practised.

24.3.3 Trickling Filters

BOD, COD, total solids, SS, DO, pH, ammonia, nitrite, nitrate and total organic nitrogen are determined to evaluate the performance of trickling filters.

24.3.4 Activated Sludge Aeration Tanks

Influent and effluent BOD and COD, MLSS, DO, ORP SVI and solids retention time or sludge age are determined as a routine. Microscopic analyses are conducted to find out whether sufficient ciliate protozoa and rotifers are present. Special tests are made for microbial growth rate and oxygen uptake rate to assess sludge growth and oxygenation efficiency.

24.3.5 Secondary Settling Tanks

Effluents are analysed for SS, settleable solids filtered and unfiltered BOD, DO, alkalinity, nitrites and nitrates as a routine. Secondary sludges are analysed for percent solids (total, volatile and fixed) and specific gravity when digestion is practised.

In special cases where sewage from T.B. Sanitoria, Infectious Diseases hospitals, etc., are treated, it may be advisable to conduct bacteriological analyses to determined *E. coli* and pathogenic organisms in the treated effluents.

24.3.6. Septic Tanks, Imhoff Tanks and Claridigester

Samples are periodically analysed for settleable solids, total and suspended solids (fixed and volatile), BOD and COD on both influent and effluents. The pH and volatile acids are determined on tank contents. Bottom sludges are periodically examined for percent solids, and organic and fixed nitrogen. Occasionally effluents are examined for pathogens and viable ova of hookworm and ascaris.

24.3.7 Sludge Digester

Characterization of digested sludge is necessary for further disposal and also to determine the sludge balance in the digester and efficiency of digestion. The digested sludge is analysed for temperature, colour, odour, texture, sludge volume, percent solids, volatile solids and alkalinity, volatile acids and pH. Digester content is analysed for pH and volatile acids. Determination for total solids. pH, alkalinity, nitrogen, suspended solids, BOD and COD are made on the supernatant.

24.3.8 Stabilization Ponds

Influent and effluent samples are analysed for SS, BOD, COD, pH, DO and turbidity to determine the efficiency of ponds. The pond samples taken from different points and at different times for 24 hours in a day are analysed for DO, pH, alkalinity and algal cell concentration to assess the condition of the pond for diurnal or seasonal variations. Biological and bacteriological analyses are carried out periodically to study the flora and fauna prevalent in the ponds. pH measurements, COD, BOD and ORP tests are essential guides in the control of anaerobic lagoons, specially where industrial wastes are treated, Colour changes are valuable guide in forecasting of adverse conditions in the pond. Visual inspection of colour or colour determinations are therefore found useful.

24.3.9 Digester Gas

Where digester gas is collected for use in the plant as fuel for development of power, a complete gas analysis should be done frequently to determine carbon dioxide and methane and other contents of the gas like moisture and H_2S.

24.4 RESIDUAL CHLORINE

Where the effluents are chlorinated before discharging into water or land, the residual chlorine test is done regularly to ensure proper chlorination at all times. Usually one sample every 4 hours is sufficient.

24.5 SPECIAL TESTS

Special tests may be required to determine the presence of materials which may create operating difficulties or retard the progress of purification. Such materials include various types of oils and grease, toxic chemicals like copper, cyanide, zinc, chromium, lead and other heavy metals, excess sulphides and ABS.

24.6 ANALYSIS REPORTS

All analyses carried out should be properly recorded, Routine daily analyses, periodic analyses and special analyses should be recorded separately. Copies of these reports should be sent to the plant superintendent immediately after the analysis is done with explanatory notes to indicate any unsatisfactory conditions or abnormalities. The superintendent should study the reports and direct the operating staff for proper corrective measures in the operation schedule. Such measures taken should be reported to the laboratory scientists, who should check the efficiency of corrective measures by resampling and analysis. Corrective measures followed by sampling and analyses should be repeated till such time satisfactory results are obtained.

Data collected over a period of time on various parameters of plant control should be analysed and represented on charts and graphs and displayed in the laboratory for ready reference by the supervisory staff and visitors. These should be included in the weekly, monthly and annual reports of the laboratory.

24.7 PERSONNEL

Laboratories of large plants should be under the charge of a qualified and experienced Analyst supported by junior technical staff having background in the fields of chemistry, biology and bacteriology. The analyst should assimilate the details of functioning of the plants by experience and acquire the necessary preparedness for receiving further specialised training including performance interpretation and application of advanced techniques which enable him to participate in the efficient operation of the treatment unit.

In the case of small plants the laboratory may be under the charge of a person having some training in analysis of sewage.

APPENDICES

APPENDIX 1

ABBREVIATIONS AND SYMBOLS

AC	asbestos cement
amp	ampere
AS	activated sludge
BOD	biochemical oxygen demand
BOD_5	5 days biochemical oxygen demand
cc	cubic centimetre
CI	cast iron
cm	centimetre
CM	cement mortar
COD	chemical oxygen demand
cumec	cubic metre per second
°C	degrees centigrade
d	day
DO	dissolved oxygen
Eq or Eqn	equation
FAR	floor area ratio
FSI	floor space index
F/M	food to micro organisms ratio
gm	gramme
GI	galvanised iron
ha	hectare
h, hr	hour
HRT	hydraulic retention time
HSV	hydraulic subsidence value
IS	Indian standard
kcal	kilo calorie
kg	kilogram
kgf	kilogram force
kL	kilo litre
kLd	kilo litre per day
km	kilometre
kw	kilowatt
kwh	kilowatt hour
l	litre
lpcd	litre per capita per day
lpd	litre per day
lpm	litre per minute
lps	litre per second
m	metre
mps	metre per second
m^2	square metre
m^3	cubic metre
meq	milliequivalent
min	minute
ml	millilitre
mL	million litres
mLd	million litres per day
MLSS	mixed liquor suspended solids
MLVSS	mixed liquor volatile suspended solids
mm	millimetre
MWL	maximum water level
NPSH	net positive suction head
ORP	oxidation reduction potential
Pr.	Primary
PSS	percent soluble sodium
PVC	polyvinyl chloride
RCC	reinforced cement concrete
rph	revolution per hour
rpm	revolution per minute
SAR	sodium absorption ratio
SS	suspended solids
S.S.T.	secondary sedimentation tank
SVI	sludge volume index
T	tonnes
TF	trickling filter
VS	volatile solids
VSS	volatile suspended solids

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APPENDIX 2

CONVERSION FACTORS

Length

1 in	=	25·4	mm
1 ft	=	0·3048	m
1 yd	=	0·9144	m
1 mile	=	1·6093	km
1 mm	=	0.0394	in
1 cm	=	0·3934	in
	=	0·0328	ft
1 m	=	3·2808	ft
1 km	=	1·0936	yd
1 km	=	0·6214	mile

Area

1 sq in	=	645·163	sq mm
	=	6·4516	sq cm
1 sq ft	=	0·0929	sq m
1 sq yd	=	0·8361	sq m
1 sq mile	=	2·59	sq km
1 acre	=	0·4047	ha
	=	4046·86	sq m
1 sq mm	=	0·00155	sq in
1 sq cm	=	0·1550	sq in
	=	0·00108	sq ft
1 sq m	=	10·7639	sq ft
	=	1·1960	sq yd
1 ha	=	2·4710	acre
	=	0·00386	sq mile
1 sq km	=	0·3861	sq mile
	=	247·105	acre

Capacity

1 gal (UK)	=	4·54609	l
	=	0·00454609	cu m
	=	0·160544	cu ft
1 gal (US)	=	0·00378541	cu m
	=	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
1 l	=	0·0353147	cu ft
	=	0·001308	cu yd
	=	0·2200	gal (UK)
	=	0·264172	gal (US)

Volume

1 cu in	=	16·8871	cu cm
1 cu ft	=	0·0283	cu m
1 cu yd	=	0·7646	cu m
1 acre ft	=	1233·48	cu m
1 cu cm	=	0·061024	cu in
1 cu m	=	35·815	cu ft
	=	1·60795	cu yd
	=	0·00081071	acre ft

Weight

1 grain	=	0·0648	g
1 oz	=	28·3495	g
1 lb	=	0·4536	kg
1 ton	=	1·01605	tonnes
1 g	=	15·45254	grains
	=	0·0352740	oz
1 kg	=	2·20462	lb
1 tonne	=	0·98421	ton

Density

1 lb/ft^3	=	16·0185	kg/m^3 or g/l
1 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 ton/in^2	=	1·5749	kg/mm^2
1 atm	=	101325·0	N/m^2
	=	760·0	mm Hg
	=	1·01325	bar
	=	14·6959	lbf/in^2
	=	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 kg/cm^2	=	14·223	lb/in^2
	=	10	m H_2O
	=	0·96784	atm
1 kg/m^2	=	0·204816	lb/ft^2
1 kg/mm^2	=	0·6850	ton/in^2
1 atm	=	68087·0	pdl/ft^2 (where 1 pdl = 0·138255 N)
1 mm Hg	=	2.78450	lb/ft^2

APPENDIX 2—*contd.*

Force

1 lbf	=	4·44822	N
	=	0·453592	kgf
1 tonf	=	9·96402	kN
1 pdl	=	0·138255	N
g (acceleration due to gravity)	=	32·1740	ft/sec²
	=	980·665	cm/sec²
1 N (or 10^5 dynes)	=	0·101972	kgf
	=	0·224809	lbf
1 kgf	=	2·20462	lbf

Energy and Power

1 horse-power	=	0·745700	kw
1 ft. lb f/s	=	1·35582	w
1 B.t.u.	=	1·05506	kJ
1 therm	=	105·506	MJ
1 ft. lbf	=	1·35582	J
1 kw	=	1·34102	Horse-power
1 kwh	=	3·6	MJ
1 J	=	0·737562	ft lbf
1 kJ	=	0·277778	wh

Velocity

1 fps	=	0·0348	m/s
	=	1·0973	km/h
1 mile/h	=	0·4470	m/s
	=	1·6093	km/h
1 m/s	=	3·2808	fps
	=	2·2369	mile/h
1 km/h	=	0·9113	fps
	=	0·6214	mile/h

Treatment Loading Rates

1 in/h	=	0·00705555	mms
1 UK gal/ft²h	=	0·0135927	mm/s
1 UK gal/ft²/h	=	1·17441	m³/m²/d
1 million UK gal/acre/d	=	0·0130016	mm/s
	=	1·12336	m³/m²/d
1 UK gal/day/ft	=	14·915	lpd/m
	=	0·014915	m³/day/m
1 mm/s	=	141·732	in/h
	=	73·5689	UK gal/ft²/h
	=	76·9130	million UK gal/acre/d.
1 m³/m²/d	=	0·851491	UK gal/ft²/h
	=	0·890187	million UK gal/acre/d.
1 m³/day/m	=	67·466	UK gal/day/ft

Run-off

1 ft³/s/1,000 acre	=	6·99724	l/s/km²
1 ft³/s/mile²	=	10·9332	l/s/km²
1 l/s/km²	=	0·142915	ft³/s/1,000 acres
	=	0·0914645	ft³/s/mile

Hardness

mg/l CaCO₃	Grains per UK gal CaCO₃ (Clark scale—British degrees)	Grains per US gal CaCO₃ (American degrees)
1·00	0·07	0·058
14·29	1·00	0·83
17·15	1·20	1·00
10·00	0·70	0·58
17·86	1·25	1·04
2·57	0·18	0·15

Parts per 100,000 $CaCO_3$ (French degrees)	Parts per 100,000 CaO (German degrees)	Parts per million Ca (Russian degrees)
0·10	0·056	0·40
1·43	0·080	5·72
1·72	0·96	8·86
1·00	0·56	4·00
1·79	1·00	7·14
0·26	0·14	1·03

APPENDIX 3—*Contd.*

Sl. No.	Indian Standard No.	Title
		Miscellaneous
59	460—1962 . . .	Test sieves.
60	771—1963 . . .	Glazed earthenware sanitary appliances (revised.)
61	772—1973 . . .	General requirements of enamelled C.I. sanitary appliances (second revision).
62	773—1954 . . .	Enamelled C.I. water closets, railway coach stocks type (revised).
63	775—1970 . . .	Brackets and supports for lavatory basin and sinks (second revision).
64	776—1962 . . .	Wooden water closet, seats and covers.
65	777—1970 . . .	Glazed earthenware tiles (first revision).
66	1200— . . .	Methods of measurement of buildings in Civil Engineering Works.
	(a) Part XVI —1959	Laying of water and sewer lines, including appurtenant items.
	(b) Part XIX —1970	Water Supply, plumbing, drains and sanitary fittings.
67	1445—1970 . . .	Mild steel dust bins.
68	1607—1960 . . .	Methods for dry sieving.
69	1701—1960 . . .	Mixing valves for ablutionary and domestic purposes.
70	2174—1962 . . .	Reinforceed concrete dust bins.
71	2431—1963 . . .	Steel wheel barrows (single wheel type).
72	2548—1967 . . .	Plastic water-closet seats and covers (second revision).
73	2556— . . .	Vitreous sanitary appliances (vitreous china)
	(a) Part I —1967 .	General requirements (first revision).
	(b) Part II —1973 .	Specific requirements of wash-down water closet (second revision).
	(c) Part III—1973 .	Specific requirements of squatting pans (second revision).
	(d) Part IV—1972 .	Specific requirements of wash basins (second revision).
	(e) Part V—1967 .	Specific requirements of laboratory sinks (first revision).
	(f) Part VI—1967 .	Specific requirements of urinals (first revision).
	(g) Part VII—1973	Specific requirements of half round channels (second revision).
	(h) Part VIII—1973	Specific requirements of siphonic washdown water closets (second revision).
	(j) Part IX —1972 .	Specific requirements of bidets (second revision).
	(k) Part X —1967 .	Specific requirements of foot rests (first revision).
	(l) Part XI—1972 .	Specific requirements for shower rose.
	(m) Part XII —1973	Specific requirements for floor traps.
	(n) Part XIII —1973	Specific requirements of traps for squatting pans.
	(p) Part XIV —1974	Specific requirements of integrated squatting pans.
	(q) Part XV —1974	Specific requirements of universal water closets.
74	2963—1964 . . .	Non-ferrous waste fittings for wash-basins and sinks.
75	3311—1965 . . .	Waste plug and accessories for sinks and wash basins.
76	3489—1966 . . .	Enamelled steel bath tubs.
77	3861—1966 . . .	Methods of measurements of area and cubical contents in buildings.
78	5219— . . .	Cast copper alloys traps.
	(a) Part I —1969 .	'P' and 'S' traps.
79	5421—1969 . . .	Glossary of terms relating to test sieves and test sieving.
80	5611—1970 . . .	Code of practice for waste stabilization ponds (facultative type).
81	5742— . . .	Terms and symbols for sieve bottoms.
	(a) Part I —1970 .	Woven and welded wire screens.
	(b) Part II —1970 .	Perforated plates.
82	5961—1970 . . .	Cast iron gratings for drainage purposes.
83	6279—1971 . . .	Equipment or grit removal devices.
84	6411—1972 . . .	Get coated glass fibre reinforced.
85	7331—1972 . . .	Code of practice for inspection and maintenance of cross drainage works.
		Measurements of fluid flow
86	1191—1971 . . .	Glossary of terms and symbols used in connection with the measurement of liquid flow with a free surface (first revision).
87	1192—1959 . . .	Velocity area methods for measurement of flow of water in open channels.
88	1193—1959 . . .	Methods of measurements of flow of water in open channels, using notches, weirs and flumes.
89	1194—1960 . . .	Forms for recording measurement of flow of water in open channels.
90	2912—1964 . . .	Recommendations for liquid flow measurement in open channel by slope area method (approximate method).

APPENDIX 3—*Contd.*

Sl. No.	Indian Standard No.	Title
91	2913—1964 . . .	Recommendations for determination of flow in tidal channels.
92	2914—1964 . . .	Recommendations for estimation of discharge by establishing stage discharge relation in open channels.
93	2915—1964 . . .	Instructions for collection of data for the determination of errors in measurement of flow by velocity area method.
94	2951— . . .	Recommendations for estimation of flow of liquids in closed conduits.
	(a) Part I —1965 .	Head loss in straight pipes due to frictional resistance.
	(b) Part II—1965 .	Head loss in valves and fittings.
95	2952—	Measurement of fluid flow by means of orifice plates and nozzles.
	(a) Part I —1964 .	Incompressible fluids.
96	3910—1966 . . .	Cup type current meter.
97	3911—1966 . . .	Surface floats.
98	3912—1966 . . .	Sounding rods.
99	4477—	Methods of measurements of fluid flow by venturi-meters.
	(a) Part I —1967 .	Liquids.
100	4858—1968 . . .	Velocity rods.
101	6059—1971 . . .	Recommendations for liquid flow measurements in open channels by weirs and flumes—weirs of finite crest width for free discharge.
102	6062—1971 . . .	Methods of measurements of flow of water in open channels using standing wave flume full.
103	6063—1971 . . .	Methods of measurement of flow of water in open channels using standing wave flume.
104	6330—1971 . . .	Recommendations for liquid flow measurement in open channel by weirs and flume-end—method for estimation of flow in rectangular channels with a free overfall (approximate method).
		Industrial Effluent
105	2296—1963 . . .	Tolerance limits for inland surface water subject to pollution.
106	2488—	Methods of sampling and test
	(a) Part I —1968 .	for industrial effluent, Part I
	(b) Part II —1968 .	for industrial effluent, Part II
	(c) Part III—1968 .	for industrial effluent, Part III
107	2490—1963 . . .	Tolerence limits for industrial effluents discharged into inland surface waters.
108	3306—1974 . . .	Tolerance limits for industrial effluents discharged into inland public sewers (first revision)
109	3307—1965 . . .	Tolerance limits for industrial effluents discharged on land for irrigation purposes.
110	4733—1968 . . .	Methods of sampling and test for sewage effluents.
111	4764—1973 . . .	Tolerance limits for sewage effluents discharged into inland surface waters (first revision)

APPENDIX 4

Computation of storm runoff and design of storm sewers

Design a system of storm sewers for the area shown in the figure No. A.4-1 based on the Rational Formula for the estimation of peak runoff.

Basic Data and Assumptions

Imperviousness

Built up and paved area—0·7

Open space, lawns, etc.—0·2

Inlet time

Built up and paved area (t_b) — 8 minutes

Open space, lawns (t_l) —15 minutes

Minimum velocity in sewer —0·8 mps

Minimum depth of cover above crown —0·5 metres

Rainfall intensity=consider one year storm as the area is central and high priced.

(Use Table 3·1 for the record of rainfall intensity and frequency of rainfall).

Use Manning's chart for Sewer design

Solution

Quantity of storm water runoff is calculated using the Rational Formula given in Sec. 3.2.1.1.

i.e. Q=10 c.i.A

Where

Q is the runoff in m³/hr

c is the coefficient of runoff

i is the intensity of rainfall in mm/hr and

A is the area of drainage district in hectares.

Storm water runoff is determined in the following manner;

(i) From the rainfall records for the last 26 years (table 3·1), the storm occurring once in a year, i.e. 26 times in 26 years, the time—intensity values for this frequency are obtained by interpolation and are as follows :

Intensity, 'i' mm/hr.	30	35	40	45	50	60
Duration, 't' minute	44	36	28·5	22·5	13·5	9·75

(ii) The generalised formula adopted for intensity and duration is

$$i = \frac{a}{t^n}$$

where i =intensity of rainfall in mm/hr
t =duration in minutes and
a and n are constants.

A graph (Fig. A-4.2) is plotted for one year storm using the values of 'i' and 't' from the above table on a log-log paper. From the line of best fit the values of 'a' and 'n' are found out. From the plotted line, values of 'a' and 'n' are 160 and 0·4 respectively.

(iii) Now using equation $i = \frac{160}{t^{0.4}}$, i.e. after substituting the values of 'a' and 'n', different values of 'i' for various values of t are calculated and tabulated as below and a curve (Fig. A-4.2) is plotted on an ordinary graph paper.

Table for intensity-duration curve for one year storm

t min	5	10	15	20	25	30	35	40	45	60	80	100	120
$i = a/t^n$	84.2	64.0	54.0	48.5	44.2	41.2	38.6	36.8	34.8	31.0	27.8	25.4	23.6

(iv) Another graph (Fig. 4.3 (a)) of runoff-coefficient 'c' vs. duration time 't' is plotted as per values given in Table 3·2 (Horner's Table).

(v) From the above two graphs (Fig. A-4.3(a) and (b)) the values of c and i for the same duration time t are determined and the curves for 10 ci vs t for the various values of imperviousness are plotted (Fig. A-4.4). The value of 10ci gives the rate of runoff in m³/hr per hectare of the tributary area. These curves are ultimately used in calculating the runoff from the tributary areas for a given time of concentration and imperviousness factor.

Design of storm sewer system

Table A-4.1 gives the various components of the storm sewer system design.

Column 1-4 identify the location of drain, street and manholes.

Column 5-6 record the increment in tributary area with the given imperviousness factors.

Column 7 gives the tributary area increment with equivalent 100 percent imperviousness factor.

Column 8 records the total area served by each drain.

Column 9 records the time of concentration at each upper end of line (drain). The time of concentration is found by taking the weighted average of the two areas.

(i.e. $t_c = \frac{A_1 . t_b + A_2 . t_l}{A_1 + A_2}$, where A_1= built up area and A_2= Area of lawns).

Column 10 records the time of flow in each drain. For example the time of flow in line 1 is calculated to be 70/(60×1·0)=1·17 min.

Column 11 is the total time of concentration for each drain.

Column 12 is the value of runoff as 10 ci read from the Fig. A-4.4 for the corresponding time of concentration.

Column 13 gives the total runoff from each tributary area.

Column 14 gives the runoff in lps from each tributary area.

Col. 15-18 record the chosen size, required grade resulting capacity, velocity of flow for each drain or line. These designs of storm sewers are computed from the Manning's chart for each required flow and maintaining a minimum velocity.

Col. 19-23 identify the profile of the drain.

Col. 19 is taken from the plan.

Col. 20=col. 19× col. 16.

Col. 21—the required drop in manholes is obtained directly from the recommended values in section 3.3.9.

Col. 22—gives invert elevation at the upper end with a minimum cover of 0·6 m at starting manholes. Thus for lines 1, 3, 6 and 9, the invert elevations are respectively 37.400, 36.700, 38.000 and 36.800. In case of a manhole having more than one inlet the drop in the manhole is considered with respect to the lowest invert level of the inlets to fix the invert level of the outlet.

Col. 23=Col. 22—Col. 20=invert elevation at the lower end of the line.

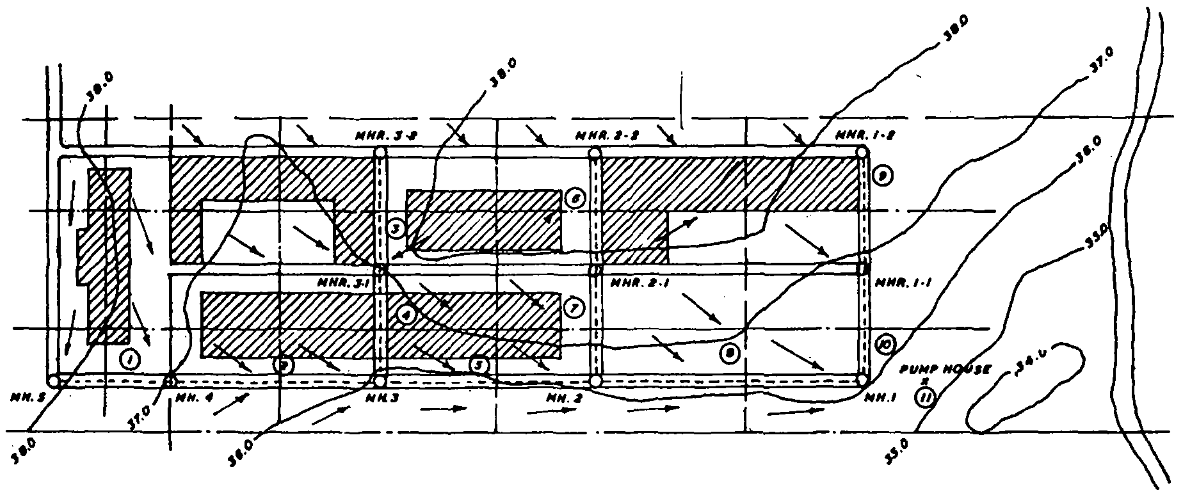

FIG. A - 4.1

SCALE :- 1 : 2500

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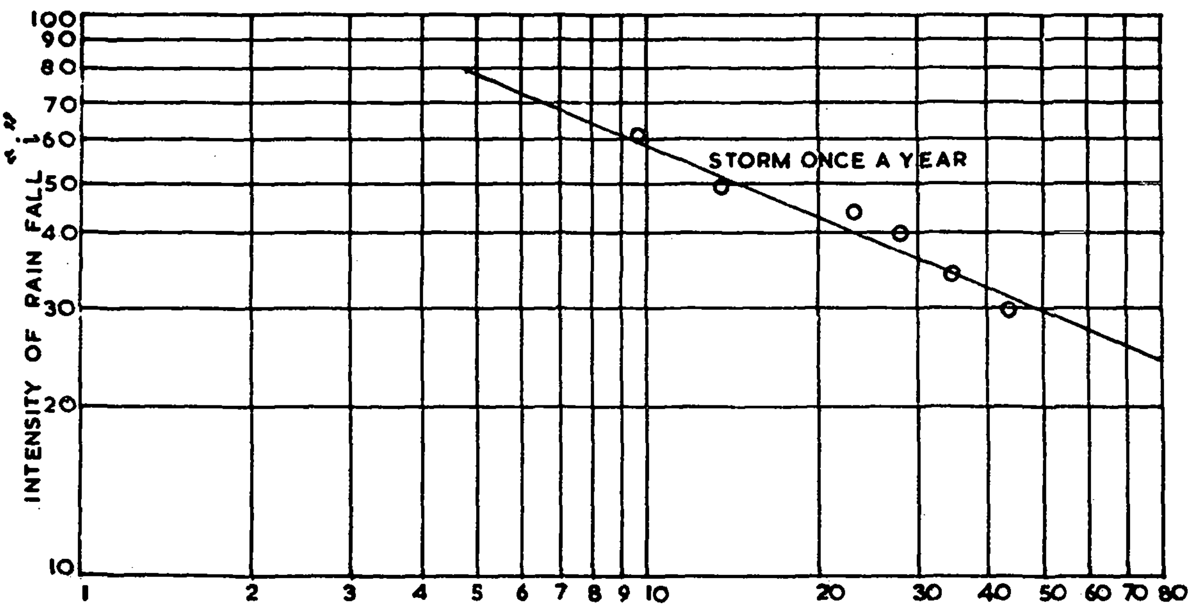
INTENSITY 'i' mm/hr	30	35	40	45	50	60
DURATION 't' (MINUTES)	44	36	28.5	22.5	13.5	9.75



DURATION IN MINUTES 't'

FIGURE: A-4.2

FORMULA ADOPTED $i = \frac{a}{t^n}$

FROM THE ABOVE GRAPH:

$$\left\{\begin{array}{l} t-10,\ i-50 \\ t-20,\ i-44 \end{array}\right\} n = 0.4, a = 160$$

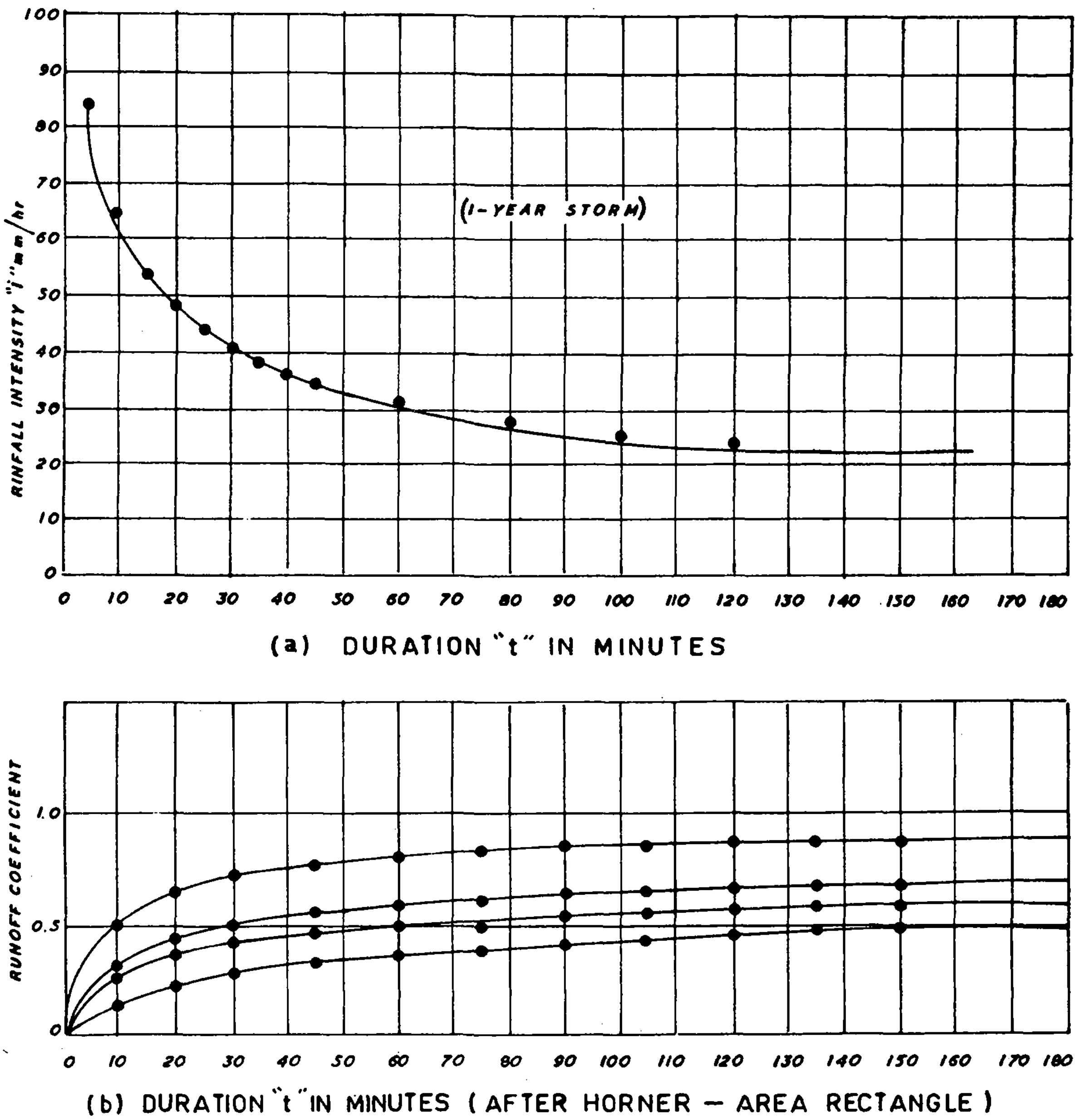


FIG. A-4·3

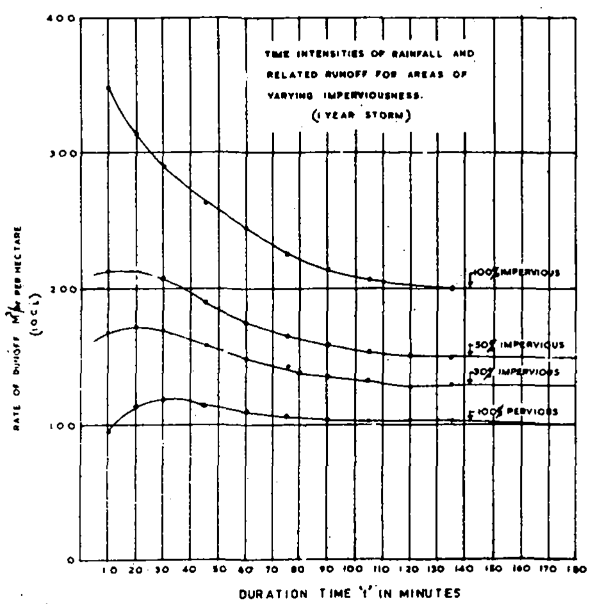


FIGURE No. A 4·4

TABLE A-4—1

Design of a storm-sewer system

Line number	Location of drain: Street	Location of drain: Manhole from	Location of drain: Manhole to	Tributary area 'a' (hectares): Increment: 0·7 Imp. factor	Increment: 0·2 Imp factor	Increment: Eq. 100% Imp. factor	Total area
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	South St. . .	5	4	0·286	0·366	0·274	0·274
2	..	4	3	0·167	0·488	0·214	0·488
3	North-South St. 2 .	R.3·2	R.3·1	0·415	0·312	0·352	0·352
4	..	R.3·1	3	0·358	0·366	0·324	0·676
5	South St. . .	3	2	0·256	0·466	0·274	1·438
6	North-South St. 3 .	R.2·2	R.2·1	0·230	0·492	0·260	0·260
7	..	R.2·1	2	0·410	0·310	0·348	0·608
8	South St. . .	R.2	1	0·256	0·466	0·274	2·320
9	North South St. 4 .	R.1·2	R.1·1	0·660	0·282	0·517	0·517
10	..	R.1·1	1	0·580	0·362	0·479	0·996
11	South St. . .	1	pump-house	0·610	0·330	0·494	3·810

t_c Time of concentration: Time of inlet to upper end t_i	Time of flow in drain t_f	Total $t_c = t_i + t_f$	Runoff m³/hr: Per hectare (10 ci)	Runoff m³/hr: Total
(9)	(10)	(11)	(12)	(13)
12.0	..	12·0	345	94·5
13·3	1·17	14·47	335	164·0
11·0	..	11·0	348	123·0
11·5	1·17	12·67	340	264·0
12·5	3·27	15·77	335	480·0
12·8	..	12·8	340	87·5
11·0	1·17	12·17	342	208·0
12·5	5·37	17·87	330	765·0
10·2	..	10·2	350	182·0
10·8	0·94	11·74	344	330·0
10·4	8·05	18·45	325	1240·0

Flow 'Q' lps	Design: Diameter mm	Design: Slope m/1000	Design: Capacity lps	Design: Velocity mps	Profile: Length m	Profile: Fall m	Profile: Drop in manhole m	Invert Elevation: Upper end	Invert Elevation: Lower end
(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)
26·6	200	10·0	32	1·0	70	0·70	0·000	37·400	36·700
46·0	250	6·65	50	1·0	125	0·83	0·025	36·675	35·845
35·0	250	6·65	50	1·0	70	0·47	0·000	36·700	36·230
74·0	350	4·55	98	1·0	70	0·32	0·050	36·180	35·860
135·0	450	3·14	160	1·0	125	0·40	0·066	35·779	35·379
25·0	200	10·0	32	1·0	70	0·70	0·000	38·000	37·300
59·0	300	5·55	70	1·0	70	0·39	0·050	37·250	36·860
214·0	600	2·22	280	1·0	160	0·36	0·200	35·179	34·819
51·0	250	10·0	60	1·25	70	0·70	0·000	36·800	36·100
92·0	350	5·0	100	1·1	70	0·35	0·050	36·050	35·700
345·0	700	1·67	400	1·0	25	0·42	0·234	34·585	34·165

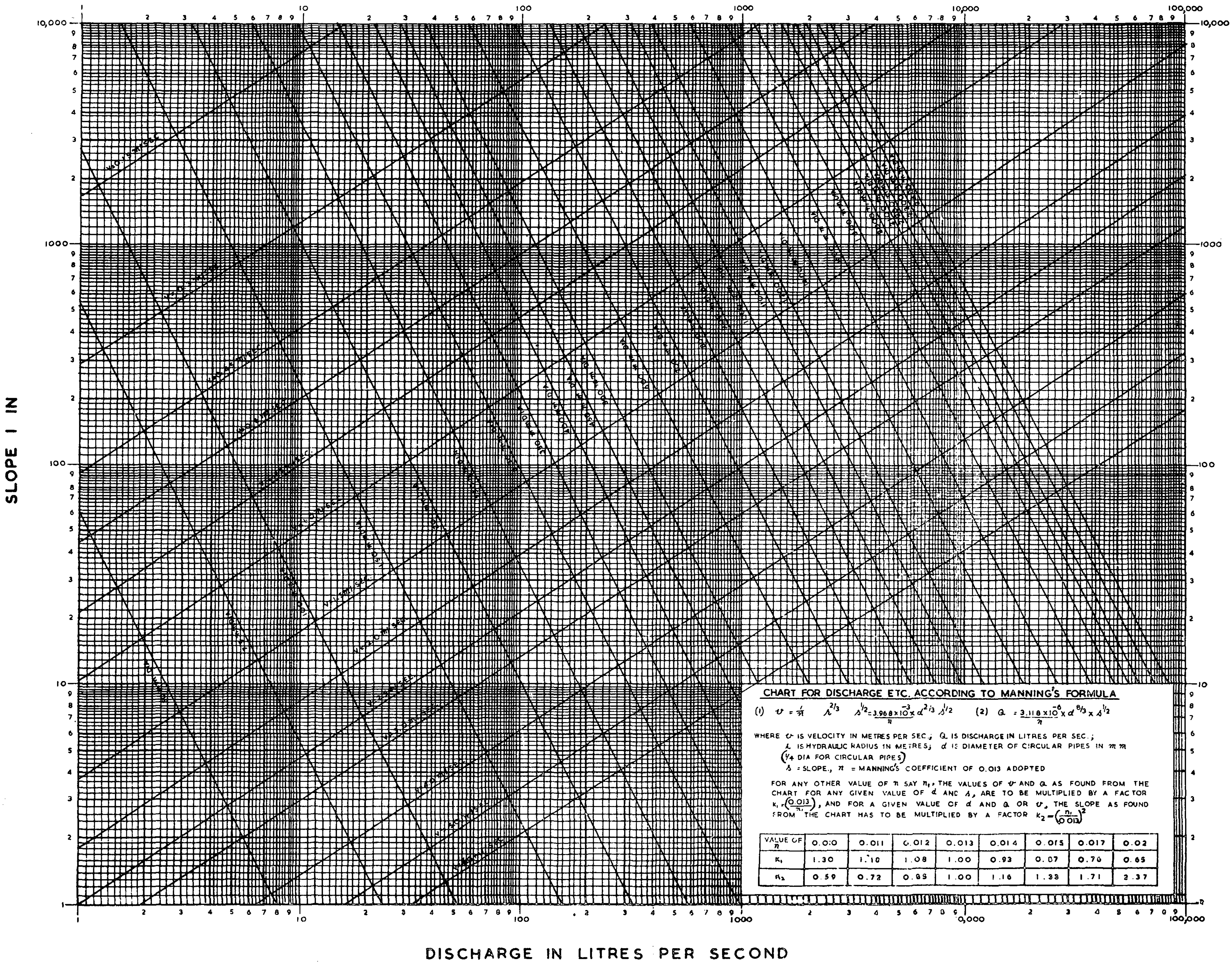


CHART FOR DISCHARGE ETC. ACCORDING TO MANNING'S FORMULA

(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) $Q = \frac{3.118 \times 10^{-6}}{n} \times d^{8/3} \times s^{1/2}$

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 PIPES 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 ANY GIVEN VALUE OF d AND s, ARE TO BE MULTIPLIED BY A FACTOR $k_1 = \left(\frac{0.013}{n_1}\right)$, AND FOR A GIVEN VALUE OF d AND Q OR v, THE SLOPE AS FOUND FROM THE CHART HAS TO BE MULTIPLIED BY A FACTOR $k_2 = \left(\frac{n_1}{0.013}\right)^2$

VALUE OF n	0.010	0.011	0.012	0.013	0.014	0.015	0.017	0.02
K_1	1.30	1.10	1.08	1.00	0.93	0.07	0.76	0.65
K_2	0.59	0.72	0.85	1.00	1.16	1.33	1.71	2.37

APPENDIX 6

Design of sanitary sewer system

Problem

Design a system of sanitary sewers for the given area shown in the figure A-6.1 with the following details :

1. Population Density . . . 300 persons/hect.
2. Water Supply . . . 250 lpd/head. (ultimate).
3. Maximum rate of infiltration . 20,000 lpd/hect.
4. Minimum depth of cover to be provided over the crown of the sewer. 1 m
5. Minimum velocity in sewer at peak flow. 0·6 mps.
6. Maximum velocity in sewer . 2·0 mps.
7. Minimum size of the sewer . 150 mm
8. Waste water reaching sewers . 90% of W/S
9. Peak flow . . . . 3·5 × Ave flow

Solution

1. Draw a line to represent the proposed sewer in each street or alley to be served. Near the line indicate by an arrow the direction in which sewage is to flow.
2. Locate the manhole, giving each an identification number.
3. Sketch the limits of the service areas for each lateral.
4. Measure the areas (ha) of the several service areas.
5. Prepare a table as shown in Table A-6.1 with the columns for the different steps in the computation and a line for each section of sewer between manholes.

Column 1 to Column 6 . for the line manhole, location of the manhole, manhole numbers ground level at starting manhole and the length of line between the manholes

Column 7—Column 8 . the corresponding area for the next street of sewer and in Col. 8 the sum of the areas are entered.

Column 9 . . . the population served by each corresponding line is entered.

Column 10 . . . shows the sewage flow (mLd) through each line. The sewage flow is assumed as 90% of the per capita water supply.
(Column 8 × 250 × 0·9)

Column 11 . . . shows the ground water infiltration for each area.
$=20,000 \times 10^{-6} \times$ Col. 8

Column 12 . . . gives the peak flow i.e. Col. 10 × 3 + Col. 11.

Column 13 . . . gives the peak flow in lps.

Column 14—15 . . indicate the diameter, and slope of the pipes determined from the Mannings' Chart.

Column 16—17 . . indicate the discharge through pipe flowing full and the actual discharge through the pipes i.e. as Col. 13.

Column 18 . . . also determined from the Manning's Chart when pipe flowing full.

Column 19 . . . calculated from the hydraulic elements curve for the circular pipes.

Column 20 . . . Col. 6 × Col. 15.

Column 21—22 . . invert levels of the lines are calculated.

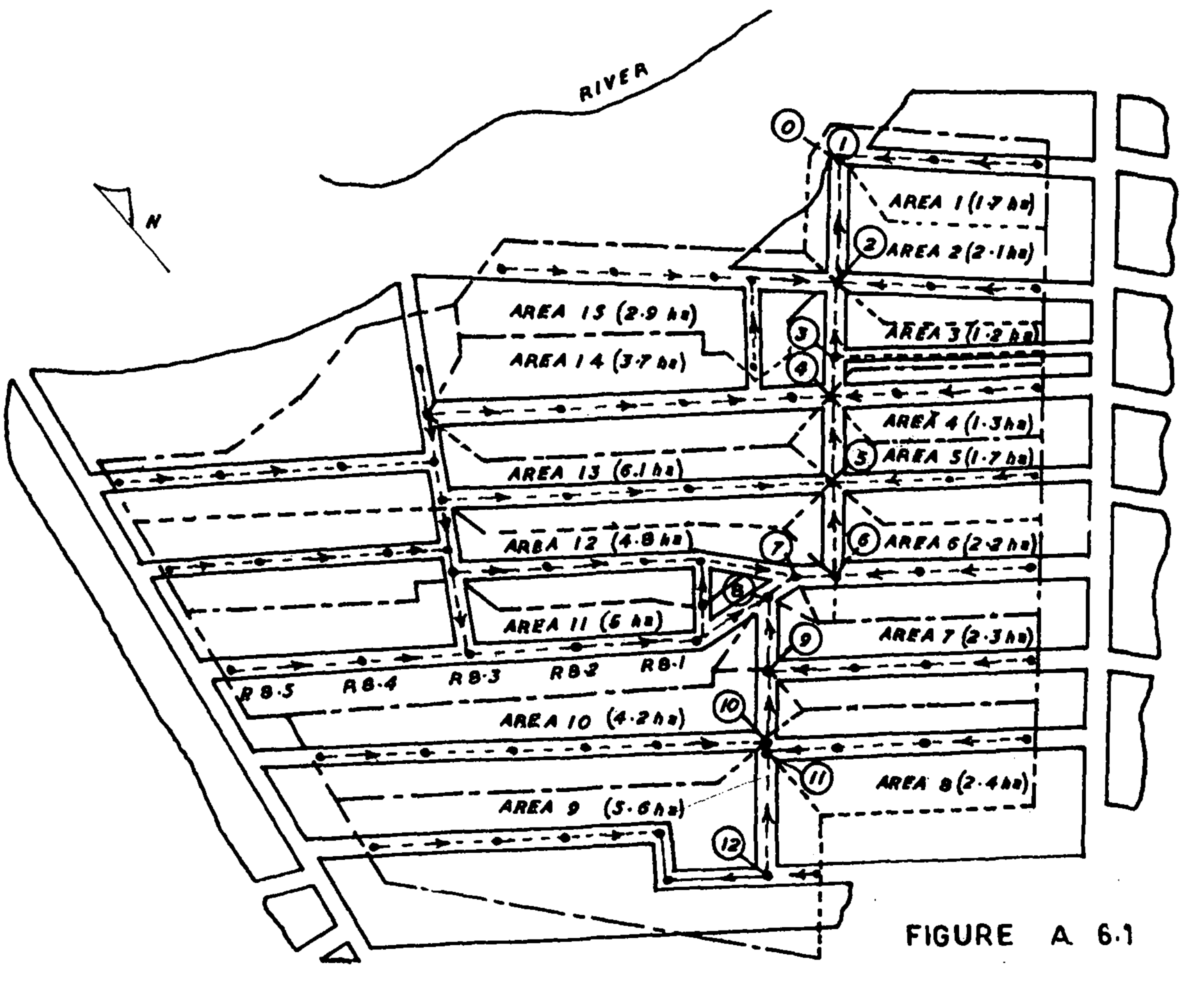


FIGURE A 6.1

TABLE A—6.1

Design of a Sewer System

Line	Location	Manhole From	Manhole To	Ground level at starting manhole	Length 'm'	Area served (ha) Increment	Area served (ha) Total	Population	Sewage flow 'mLd'	Ground water infiltration mLd	Peak flow mLd	Peak flow lps
1	2	3	4	5	6	7	8	9	10	11	12	13
1	Street. . . .	R.8·5	R.8·4	38·275	120	0·80	0·80	240	0·054	0·016	0·205	2·37
2	„	R.8·4	R.8·3	37·960	116	1·20	2·00	600	0·135	0·040	0·512	5·92
3	„	R.8·3	R.8·2	36·873	114	1·40	3·40	1020	0·230	0·068	0·873	10·10
4	„	R.8·2	R.8·1	36·895	116	0·90	4·30	1290	0·290	0·066	1·10	12·73
5	„	R.8·1	8	36·420	75	0·70	5·0	1500	0·34	0·10	1·29	14·92
6	„	8	7	36·117	41	14·5	19·5	5850	1·32	0·39	5·01	57·96
7	„	7	6	35·830	26	4·8	24·3	7300	1·64	0·48	6·22	71·96
8	Main Street .	6	5	35·105	88	2·2	26·5	7950	1·80	0·53	6·83	79·02
9	„ . .	5	4	34·412	86	7·8	34·3	10300	2·31	0·68	8·76	101·35
10	„ . .	4	3	34·181	36	5·0	39·3	11800	2·65	0·70	10·05	116·28
11	„ . .	3	2	34·105	77	1·2	40·5	12150	2·73	0·80	10·35	119·75
12	„ . .	2	1	33·905	117	5·0	45·5	13650	3·07	0·91	11·65	134·79
13	„ . .	1	0	33·250	41	1·7	47·2	14200	3·2	0·94	12·14	140·46

Diameter mm	Slope	Discharge lps Q Full	Discharge lps Q Actual	Velocity mps V Full	Velocity mps v Actual	Total fall 'm'	Invert Elevation 'm' Upper end	Invert Elevation 'm' Lower end
14	15	16	17	18	19	20	21	22
150	·008	14	2·37	0·75	0·57*	0·96	37·125	36·165
150	·008	14	5·92	0·75	0·72	0·93	36·135**	35·205
150	·008	14	10·10	0·75	0·82	0·91	35·175	34·265
150	·008	14	12·73	0·75	0·86	0·93	34·235	33·305
200	·005	24	14·92	0·70	0·74	0·38	34·275	33·895
300	·005	70	57·96	1·0	1·13	0·21	33·845	33·635
350	·005	100	71·96	1·2	1·32	0·13	33·605	33·475
350	·005	100	79·02	1·2	1·32	0·44	33·445	33·005
400	·0033	125	101·35	1·0	1·12	0·29	32·975	32·685
400	·0033	125	116·28	1·0	1·14	0·12	32·655	32·535
400	·0033	125	119·75	1·0	1·14	0·26	32·505	32·245
450	·0033	160	134·79	1·0	1·12	0·39	32·208	31·818
450	·0033	160	140·46	1·0	1·12	0·14	31·788	31·648

*Since VEL is less than 0·6 mps, flushing once a day is necessary.

**A minimum level difference of 30 mm has been provided between the incoming and outgoing sewers to provide necessary slope in the manhole.

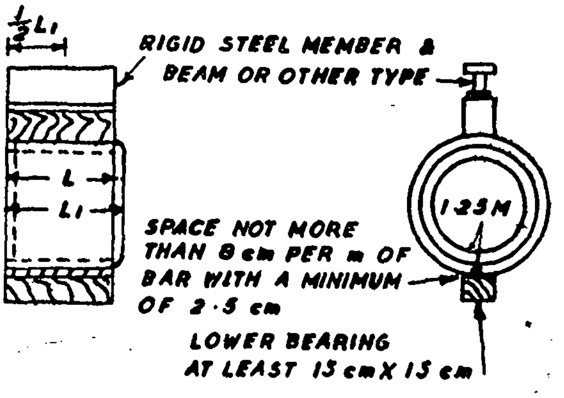
APPENDIX 7

Three Edge Bearing Tests for Pipe Strength

The load which the pipe must withstand without failure is termed three-edge bearing strength. For unreinforced concrete pipes, the point of load at which the pipe cracks and fails is the termination of a 3-edge bearing test.

For reinforced concrete pipes, these specifications provide two criteria for passing the 3-edge bearing test: first, there is an intermediate load based on the appearance of a crack 0.25 mm wide and 0.3 m long. The final requirement for reinforced pipe is the ultimate three edge bearing strength at the final failure of the pipe where no further load increase can be supported.

In conducting this test, the pipe is placed horizontally on two parallel wooden rails resting on 15cm × 15cm bearing block or other solid support that extends the length of the pipe. An upper bearing block is placed on the top of the pipe. Next, a rigid I-beam or other structural member is placed on the upper bearing block to apply the load to the block.



THREE EDGE BEARING TEST

FIG. A-7·1

Three edge bearing strengths of prestressed, unreinforced concrete pipes are given in table A—7.1.

TABLE A—7.1

Dia of pipe mm	Load to produce 0·25 mm crack (kg/linear meter)				Ultimate load (kg/linear meter)				
	Pre-stressed concrete	Concrete			Pre-stressed concrete	Concrete			
		NP_2, P_1, P_2 & P_3	NP_3	NP_4		NP_1	NP_2, P_1, P_2 & P_3	NP_3	NP_4
80	1000	1040	—	—	2300	—	1560	—	—
100	1000	1040	—	—	2300	1560	1560	—	—
125	1000	—	—	—	2300	—	—	—	—
150	1000	1040	—	—	2300	1560	1560	—	—
200	—	—	—	—	—	—	—	—	—
250	1100	1140	—	—	2500	1670	1710	—	—
300	1200	1200	—	—	2700	1790	1800	—	—
350	1300	1260	3040	—	2900	1880	1890	4560	—
400	1400	1360	3460	3460	3100	2020	2040	5190	5190
450	1500	1480	3760	—	3400	2230	2220	5640	—
500	1700	1660	4160	4160	3700	—	2490	6240	6240
600	1900	1900	4720	4720	4300	—	2850	7080	7080
700	2100	2100	5320	5320	4700	—	3150	7980	7980
800	2400	2300	6060	6060	5400	—	3450	9090	9090
900	2500	2500	6760	6760	5700	—	3750	10140	10140
1000	2700	2680	7400	7400	6000	—	4020	11100	11100
1100	2800	2780	8200	8200	6200	—	4170	12300	12300
1200	2900	2880	9000	9000	6500	—	4320	13500	13500
1400	3000	2980	—	10630	6700	—	4470	—	15950
1600	3000	2980	—	12200	6700	—	4470	—	18300
1800	3000	2980	—	13800	6700	—	4470	—	20700

27—480 M. of W&H/ND/79

APPENDIX 8

Characteristics of common gases causing hazards

(All percentages are per cent by volume in air)

Sl. No.	Name of Gas	Chemical Formula	Common Properties	Specific gravity or vapour density (air=1)	Physiological effects	Maximum Safe Exposure limit (%)		Explosive limit (percent)		Likely location of highest concentration	Most Common Sources
						60-minutes	8-hours	Lower	Upper		
1	2	3	4	5	6	7	8	9	10	11	12
1	Carbon dioxide	CO_2	Colourless, odourless; when breathed in large quantities may cause acid taste. Non-inflammable.	1·53	Cannot be endured at 10 % for more than few minutes, even if subject is at rest and oxygen content is normal. Acts on respiratory nerves.	4·0 to 6·0	0·5	..	..	At bottom; when heated, may stratify at points above bottom.	Products of combustion sewer gas sludge gas. Also issued from carbonaceous strata.
2	Carbon monoxide	CO	Colourless, odourless, tasteless, inflammable, poisonous, non-irritating.	0·97	Combines with haemoglobin of blood. Headache in few hours at 0·02%; unconsciousness in 30 minutes at 0·2 to 0·25%, fatal in 4 hours at 0·1%.	0·04	0·005	12·5	74·0	Near top, especially, if present with illuminating gas.	Manufactured fuel gas, fuel gas products, combustion products of motor exhausts; fires of almost any kind.
3	Chlorine	Cl_2	Yellowish green colour, choking odour, detectable in vey low concentrations, non-inflammable.	2·49	Irritates respiratory tracts, kills most animals in very short time at 0·1%.	0·0004	0·0001	..	..	At bottom	Chlorine cylinder and feed line leaks.
4	Gasoline	C_6H_{12} to C_9H_{20}	Colourless, odour noticeable at 0·03%; inflammable.	3·0 to 4.0	Anaesthetic effects when inhaled; rapidly fatal at 2·4%; dangerous for short exposure at 1·1 to 2·2%.	0·4 to 0·7	0·1	1·3	6·0	At bottom	Service Stations, garages, storage.
5	Hydrogen	H_2	Colourless odourless, tasteless, inflammable.	0·07	Acts mechanically to deprive tissues of oxygen; does not support life.	..	..	4·0	74·0	At top	Manufactures fuel gas sludge.
6	Hydrogen Sulphide	H_2S	Rotten egg odour in small concentrations; odour not evident at high concentrations, colourless, inflammable.	1·15	Exposure for 2 to 15 minutes at 0·01% impairs sense of smell; exposure to 0·07 to 0·1% rapidly causes acute poisoning. Paralyses respiratory centre; death in few minutes at 0·2%.	0·02	0·001	4·3	46·0	Near bottom but may be above bottom if air is heated and highly humid.	Coal gas, petroleum, sewer gas, fumes from blasting, sludge gas.

7	Methane .	CH_4	Colourless, odourless, tasteless, highly inflammable; non-poisonous.	0·55	Acts mechanically to deprive tissues of oxygen; does not support life.	Probably no limit provided oxygen percentage is sufficient for life.	1·0	5·0	15·0	Normally at top extending to a certain depth.	Natural gas, sludge gas, manufactured fuel gas, sewer gas, in swamps or marshes.
8	Nitrogen .	N_2	Colourless tasteless, non-flammable. Principal constituent of air (about 79%).	0·97	Physiologically inert.	..	..	..	..	Near top but may be found at bottom.	Sewer gas, sludge gas also issues from some rock strata.
9	Oxygen (in air)	O_2	Colourless, tasteless, odourless, supports combustion, non-poisonous.	1·11	Normal air contains 21% of oxygen; man can tolerate down to 12% minimum safe limit 8 hours exposure 14 to 16%. Below 10 dangerous to life. Below 5 to 7% probably fatal.	..	..	..	..	Variable at different levels.	Oxygen depletion; from poor ventilation and absorption or chemical consumption of available oxygen.
10	Sludge Gas .	About 60% methane and 40% carbon dioxide with small amounts of H_2,N_2,H_2S, O_2	May be practically odourless, colourless, inflammable.	0·94	Will not support life.	Would vary widely with composition.	..	5·3	19·3	Near top of structure.	From digestion of sludge in tanks.

APPENDIX 9

Equipment and simple tests for detection of gases and oxygen deficiency

Combustible gas indicators are used for testing the atmosphere for hazardous concentration of inflammable gases and vapours and for making quantitative estimates of the percentage of combustible gas present. The indicator consists of a battery operated unit, which oxidises or burns a sample of the atmosphere to be tested over a heated catalytic filament which is a part of a balanced electrical circuit. Combustibles in the samples are burned on the hot wire, thus raising its temperature and increasing its resistance in proportion to the concentration of the combustibles in the sample. The imbalance in the electrical circuit causes the deflection of the pointer of the meter which indicates on a scale, the concentration of combustible gases or vapours in the sample. This scale is calibrated in percentages of the lower explosive limit. The indicator is generally calibrated for a single specific inflammable gas, but may also be calibrated for known mixtures of gases and vapours. The types of combustible gas indicator may be selected to suit the gas or vapour usually encountered.

Carbon monoxide indicator may be used to detect the percentage of the gas present. There are both hand operated and battery operated units which determine electrically the percentage of carbon monoxide present. They are very sensitive to low concentrations of gas and reliably indicate low but dangerous concentrations of carbon monoxide.

The sample of the atmosphere drawn into the indicator is oxidised to carbon dioxide by catalytic action. The heat liberated by oxidation is proportional to the amount of carbon monoxide present and is measured by a differential thermocouple in series with the indicating meter which is calibrated to read directly the percentage of carbon monoxide in the atmosphere.

Colorimetric detectors are used to detect specific gases like carbon monoxide, hydrogen sulphide etc. In a specific gas detector, when a sample of the atmosphere is drawn into the instrument, that specific gas reacts chemically with the special substance in the detector producing a change in colour. The colour with its intensity produced is compared with a chart to estimate the percentage of the specific gas present.

In the carbon monoxide detector the chemical used is iodine pentoxide or palladium chloride. In hydrogen sulphide detector the chemical used is lead acetate.

Oxygen deficiency indicator is an adaptation of the flame safety lamp used by miners, for testing the atmosphere suspected of being deficient in oxygen. Normally the indicator is used from an external source to test the suspected atmosphere. The sample of air is drawn in, using an aspirator bulb and the flame inside the lamp is observed. When the atmosphere is normal the flame of the lamp will have normal appearance. With decreased oxygen content in the tatmosphere and the absence of another combustible gas, he flame will be dimmer. When the oxygen content of the atmosphere is as low as 16% or lower; the flame will be extinguished.

At altitudes more than 1500m above sea level, the flame may continue to burn even if the percentage of oxygen in the atmosphere is less than 16%. Hence this possibility must be considered at high altitudes.

Simple tests: In the absence of the indicators and detectors mentioned above, the following simple tests must be conducted after providing sufficient forced or natural ventilation.

In asphyxiating conditions, a safety lamp must be used. The lamp should burn continuously for at least 5 minutes in the atmosphere under test. It is essential to check if the lamp is undamaged before being used.

For hydrogen sulphide, a filter paper moistened with 5% solution of lead acetate is exposed for five minutes to the atmosphere under test. As hydrogen sulphide is heavier than air, the atmosphere at the bottom of the manhole should be tested. The presence of hydrogen sulphide gas is indicated by the paper turning grey or brown. The greater the percentage of the gas, the darker will be the colour.

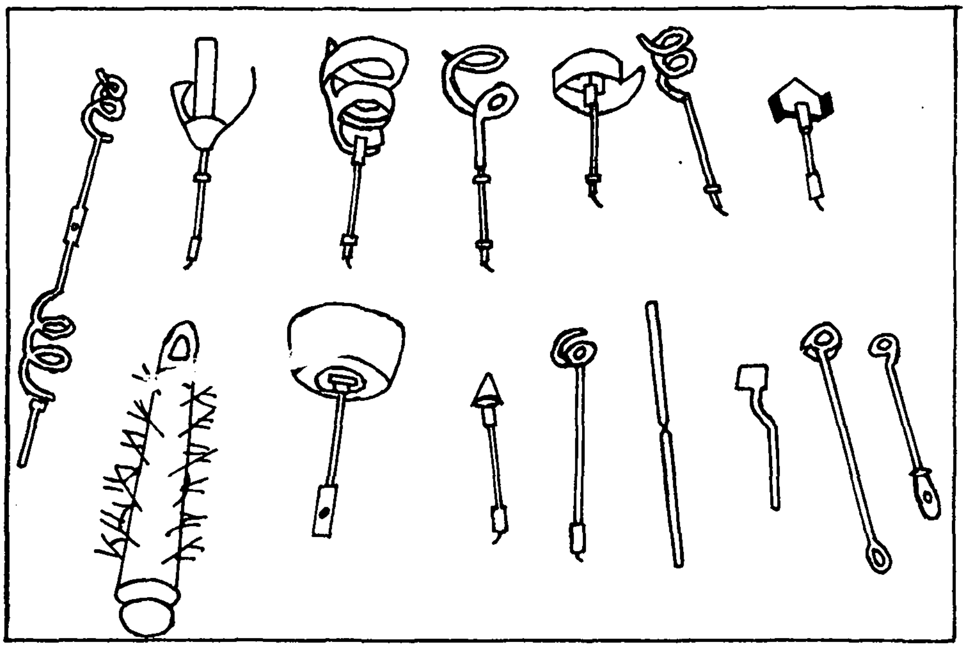
Detectors and indicators for various gases and oxygen deficiency normally encountered in sewage works are as follows :

Gas or Vapour	*Detector*
Hydrogen Sulphide	Lead acetate impregnated paper, (qualitative) Hydrogen sulphide ampoules, Hydrogen sulphide detector (qualitative).
Methane	Combustible gas indicator, Oxygen deficiency indicator, Methane alarm.
Carbon dioxide	Oxygen deficiency indicator.
Nitrogen	Oxygen deficiency indicator.
Oxygen	Oxygen deficiency indicator.
Carbon monoxide	Carbon monoxide indicator, Carbon monoxide tube (quantitative).
Hydrogen	Combustible gas indicator. Oxygen deficiency indicator.
Gasoline	Combustible gas indicator. Oxygen deficiency indicator (for concentration over 0.3%).
Sludge	Combustible gas indicator Oxygen deficiency indicator, Methane alarm.
Chlorine	Aqueous ammonia, Odour.

APPENDIX 10

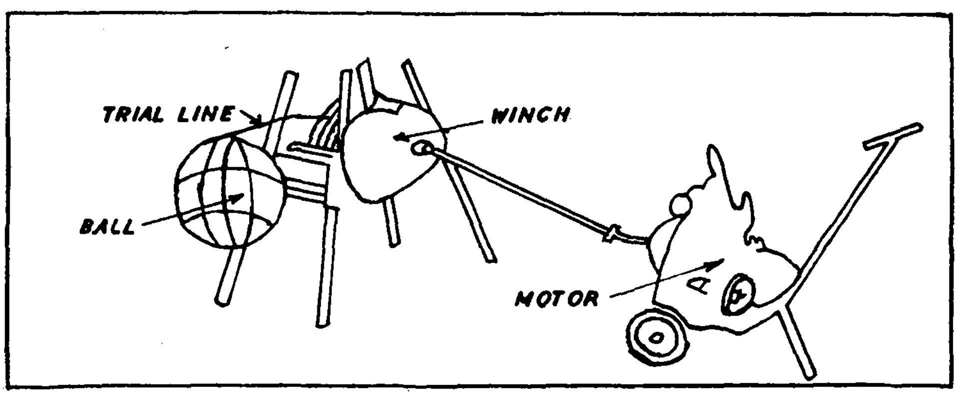
Sewer cleaning equipment and devices

SEWER CLEANING EQUIPMENT AND DEVICES



ROOT CUTTERS

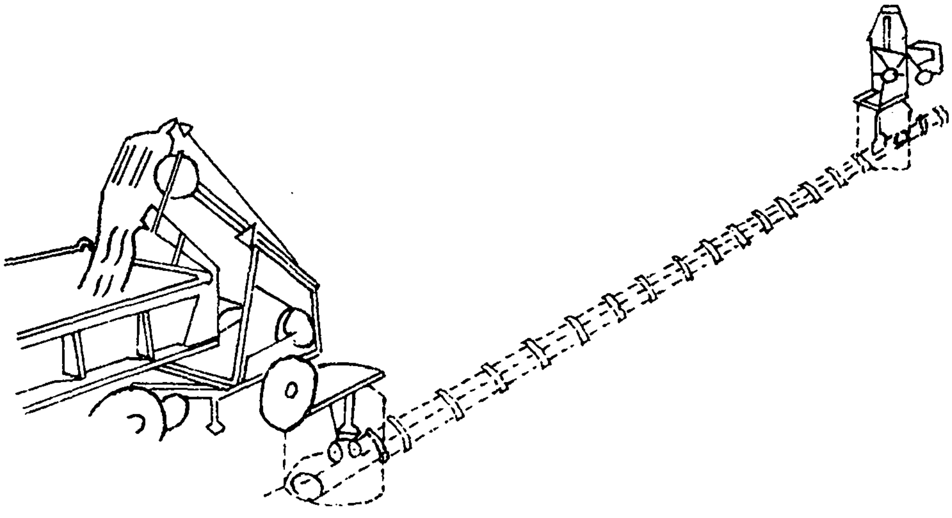
FIG. A-10-1



SEWER BALL WITH MECHANICAL ARRANGEMENT

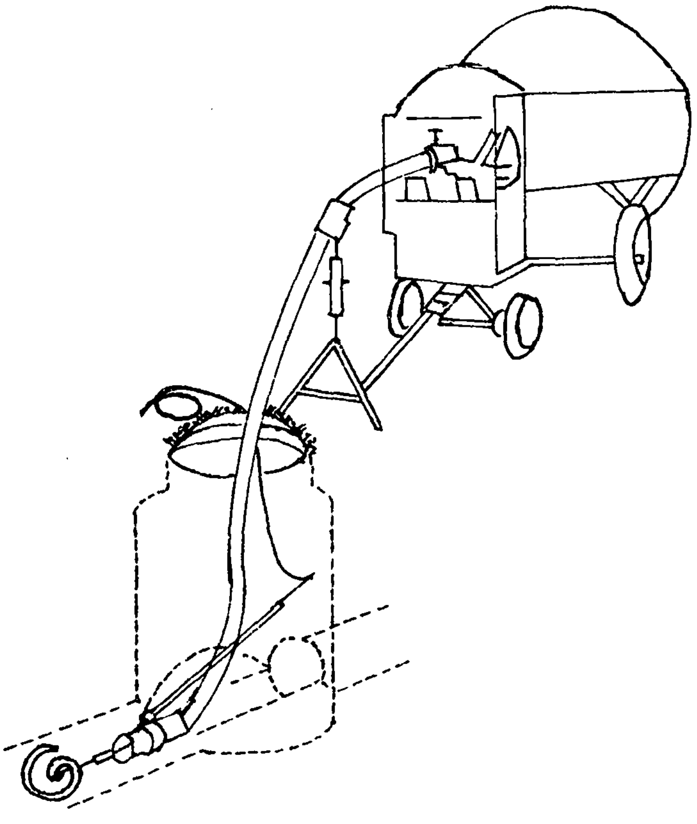
FIG. A-10-2

28—480 M. of W&H/ND/79



SEWER CLEANING BUCKET MACHINE

FIGURE: A – 10.3



RODDING MACHINE WITH FLEXIBLE SEWER RODS

FIGURE: A – 10.4

APPENDIX 11

Screens

Estimate the screen requirements for a plant treating a peak flow of 50 mLd sewage.

Solution

Q max=50 mLd=578.8 lps=0.5788 m^3/sec.

Desired velocity through screen V=0.8 mps

(at ultimate flow)

Net area of screen=0.5788/0.8=0.7225 m^2=0.72 m^2

Adopting screens with bars of 10 mm width & 50 mm clear opening, gross area= $\frac{0.72\times6}{5}$ =0.864 m^2=0.86 m^2

Velocity above screen $V = \frac{0.8\times5}{6} = 0.67$ mps

Headloss through the screen=0.0729 (V^2-v^2)=0.0729 $(0.8^2-0.67^2)$.

=0.01343 m say 0.013 m.

If the screen openings are half plugged with screenings leaves and debris, velocity through the screen is doubled.

Headloss=0.0729 × $(1.6^2-0.67^2)$

=0.0729×2.27 × 0. 93

=0.1538=0.15

A regime of cleaning has to be established, based on this headloss of 0.15 m (which indicates that the screens are half plugged).

Assuming that the inclination of the screen to horizontal is at 60°, the gross area of screen needed would be:

$$\frac{0.86}{\sqrt{3}/2}=0.9931=1 \text{ m}^2$$

29—480 M. of W&H/ND/79

APPENDIX 12

Grit Chamber

Design a grit chamber with flat surfaces and a Parshall flume for velocity control, the maximum and minimum flows being 50 mLd (578.8 lps) and 10 mLd (115.8 lps) respectively. Velocities at maximum and minimum rates of flow are to be controlled at 0.3 mps.

Solution

From the Table 11-2, for a Q_{max}=50 mLd, a throat width of 300 mm or 0.30 m appears to be suitable.

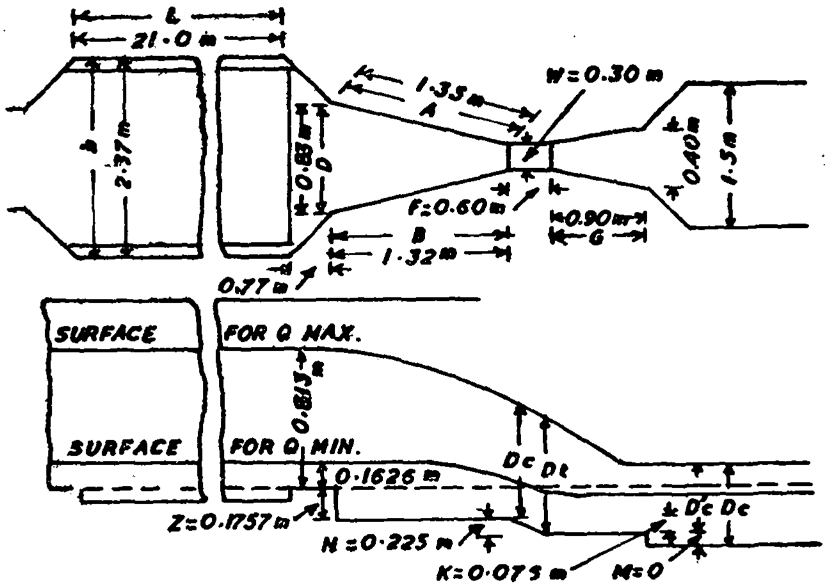


FIG. A 12·1 PARSHALL FLUME FOR GRIT CHAMBER

Determination of value of Z

From equation (11-11)

$$\frac{d_{min}}{d_{max}} = \frac{10}{50} = \frac{1.1\left[\dfrac{115.8}{2264\times0.30}\right]^{2/3} - Z}{1.1\left[\dfrac{578.8}{2264\times0.30}\right]^{2/3} - Z}$$

$$\text{or} \quad \frac{1}{5} = \frac{0.3383 - Z}{0.9888 - Z} \quad \text{or } Z = 0.1757\text{m}$$

Determination of depths of flow in grit chamber for the various rates of flow :

Using the expression, $\qquad D = 1.1\left[\dfrac{Q}{2264W}\right]^{2/3} - Z,$

the values obtained are shown in Col. 3 of the accompany table.

d_{min}=0.3383—0.1757=0·1626 m ; and
d_{max}=0.9888—0.1757=0.8131 m

Rate of flow mLd	*Rate of flow* lps	*Depth of flow* (m)	*Cross-sectional Area* (m^2)	*Velocity* (mps)
1	2	3	4	5
10	115.8	0.1626	0.3853	0.3005
15	173.7	0.2675	0.6339	0.2740
20	231.6	0.3612	0.8559	0.2706
25	289.4	0.4472	1.059	0.2732
30	347.4	0.5279	1.251	0.2777
35	405.3	0.6039	1.431	0.2832
40	463.2	0.6766	1.603	0.2889
45	521.1	0.7462	1.768	0.2947
50	578.8	0.8131	1.927	0.3004

Computation of width of grit chamber :

Using equation (11—13)

$$b = \frac{Q_{max}}{1000\times d_{max}\cdot v_{max}} = \frac{Q_{min}}{1000\times d_{min}\cdot v_{min}}$$

$$= \frac{578.8}{1000\times0.8131\times0.3} = 2.372\text{m say } 2.37$$

Cross sectional areas for different flows :

Values of $b \times d$ for the different rates of flow are shown in Col. 4.

Velocities for the different flows :

Obtained by using the expression, $v = \frac{Q}{1000A}$ are shown in Col. 5.

Variation of the lowest value of velocity from the maximum velocity is less than 10% which is permissible.

Check for conditions of free flow :

It is necessary that free flow conditions exist at the throat entrance and flume exit, if the design velocities in the grit chamber are to be unaffected.

The free flow conditions are assured in a Parshall flume when the critical depth D_c is not affected by downstream conditions. Such conditions prevail when the total energy H at the section where the depth of flow D_c satisfies the relation

$$D_t + \left[\frac{Q}{D_t W}\right]^2 \frac{1}{2g} \gtrless H+N \qquad .....(i)$$

$$\text{or } D_t + \left[\frac{Q}{1000\, D_t W}\right]^2 \frac{1}{2g} \gtrless H+N \text{(for Q in lps)}...(ii)$$

Neglecting loss of energy due to friction

$$H = D_t + Z = 1.1 H_A \qquad ......(iii)$$

$$\text{and } Q = 2264 \times W\, H_A^{3/2} \qquad ......(iv)$$

Substituting (iii) and (iv) in (ii) we get

$$\frac{D_t}{H_A} + 0.2615\left[\frac{H_A}{D_t}\right]^2 - 1.1 \gtrless \frac{N}{H_A} \qquad ....(v)$$

Differentiating (v) with respect to D,

$$\frac{1}{H_A} - \frac{D_t}{H^2_A}\cdot\frac{dH_A}{dD_t} + 0.2615\left[-2\frac{H^2_A}{D^3_t} + \frac{1}{D^2_t}.2H_A\frac{dH_A}{dD_t}\right] = \frac{N}{H^2_A}\,\frac{dH_A}{dD_t}$$

$$\text{or } \frac{dH_A}{dD_t}\left[\frac{D_t}{H^2_A} + 0.5230\,\frac{H_A}{D^2_t} + \frac{N}{H^2_A}\right] = \frac{1}{H_A} + 0.5230\,\frac{H^2_A}{D^3_t}.$$

For Q to be a maximum $\frac{d\,H_A}{dD_t} = 0$

$$\therefore \frac{1}{H_A} = 0.5230.\frac{H^2_A}{D^3}$$

$$\text{or } \frac{D}{H_A} = (0.5230)^{1/3} = 0.8058$$

Substituting this value in (v)

$$0.8058 + \frac{0.2615}{0.8058^2} - 1.1 \gtrless \frac{N}{H_A}$$

$$\text{i.e. } 0.1085 \gtrless \frac{N}{H_A}$$

$$\text{or } H_A \gtrless 9.226\ N$$

$$\therefore \quad Q_{max} \not> 2264W \times (9.226N)^{3/2}$$

$$\not> 63450\ W\ N^{3/2}$$

For a throat width of 0.3 m and N from Table 11-2 equal to 0.225 m, Q_{max}=2032 lps.

Since this is larger than the Q_{max} of 578.8 lps, free flow conditions prevail.

Tail Water conditions :

For preventing submergence

$$D_c + K \geq D'_c$$

$$D'_c + M \geq D_e$$

Depth at critical sections

$$D_c \text{ (and } D'_c) = \frac{V^2}{g} = \frac{Q^2}{10^6\, b^2 D^2_c \times g} \qquad ...(vi)$$

$$\text{or} \quad D_c = \sqrt[3]{\frac{Q^2}{10^6 b^2\, g}}$$

From Table 11-2, width at 'C'=0.6 m

$$\therefore \quad D_c = \sqrt[3]{\frac{578.8^2}{10^6 \times 0.3^2 \times 9.8}} = 0.7243\text{m}$$

$$\text{and } D'_c = \sqrt[3]{\frac{578.8^2}{10^6 \times 0.6^2 \times 9.8}} = 0.4488\text{m}$$

D_e can be calculated by equating the specific forces at the sections at D'_c and D_c, the pertinent equation being

$$\frac{Q^2}{10^6 g\ A_1}\quad \bar{Z}_1 A_1 = \frac{Q^2}{10^6 g\ A_2} + \bar{Z}_2 A_2 \quad ...(vii)$$

Where $\bar{Z}_1$, and $\bar{Z}_2$ are the depths from the surface of the centroids of the section.

Since the sections are rectangular $\bar{Z}_1$ and $\bar{Z}_2$ are at half depths.

Assuming a width at the section at D_e of 2 m,

$$\frac{578.8^2}{10^6 \times 9.8 \times 0.4488 \times 0.6} + \frac{0.4488}{2} \times 0.4488 \times 0.6$$

$$= \frac{578.8^2}{10^6 \times 9.8 \times D_e \times 2} + \frac{D_e}{2} \times D_e \times 2$$

whence $D^3_e = 0.18733\ D_e + 0.01709$

Solving $D_e = 0.376$ m

Assuming a width at the Section at D_e of 1.5 m

$$\frac{578.8^2}{10^6 \times 9.8 \times 0.4488 \times 0.6} + \frac{0.4488}{2} \times 0.4488 \times 0.6$$

$$= \frac{578.8^2}{10^6 \times 9.8 \times 0.4488 \times 0.6} + \frac{D_e}{2} \times D_e \times 1.5$$

whence $D^3_e = 0.2497\ D_e + 0.3038 = 0$

Solving $D_e = 0.422$ m.

(Velocity at D_e = 0.9143 mps)

For Q_{min} = 115.8 lps,

$$D_c = \sqrt[3]{\frac{115.8^2}{10^6 \times 0.3^2 \times 9.8}} = 0.2477\text{m}$$

$$D'_c = \sqrt[3]{\frac{115.8^2}{10^6 \times 0.6^2 \times 9.8}} = 0.1535\text{m}$$

and D_c = 0.144

For Q_{max}

$D_c+K=0.7243+0.075 > 0.4488(D'_c)$

$D'_c+M=0.4488+0>0.422(D_c$ for 1.5m)

For Q_{min}

$D_c+K=0.2477+0.075>0.1535(D'_c)$

$D'_c+M=0.1535+0>0.144(D_c$ for 1.5m)

All conditions for free flow are satisfied and a Parshall flume of 300 mm throat width is the required one.

From Table 11-1, for removing particles of specific gravity 2.65 and size 0.15 mm, the ideal overflow rate is 1300 $m^3/d/m^2$ at 10° C. For the lowest temperature of 15°C the overflow rate will be 1300×1.15=1495 or say 1500.

Assuming a correction factor of 2/3, the design overflow rate works out to 1000 $m^3/d/m^2$

Q_{max}=50 mLd = 50,000 m^3/d

$$\frac{Q}{A} = 1000\text{m}^3/\text{d}/\text{m}^2$$

$$\text{Area of grit chamber} = \frac{50{,}000}{1{,}000} = 50\text{m}^2$$

Width =2.37 m as already determined

$$\text{Length} = \frac{50}{2.37} = 21\text{m}$$

$$\text{Detention period} = \frac{V}{Q} = \frac{50 \times 0.8131 \times 1440}{50{,}000} = 1.17\text{min}$$

(about 1 min usually adopted)

Providing a free board of 0.187m, depth=1m

Hence, dimensions of grit chamber are 21m×2.37m×1m

APPENDIX 13

Secondary Sedimentation Tank

Design a Secondary Settling Tank of an activated sludge treatment plant for 50 mLd (peak flow) operating with an MLSS of 3000 mg/l

Solution.

Peak factor $=2.25$

Av. flow $=\dfrac{50}{2.25}=23$ mLd.

Adopting a surface loading rate of 20 $m^3/d/m^2$ at average flow,

Surface area required

$$=\frac{23\times10^6}{10^3\times20}=1150m^2$$

Check Surface loading for peak flow :

$$\frac{50\times10^6}{10^3\times1150}=43.48m^3/d$$

(checks as it is in the prescribed range of 40-50)

For a solids loading of 125 kg/day/m^2 at average flow, area required

$$=\frac{23\times10^6\times3000}{1000\times125\times1000}=552m^2$$

Area needed for peak flow at a solids loading of 250 kg/day m^2

$$=\frac{50\times10^6\times3000}{1000\times250\times1000}=600\ m^2$$

The higher surface are a of 1150 m^2 is to be adopted.

Adopting a circular tank,

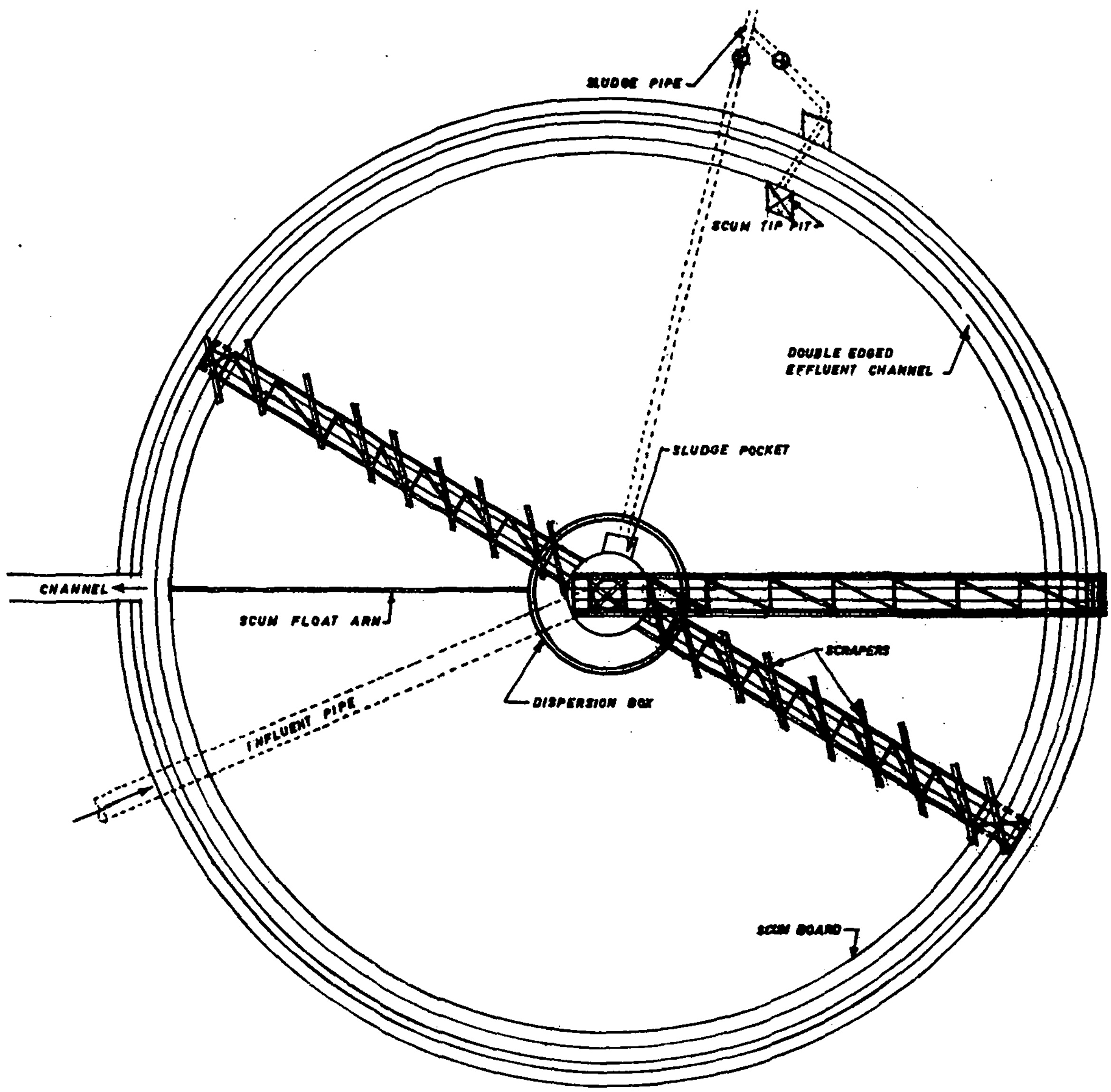
$$d=\sqrt{\frac{1150\times4}{\pi}}=38.26\text{ m or say 38 m}$$

Weir loading

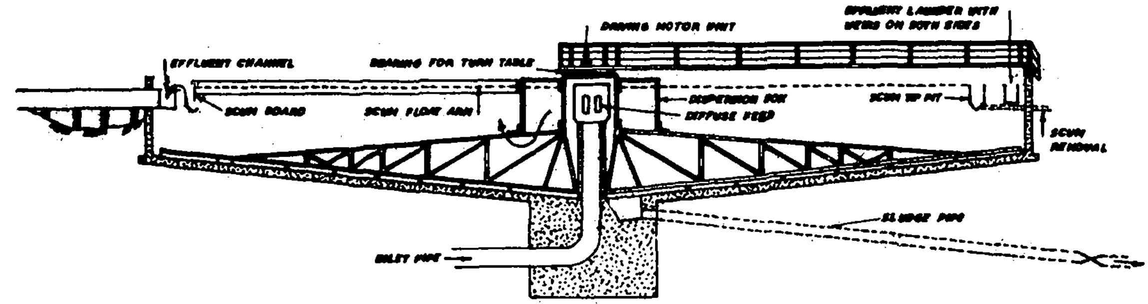
$$=\frac{23\times10^3}{\pi\times38}=192.6m^3/d/m\ (>\text{than permissible})$$

So, provide a trough instead of a single weir at the Periphery. Typical details of Sedimentation tank are given in Fig. A-13.1(a)

30—480 M. of W&H/ND/79



TYPICAL DETAILS OF SEDIMENTATION TANK
FIGURE: A-13.1(a)



SECTION

FIG. A 13·1(b) TYPICAL DETAILS OF SEDIMENTATION TANK

APPENDIX 14

Activated Sludge Plant

Design a conventional activated sludge plant to treat settled domestic sewage with diffused air aeration system given the following data :

Population	1,50,000
Per capita sewage contribution .	150 lpcd
Settled sewage BOD_5 . .	200 mg/l
Average flow	22.5 mLd
Effluent BOD_5 required . .	10 mg/l

Solution :

Efficiency required $= \dfrac{190}{200} = 95\%$

From Table 13-1,

select F/M=0.2 and MLSS(X_t)=3000 mg/l

$$\frac{F}{M} = \frac{Q.L_A}{\dfrac{VX_t}{1000}} \text{ or } 0.2 = \frac{22.5 \times 200 \times 1000}{V \times 3000}$$

Volume of aeration tank

$$V = \frac{22.5 \times 200 \times 1000}{0.2 \times 3000} = 7{,}500 \text{ m}^3$$

Check for Hydraulic Retension Time :

$$HRT = \frac{V}{Q \times 1000} \times 24 = \frac{7500 \times 24}{22.5 \times 1000}$$

= 8 hrs. (within the limits)

Volumetric loading

$$\frac{Q \times L_a}{V} = \frac{22.5 \times 200}{7500} = 0.6 \text{ kg } BOD_5/m^3$$

(within the prescribed range of 0.3 to 0.7)

Return Sludge (for SVI=100)

$$\frac{Q_r}{Q} = \frac{X_t}{\dfrac{10^6}{SVI} X_t} = \frac{3000}{\dfrac{10^6}{100} - 3000} = 43\%$$

(within the prescribed range of 25 to 50%)

Tank dimensions :

Adopting a depth of 3 m and width of 4.5 m, length of aeration channel needed :

$$= \frac{7500}{3 \times 4.5} = 555 \text{ m or } 560 \text{ m}$$

Provide a continuous channel with six baffles, each length being 80 m to give a total length of 560 m. Total width of each unit (including 6 baffles of 0.25 m thickness) $= 7 \times 4.5 + 1.5 = 33$m

free board=0.5 m

∴overall dimensions of the aeration tank are $80 \text{m} \times 33 \text{m} \times 3.5$ m.

Check for horizontal velocity :

$$Q + Q_r = \frac{2.5(1 + 0.43) \times 10^3}{24 \times 60} = 22.34 \text{ m}^3/\text{min.}$$

$$V = \frac{22.34}{3 \times 4.5} = 1.655 \text{ m/min.}$$

(O. K. being about 1.5 m/min.)

Air requirements :

Air needed = 100 m³/day per kg BOD_5 removed

$$= \frac{100 \times 190 \times 22.5}{24 \times 60}$$

= 296.9 or 300 m³/min.

Standard diffuser plates of $0.3 \text{m} \times 0.3 \text{m} \times 25$ mm passing 1.2m³ of air/min./m² with 0.3 mm pores are chosen.

Total no. of plates needed

$$= \frac{300}{0.3 \times 0.3 \times 1.2} = 2780.$$

A plate concentration of 30% extra is provided in the first half of the tank to take care of more frequent clogging in this zone.

No. of plates needed in the first half of the tank = 1800

A clear separation of plates of 0.9 m to avoid interference from rising streams of bubbles, the c/c distance of the rows of plates=0.9+0.3=1.2 m.

Providing 8 plates in a row of 1.2 m spacing,

plates in the first 276 m length=230 rows × 8 in each row =1840

Balance (2780—1840)=940 plates are to be provided in

560—276=284 m. Spacing$= \dfrac{284 \times 8}{940} = 2.42$ m or say 2.4 m c/c.

Check for minimum air availability

In the second half 940 diffuser plates give

$940 \times 0.3 \times 0.3 \times 1.2$ m³/min;

∴ air available per m of channel :

$$\frac{940 \times 0.108}{284}$$

= 0.36 m³/min.

Satisfactory as it is more than the prescribed values 0.25 m³/min.

31—480 M. of W&H/ND/79

APPENDIX 15

Design of Trickling Filters

Problem

Design a high rate trickling filter plant to treat settled domestic sewage with a BOD_5 of 200 mg/l for an average flow of 22·50 mLd to satisfy an effluent BOD_5 of 10 mg/l. Take peak factor as 2.25.

Solution

Since the effluent BOD_5 required is less than 30 mg/l a two-stage filtration plant has to be adopted. While designing the plant, the filters are designed for average flow only. However, the distribution arms, underdrainage system, other pipelines etc. are designed for the peak flow and checked for average flow.

For the scheme shown in figure 14.B.(a), the design is as follows :—

Adopting an organic loading of 800 g/d/m³ (incoming BOD_5 only)

Volume of 1st. stage filter

$$=22.50\times10^6 \times\frac{200}{10^3}\times\frac{1}{800}=5625 \text{ m}^3$$

Adopting a depth of 1·5m,

Filter area needed

$$=\frac{5625}{1.5}=3750 \text{ m}^2$$

Using a Circular filter, dia

$$=\sqrt{\frac{3750\times4}{\pi}}$$

$$=69.11 \text{ m}$$

Since rotary distributors are available indigenously only upto 60 m, it is desirable to have at least two units.

∴ dia of filter

$$=\sqrt{\frac{3750\times4}{\pi\times2}}=48.88\text{m}$$

say = 50 m.

Applying Rankin's formula for the first stage filter and varying values of R_1=0·5; 0·75; 1·0; 1·5; 2·0; 2·5 and 3·0

Efficiency of First stage filter

$$=E_2=\frac{1+R_1}{1.5+R_1}$$

gives values of 75; 77·78; 80; 83·33; 85·77; 87·50 and 88·88% respectively. These values are entered in column 2 & 3 of table respectively.

Similarly the efficiency of second stage filter =

$$E_3=\frac{1+R_2}{2+R_2}$$

for various values of R_2 are entered in columns 5 and 6 of Table. Column 4 gives the BOD_5 passing through the first stage filter.

Now, the combined efficiency of the filters required to give an effluent BOD_5 of 10 mg/l=95%

Efficiency of two stages $E_c = E_2 + E_3\,(1-E_2)$;

for a R_1 value of 0·5, this will be

$0{\cdot}95=0{\cdot}75 + E_3\,(1-0{\cdot}75)$

or $E_3=0{\cdot}8$

For $E_3=0{\cdot}8$

R_2 value from column 5 of Table = 3·0

Similarly R_2 values for various E_3 values for different R_1 values to obtain 95% efficiency are given in column 7 of table.

TABLE

Sl. No.	R_1 Recirculation ratio of 1st. stage filter	E_2 Efficiency of 1st. stage filter	S_4 BOD_5 passing through 1st. stage filter	R_2 Recirculation ratio of 2nd stage filter	E_3 Efficiency of 2nd stage filter	R_2 values various R_1 values to give 95% efficiency
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	0·50	75·00	50·00	0·50	60·00	3·00
2	0·75	77·78	44·44	0·75	63·64	2·50
3	1·00	80·00	40·00	1·00	66·67	2·00
4	1·50	83·33	33·33	1·50	71·43	1·50
5	2·00	85·77	28·66	2·00	75·00	1·00
6	2·50	87·50	25·00	2·50	77·78	0·50
7	3·00	88·88	22·22	3·00	80·00	..

The hydraulic loadings for different R_1 values, in terms of kLd/m² for the average flow

$$=\frac{22.50\times10^6}{2\times10^3}\times\frac{4}{\pi\times50^2}(1+R_1) \text{ is worked out}$$

R_1	Hyd. loading (m³/d/m²)
0·50	8·6
0·75	10·03
1·00	11·47
1·50	14·33
2·00	17·20
2·50	20·06
3·00	22·94

Choose R_1=2 for First Stage Filter and R_2=1 for Second Stage Filter.

Organic loading (recirculation included) for the 2 filters of dia 50 m and depth 1.5 m

$$=22.5\times10^6\frac{(200+2\times28.6)}{10^3}\times\frac{4}{2\times\pi\times50^2\times1.5}$$

= 982.8 g/d/m³

This is less than 1800 g/d/m³ and therefore the equations are applicable.

Choosing an organic loading 500 g/d/m³

Volume of 2nd stage filter

$$= 22.50 \times 10^6 \times \frac{28.6}{10^3} \times \frac{1}{500}$$

$= 1287\ m^3$

Adopting a depth of 1 m

area of filter $= 1287\ m^2$

Check for hydraulic loading

$$= 22.50 \times 10^3 \times \frac{(1+1)}{1287} = 34.95\ kLd/m^2$$

which is more than permissible.

∴ Area required for the maximum permissible hydraulic loading of 30 kLd/m²

$$= 22.5 \times 10^3 \times \frac{(1+1)}{30} = 1500\ m^2$$

Adopting Circular Filter, dia

$$= \sqrt{\frac{1500 \times 4}{\pi}}$$

$= 43.71$ m

say $= 45$ m.

So adopt 1 unit of 45 m dia and 1 m depth for 2nd Stage Filter.

DETAILED DESIGN OF 1ST STAGE FILTER

Rotary Distributor

This is designed for the peak flow + the recirculation of the average flow at the rates prescribed. In this case the recirculation is 2 times the average flow.

∴ total flow through the filters at peak flow with 2.25 peak factor

$= 22{\cdot}5 \times 2{\cdot}25 + 2 \times 22.5 = 95{\cdot}625$ mLd

or $= 1{\cdot}108\ m^3/s$

This flow is divided into two units.

∴ flow through each unit at peak flow $= 0{\cdot}554\ m^3/s$.

Adopting a velocity of 2 mps, dia of central column

$$= \sqrt{\frac{0.554 \times 4}{\pi \times 2}} = 0.594\ m$$

Provide a central column $= 0{\cdot}6$ m.

Check for velocity at average flow :

Ave. flow $= 22{\cdot}5 \times 10^6 \times (1+2) = 67{\cdot}5$ mLd

$= 0{\cdot}7812\ m^3/s$

∴ Vel. at ave. flow

$$= \frac{0.7812}{2} \times \frac{4}{\pi \times (0.6)^2}$$

$= 1{\cdot}38$ mps

(> 1 mps stipulated)

Distributor

Assuming rotary reaction spray type distributor with 4 arms

discharge per arm

$$= \frac{0.554}{4} = 0.1385\ m^3/s$$

Dia of Filter = 50 m.

$$\text{Arm length} = \frac{50-2}{2} = 24\ m \text{ with 4 sections}$$

of 6 m each.

The flow in the arms has to be adjusted for every section of 6 m length in the proportion of the area covered by these lengths of the arm. Therefore, the area covered by the different lengths of the arm are calculated.

Let A_1, A_2, A_3 and A_4 be the areas covered by each length of arm starting from the centre. Allowing for 0·7 m dia in centre to be used up for central column etc. the areas are

$$A_1 = \pi\,(6.35^2 - 0.35^2) = 126.3\ m^2$$
$$A_2 = \pi\,(12.35^2 - 6.35^2) = 352.6\ m^2$$
$$A_3 = \pi\,(18.35^2 - 12.35^2) = 578.9\ m^2$$
$$A_4 = \pi\,(25.00^2 - 18.35^2) = 905.5\ m^2$$

∴ the proportionate area for each length of arm 1st. i.e. from column to 6·35 m

$$= \frac{A_1}{A_1+A_2+A_3+A_4}$$

$$= \frac{126.3}{1963} = 6.4\%$$

Similarly	2nd . .	18·0%
	3rd . .	29·5%
	4th . .	46·1%

Orifices

Assuming a dia of 25 mm for the orifices with a C_d value of 0·6 and head causing flow equal to 1·5 m discharge thro' each orifice

$$= C_d A \sqrt{2gh}$$

$$= 0.6 \times \frac{\pi}{4} \times 0.025^2 \times \sqrt{2 \times 9.8 \times 1.5}$$

$= 0.001596\ m^3/s$.

∴ no. of orifices required in each arm

$$= \frac{\text{Total discharge thro' arm}}{\text{discharge thro' each orifice}}$$

$$= \frac{0.1385}{0.001596} = 86.82 \quad \text{Say 87 nos.}$$

No. of orifices in each section of the arm is

1st Section $\frac{6.4}{100} \times 87 \approx 6$ nos.

2nd ,, $\frac{18}{100} \times 87 \approx 16$ nos.

3rd ,, $\frac{29.5}{100} \times 87 \approx 26$ nos.

4th ,, $\frac{46.1}{100} \times 87 \approx 39$ nos.

Diameter of different sections of the arm

The flow through velocity in the arm should be less than 1·2 mps.

(a) Discharge thro' 1st. section $= 0.1385\ m^3/s$

∴ Cross sectional area with 1.2 mps

$$= \frac{0.1385}{1.2}$$

$= 0.1154\ m^2$

Assuming circular section, dia of pipe

$$= \sqrt{\frac{0.1154\times4}{\pi}} = 0.3834 \text{ m}$$

$$= \text{say } 400 \text{ mm.}$$

(b) Discharge thro' second section

$$=(1-0.064)\times0.1385 = 0.1296 \text{ m}^3/\text{s}$$

For a velocity of 1.2, dia needed

$$= \sqrt{\frac{0.1296\times4}{1.2\times\pi}} = 0.371 \text{ m}$$

$$= \text{say } 370 \text{ mm.}$$

$$\text{Velocity} = \frac{0.1296\times4}{\pi\times0.37^2} = 1.2 \text{ mps.}$$

(c) Discharge thro' 3rd Section

$$=[1-(0.064+0.18)]\times0.1385=0.1047\text{m}^3/\text{s}$$

For a velocity of 1.2 mps, dia

$$= \sqrt{\frac{0.1047\times4}{1.2\times\pi}} = 0.3334 \text{ m}$$

$$= \text{say } 340 \text{ mm.}$$

$$\text{Velocity} = \frac{0.1047\times4}{\pi\times0.34^2} = 1.154 \text{ mps.}$$

(d) Discharge thro' 4th Section

$$= 0.461\times0.1385= 0.06386\text{m}^3/\text{s}$$

For a velocity of 1.2 mps. dia

$$= \sqrt{\frac{0.06386\times4}{1.2\times\pi}} = 0.2665 \text{ m}$$

Choose 270 mm

$$\text{Velocity} = \frac{0.06386\times4}{\pi\times0.27^2} = 1.116 \text{ mps.}$$

Spacing of Orifices

1st Section 6 nos. in 600 cm i.e.

$$\frac{600}{6} = 100 \text{ cm c/c}$$

2nd Section 16 nos. in 600 cm i.e.

$$\frac{600}{16} = 37.5 \text{ cm c/c}$$

3rd Section 26 nos. in 600 cm i.e.

$$\frac{600}{26} = 23 \text{ cm c/c}$$

4th Section 39 nos. in 600 cm i.e.

$$\frac{600}{39} = 15 \text{ cm c/c}$$

Under drainage system

Total discharge through each filter at peak flow

flow$=0.554 \text{ m}^3/\text{s}$

The under drainage system is designed with a peripheral collecting channel fed by semi-circular laterals placed at 0·5 m c/c with a slope of 2·5% in each half circle. The invert level of all laterals at their junction with the peripheral main collecting channel is kept at the same R.L.

Ave discharge per lateral

$$= \frac{0.554}{100\times2} = 0.00277 \text{ m}^3/\text{s}$$

$$ar^{2/3} = \frac{nq}{s^{1/2}} = \frac{0.015\times0.00277}{0.025^{1/2}} = 0.0002630$$

The laterals are designed to flow half full to provide for proper ventilation

$$\text{i.e. } \frac{a}{A} = 0.25; \quad \frac{a}{d_o^2} = \frac{\pi}{4} \times 0.25=0.1962$$

From Appendix 26

for $\frac{a}{d_o^2}$ of 0.1962, value of $\frac{ar^{2/3}}{d_o^{8/3}}$

$$=0.05915$$

$$\therefore \quad d_o^{8/3} = \frac{ar^{2/3}}{0.05915} \times \frac{0.0002630}{0.05915}$$

$$\therefore \quad d_o = \left[\frac{0.0002630}{0.05915}\right]^{3/8} = 0.1312 \text{ m.}$$

Adopting 14 cm dia

$$\frac{ar^{2/3}}{d_o^{8/3}} = \frac{0.0002630}{(0.14)^{8/3}} = 0.04975$$

From Appendix 26

$$\text{Corresponding } \frac{a}{d_o^2} = 0.1711$$

$$\text{Velocity} = \frac{0.00277}{0.1711\times0.14^2} = 0.8260 \text{ mps}$$

$(>0.75$ required)

Check for Velocity at average flow

Total discharge $= 22{\cdot}50 + 2\times22{\cdot}50=67{\cdot}50$ mLd

$= 0{\cdot}7812 \text{ m}^3/\text{s.}$

$\therefore$ Flow thro' each filter $= 0{\cdot}3906 \text{ m}^3/\text{s}$

$$\text{Average flow per lateral} = \frac{0.3906}{100\times2} = 0.00195 \text{ m}^3/\text{s}$$

$$ar^{2/3} = \frac{0.015\times0.00195}{0.025^{1/2}} = 0.000185$$

$$\text{and} \quad \frac{ar^{2/3}}{d_o^{8/3}} = \frac{0.000185}{(0.14)^{8/3}}$$

$$= 0.03501$$

$$\text{Corresponding} \quad \frac{a}{d_o^2} = 0.1363$$

$$\text{Velocity} = \frac{0.00195}{0.1363 \times 0.14^2}$$

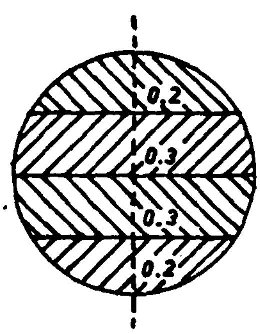
$$= 0.73 \text{ mps}$$

$$(>0.6 \text{ required})$$

The laterals are covered with perforated blocks capable of withstanding the load of the filter media. It should be ensured that there is at least 15% of the total filter area available in the form of inlet openings for the flow into the laterals to ensure proper ventilation.

In the present design the total surface area of the laterals at the floor level of the filter is about 20% of the filter area. Therefore, it is to be provided with cover blocks having about 75% openings so that the inlet area available is about 15% of the filter area.

Design of main Collection Channel



It is desirable to provide the main collection channel along the periphery of the filter as shown in sketch. The flow is divided into two and the flow from each semi-circle is collected in the peripheral main channel which is laid to a constant slope of 0.5%. The filter can be divided into four segments and the main channel checked to see if free fall conditions exist while flow from the laterals of each segment falls into it.

To provide a free fall from the invert of the laterals assume the depth of flow to be 5% less than the depth of semi-circular section.

$$\text{i.e.} \quad \frac{y}{d_o} = \frac{0.95}{2}$$

$$= 0.475$$

1st Segment

$$q = 0.1 \times 0.554 = 0.0554 \text{ m}^3/\text{s}$$

From Appendix 26

$$\text{for} \quad \frac{y}{d_o} = 0.475; \quad \frac{ar^{2/3}}{d_o^{8/3}}$$

$$= 0.1426 \quad \& \quad \frac{a}{d_o^2} = 0.3677$$

For a slope of 0.5% and n=0.015

$$ar^{2/3} = \frac{nq}{s^{1/2}} = \frac{0.015 \times 0.0554}{0.005^{1/2}}$$

$$= 0.01175$$

$$d_o = \left[\frac{0.01175}{0.1426}\right]^{3/8}$$

$$= 0.3921 \text{ m.}$$

Adopting 40 cm or 0·4 m dia & 0.5% slope

$$\frac{ar^{2/3}}{d_o^{8/3}} = \frac{0.01175}{(0.4)^{8/3}} = 0.1577$$

$$\text{And for this} \quad \frac{y}{d_o} = 0.503 \quad \& \quad \frac{a}{d_o^2}$$

$$= 0.3964$$

$$\text{Velocity} = \frac{0.0554}{0.03964 \times 0.4^2}$$

$$= 0.8734 \text{ mps}$$

$$(>0.75 \text{ required})$$

2nd Segment

$$q = 0.25 \times 0.554 = 0.1385 \text{ m}^3/\text{s}$$

Vertical depression at the end of the 2nd section

$$= \frac{\pi D}{4} \times \frac{0.5}{100} = 0.2 \text{ m}$$

Total additional flow in this section

$$= 0.15 \times 0.554 = 0.8310 \text{ m}^3/\text{s}$$

Flow that can be accommodated

$$= 0.2 \times 0.4 \times 1 = 0.08$$

(assuming 1 mps velocity)

Hence choose a bigger section; say 50 cm.

Redesign of 1st Segment

$$ar^{2/3} = 0.01175$$

$$\frac{ar^{2/3}}{d_o^{8/3}} = \frac{0.01175}{(0.5)^{8/3}} = 0.0746$$

For this value

$$\frac{a}{d_o^2} = 0.2284$$

$$\& \quad \frac{y}{d_o} = 0.3325$$

$$\text{Velocity} = \frac{0.0554}{0.2284 \times 0.5^2}$$

$$= 0.9266 \text{ mps}$$

$$(>0.75 \text{ mps required})$$

Check for average flow (recirculation included) flow in Segment

$$1 = \frac{0.7812}{2} \times 0.1$$

$$= 0.03906 \text{ m}^3/\text{s}$$

$$\therefore \quad \frac{ar^{2/3}}{d_o^{8/3}} = \frac{0.03906 \times 0.015}{0.005^{1/2}} \times \frac{1}{(0.5)^{8/3}}$$

$$= 0.0526$$

$$\text{For this} \quad \frac{a}{d_o^2} = 0.1777 \quad \& \quad \frac{y}{d_o} = 0.2768$$

$$\text{Velocity} = \frac{0.03906}{0.1777 \times 0.5^2}$$

$$= 0.8794 \text{ mps}$$

$$(>0.6 \text{ mps required})$$

2nd Segment

$q = 0.1385\ m^3/s$

$$ar^{2/3} = \frac{0.1385 \times 0.015}{0.0005^{1/2}} = 0.02939$$

$$\frac{ar^{2/3}}{d_o^{8/3}} = \frac{0.02939}{(0.5)^{8/3}} = 0.1865$$

For this value

$$\frac{a}{d_o^2} = 0.4501 \ \& \ \frac{y}{d_o} = 0.5575$$

$\therefore$ Velocity

$$= \frac{0.1385}{0.4501 \times 0.5^2} = 1.231 \text{ mps}$$

depth of flow

$= 0.5575 \times 0.5 = 0.27875$ m
$= 0.28$ m

depth from invert of channel to invert of lateral

$= 0.25 + 0.20 = 0.45$ m

$\therefore$ Clearance

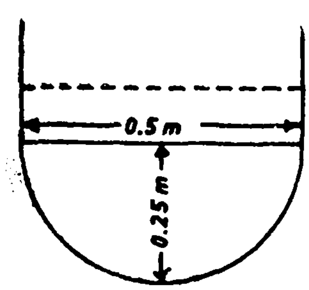
$= 0.45 - 0.28 = 0.17$ m

ensuring free flow conditions.

3rd Segment

$q = 0.4 \times 0.554 = 0.2216\ m^3/s$

Assuming depth of flow above



semi-circular section to be 'X'

$$r = \frac{d}{p} = \frac{\frac{\pi R^2}{2} + 0.5 \times x}{\pi R + 2 \times x}$$

$$= \frac{\frac{\pi \times 0.25^2}{2} + 0.5x}{\pi \times 0.25 + 2x}$$

$$= \frac{0.0962 + 0.5x}{0.7854 + 2x}$$

$$\therefore 0.2216 = \frac{1}{0.015}\left[0.0962 + 0.5x\right] \times \left[\frac{0.0962 + 0.5x}{0.7854 + 2x}\right]^{2/3} \times (0.005)$$

or
$$\frac{(0.0962 + 0.5x)^{5/3}}{(0.7854 + 2x)^{2/3}} = 0.04810$$

Solving the LHS for different values of x by trial and error.

Value of x		Value of RHS
0·1	. . . .	0·04151
0·13	. . . .	0·04637
0·14	. . . .	0·04817

$\therefore x = 0.14$ m

and depth of flow $= 0{\cdot}25 + 0{\cdot}14 = 0{\cdot}39$ m against available depth of $0{\cdot}25 + 0{\cdot}3 = 0{\cdot}55$ m which ensures free flow conditions.

$$\text{Velocity} = \frac{0.2216}{0.0962 + 0.07} = 1.334 \text{ mps.}$$

4th Segment

$q = 0{\cdot}5 \times 0{\cdot}554 = 0{\cdot}277\ m^3/s$

let 'y' be the depth of flow above the semi-circular section, then

as in 3rd segment

$$r = \frac{a}{P} = \frac{0.0962 + 0.5y}{0.7854 + 2y}$$

$$\therefore 0.277 = \frac{1}{0.015} \frac{(0.0962 + 0.5y)^{5/3}}{(0.7854 + 2y)^{2/3}} \times 0.005^{1/2}$$

or
$$\frac{(0.0962 + 0.5y)^{5/3}}{(0.7854 + 2y)^{2/3}} = 0.05876$$

Solving the LHS by trial and error for different values of 'y'

$y = 0.2$ m

and depth of flow

$= 0.25 + 0.2 = 0.45$ m

against available depth of

$0.25 + 0.4 = 0.65$ m

ensuring free flow conditions.

$$\text{Velocity} = \frac{0.277}{0.0962 + 0.1} = 1.412 \text{ mps}$$

Design of exit channel

$q = 0.554\ m^3/s$ for each filter

Assuming a channel of rectangular section with a slope of 0.5%

$$P = 2d + w \text{ and } A = w.d;\ r = \frac{A}{P}$$

$$\text{or } r = \frac{w.d}{2d + w}$$

$$q = \frac{1}{n} ar^{2/3} s^{\frac{1}{2}};\ 0.554 = \frac{1}{0.015} \times w.d. \times \frac{(w.d.)^{\frac{1}{2}}}{(2d+w)} \times 0.001^{\frac{1}{2}}$$

or $\frac{(w.d)^{5/3}}{(2d+w)} = 0.1175$

Assuming a depth of 0·4 m to prevent submergence and solving by trial and error

$w = 0.855$

Now for a value of $w = 0{\cdot}855$ what is the value of 'd'

i.e. $\frac{(0.885\ d)^{5/3}}{(2d+0.855)^{2/3}}$

$= 0.1175$

$d = 0.395$ m ($<$ 0.455 m)

∴ Effluent channel from each filter will be of size 0·855 m × 0·455 m with 0·5% slope.

Ventilation

Since the filter is large having a dia of 50 m provision for open grating area has to be made at 1/250 of the filter area

∴ Area of grating needed

$= \frac{\pi \times 50^2}{4 \times 250}$

$= 7.85\ m^2$

say $8.0\ m^2$

∴ Provide 8 nos. of gratings of size 4 m × 0·25 m providing a total of 8·0 m^2 ventilation area.

2nd Stage filter

The details of the second stage filter is also worked out on these lines.

Typical sketch of a Trickling Filter is shown in Fig. A-15.1.

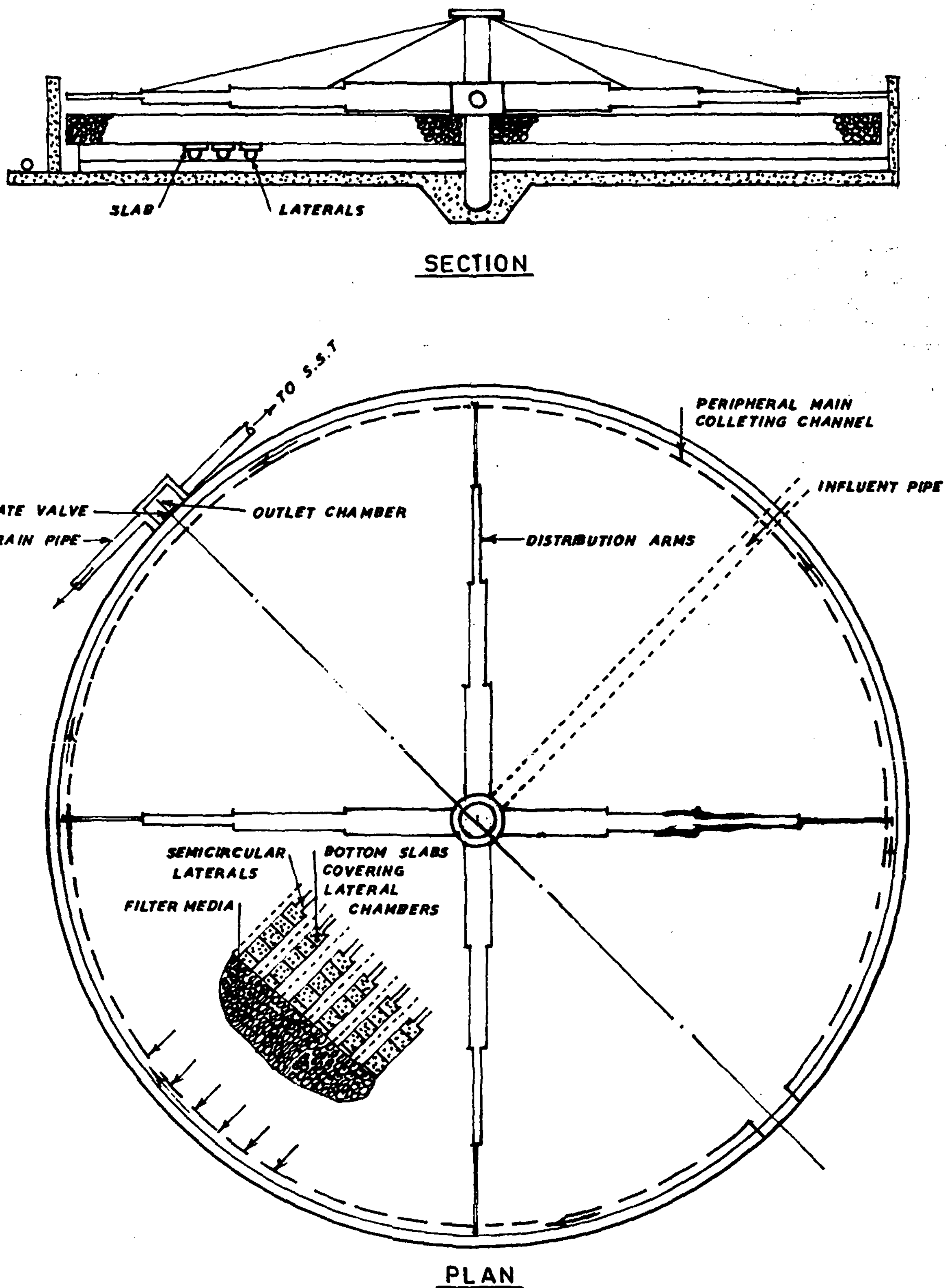


FIG. A 15·1 TYPICAL DETAIL OF TRICKLING FILTER

APPENDIX 16

Waste Stabilization Pond

Design an oxidation pond based on the following given data :

Location . . .	26° Latitude
Elevation . . .	1000 m above sea level
Mean monthly temperature.	25°0 max. & 10°0 min.
Population served .	10,000
Sewage flow . .	150 lpcd
BOD_5 for raw sewage	300 mg/l
Desired effluent BOD_5 .	30 mg/l
Sky clearance factor .	0·60
Per capita BOD contribution per day .	0·045 kg/day
Pond removal constant at 20° C . .	0·1/d

Solution

Areal BOD_5 loading at 26° N latitude=210 kg/ha/day

correction factor for elevation$=1+(0{\cdot}003\times10)$

$= 1{\cdot}03$

correction factor for sky clearance $= \dfrac{100}{104.5}$

corrected loading factor

$$=\frac{210}{1.03}\times\frac{100}{104.5}$$

$= 195.1$ or 195 kg/ha/day

Applied BOD_5

$= 10{,}000\times0.045=450$ kg/day

Pond area

$$=\frac{450}{195} = 2.309 \text{ ha or } 2.3 \text{ ha}$$

Pond removal constant at 10°C,

$$P_T = 0.1\times1.047^{-10}$$
$$=0.06310$$

detention period

$$t=\frac{1}{P_T}\log\frac{L}{L-Y}$$

$$=\frac{1}{0.0631}\log\frac{300}{30}$$

$$=\frac{1}{0.0631}$$

$= 15.85$ days or 16 days.

Pond Volume

$$=\frac{150\times10{,}000\times16}{1000} = 24{,}000 \text{ m}^3$$

$$\therefore \text{depth}=\frac{24{,}000}{2.3\times10{,}000} = 1.045 \text{ m say } 1.1\text{m}$$

Sludge Accumulation

Per Capita

Suspended solids in raw sewage	= 90 gms
Settleable solids	=54 gms
Fixed solids fraction in the Settleable solids	=16 gms
Volatile matter in the Settleable Solids	=38 gms
Volatile matter destroyed . .	$=2/3\times38$ gms
Volatile matter remaining in sludge	$=\frac{1}{3}\times38=12.67$ gms

During digestion 25% of the volatile matter is converted into fixed solids

$\therefore$ Total fixed solids

$$=16+\frac{38}{4} = 25.5 \text{ gms}$$

Total sludge solids per capita

$=25.5+12.67=38.17$ gms

Assuming a moisture of 80% and specific gravity of sludge of 1·05, volume of wet sludge accumulated per day

$$= 38.17\times\frac{100}{20}\times\frac{1}{1.05} = 181.8 \text{ ml.}$$

Vol. of sludge per year from 10,000 population

$$=\frac{181.8\times365\times10{,}000}{10^6} = 663.4 \text{ m}^3$$

depth of accumulation per year

$$=\frac{663.4}{2.3\times10^4} = 0.02885 \text{ m}$$

So a cleaning schedule of at least once in 5 years may be adopted.

Adopting a parallel-series system of 6 ponds with 4 primary ponds and 2 secondary ponds of equal area with 2 primary ponds feeding a secondary pond(which would also give the required primary pond area as 65 to 70% of the total pond area)

Area of each pond

$$=\frac{2.3}{6} = 0.39 \text{ or } 0.4 \text{ ha.}$$

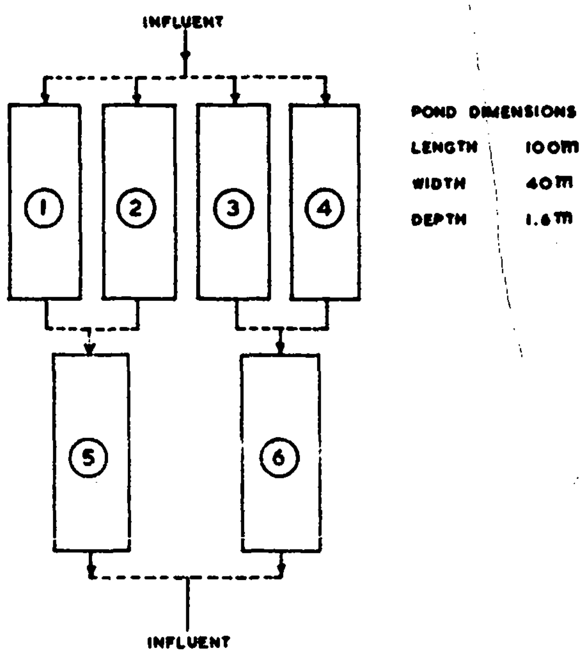
Adopting rectangular ponds of length to breadth=3:1

breadth $= 36.06$ m or 40 m

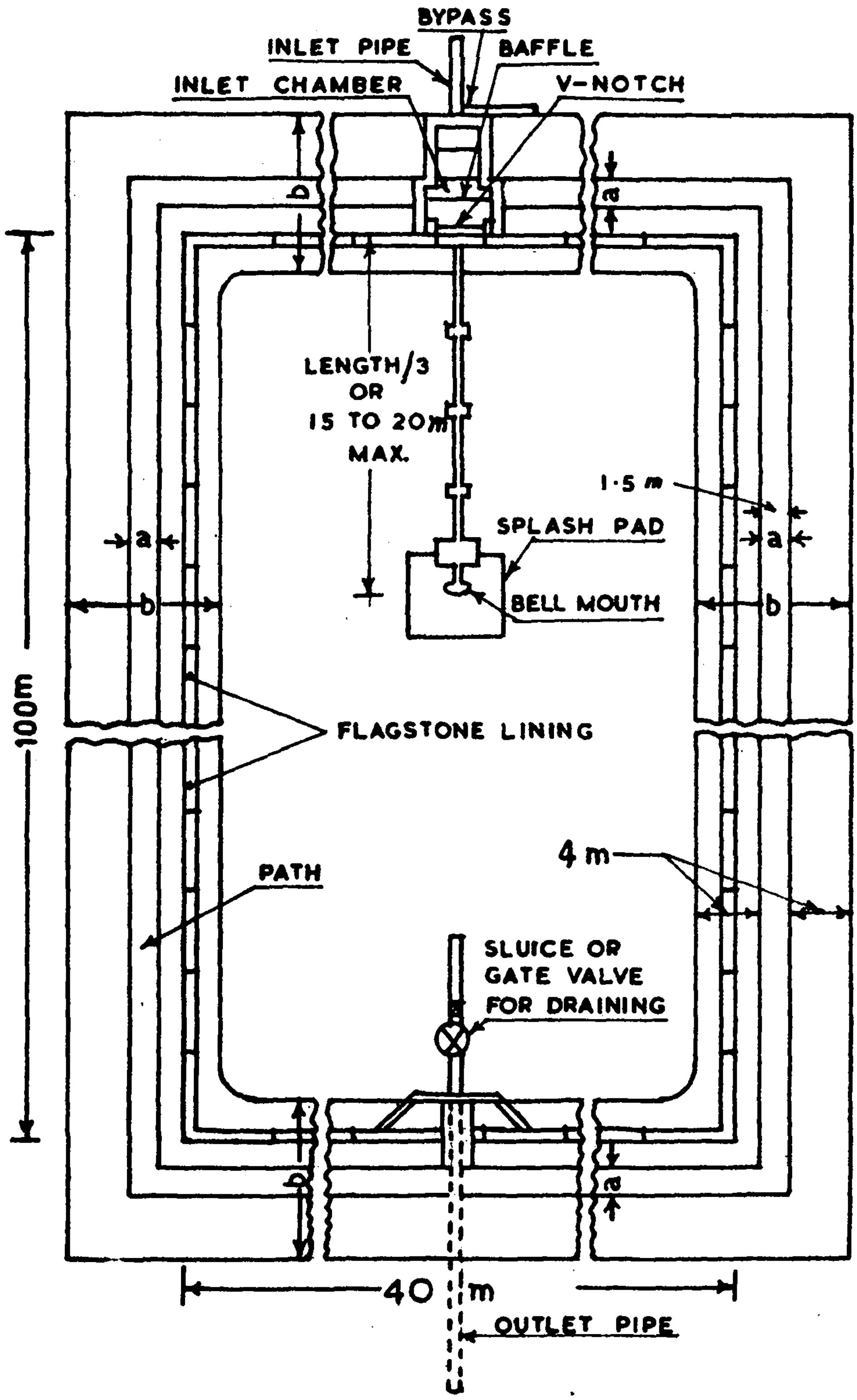
$$\text{length} = \frac{4000}{40} = 100 \text{ m}$$

Adopt units of 40 m × 100 m × 1·6 m (including 0·5 m freeboard)

Typical details are shown in Fig A-16.1, A-16.2 and A-16.3.



LAYOUT PLAN OF STABILIZATION PONDS
FIGURE. A-16.1



a = TOP WIDTH OF THE BUND

b = BOTTOM WIDTH OF THE BUND

TYPICAL PLAN OF A WASTE STABILIZATION POND

FIG. A-16.2

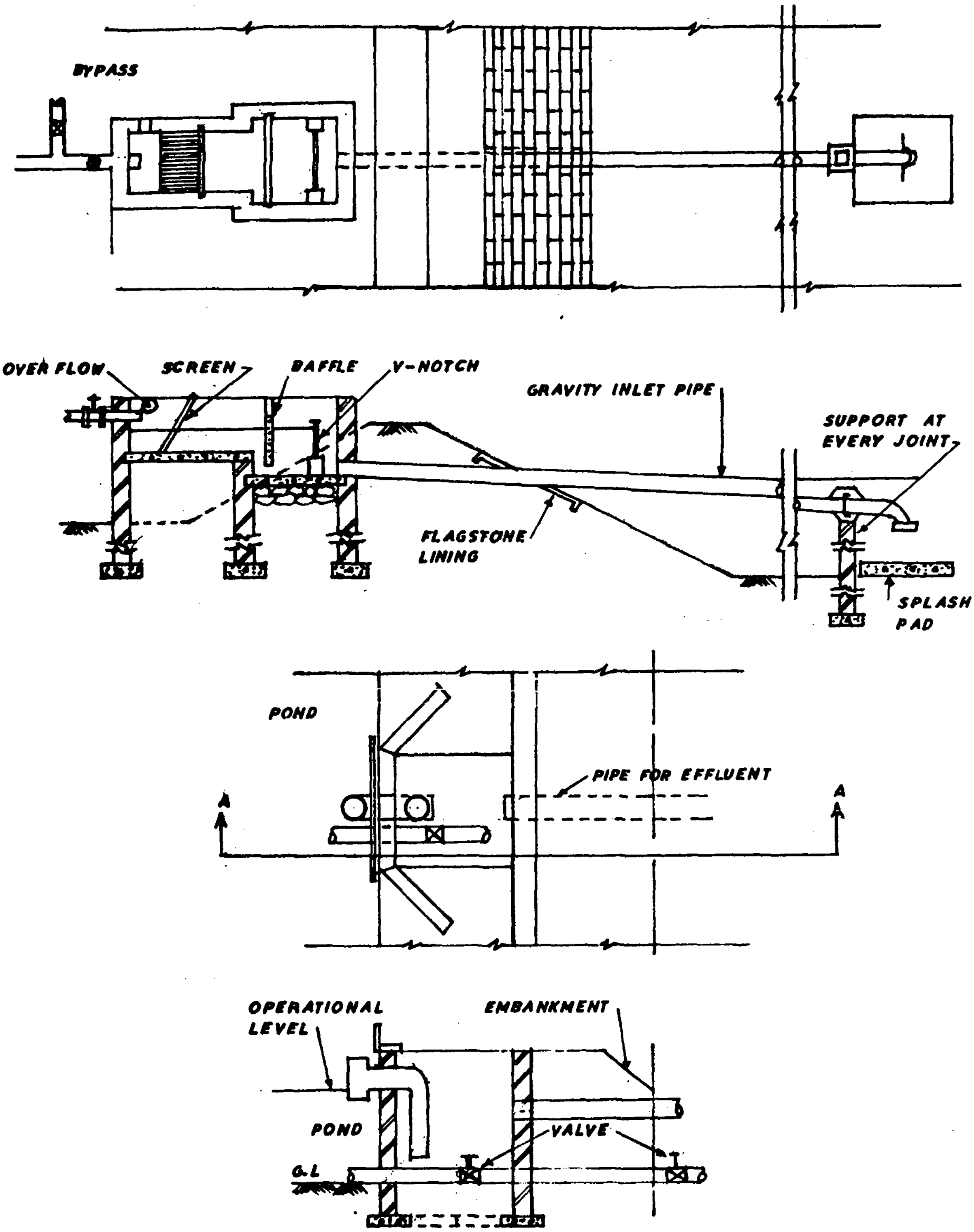


FIG. A 16.3 TYPICAL DETAILS OF INLET AND OUTLET CHAMBER

APPENDIX 17

Oxidation Ditch

Design an oxidation ditch to treat sewage with an initial BOD_5 of 300 mg/l from a contributory population of 5000 to give an effluent BOD_5 of 20 mg/l.

Total flow of sewage at 150lpcd $= 5000 \times 150/10^6 = 0{\cdot}75$ mLd

BOD_5 removal required

$$= \frac{300-20}{300} = 93\%$$

Choose an $F/M = 0{\cdot}1$ and $MLSS = 3{,}000$

$$\frac{F}{M} = \frac{Q\ La}{\left(\frac{V}{1000}\right) X_t};$$

$$0.1 = \frac{0.75 \times 300 \times 1000}{V \times 3000}; \quad \therefore V = 750\ m^3$$

$$HRT = \frac{V \times 24}{Q \times 1000} = \frac{750 \times 24}{0.75 \times 1000} = 24 \text{ hrs}$$

(within the specified range)

Volumetric loading :

$$\frac{Q \times La}{V} = \frac{0.75 \times 300}{750} = 0.3\text{kg } BOD_5/m^3$$

(in the prescribed range of 0.2 to 0.4)

return sludge (for SVI = 100)

$$\frac{Qr}{Q} = \frac{X_t}{\frac{10^6}{SVI} - X_t} = \frac{3000}{\frac{10^6}{100} - 3000} = 0.43$$

(in the prescribed range of 0·35 to 1·5)

O_2 for BOD_5 removal $= 1{\cdot}2$ kg/kg of BOD removed
$= 1{\cdot}2 \times 0{\cdot}75 \times 280 = 252$ kg/day
$= 10{\cdot}5$ kg/hr.

Oxygenation capacity of the cage rotor of dia 70 cm, 75 rpm, at immersion depth of 16 cm $= 2{\cdot}8$ kg of O_2/hr/m length.

$\therefore$ length of rotor needed on the basis of oxygenation capacity

$$= \frac{10.5}{2.8} = 3.75 \text{ m or say } 4 \text{ m}$$

length of rotor needed from velocity consideration of 0·3 mps (assuming 150m³ of ditch volume per metre length of rotor for adequate circulation) $= \frac{750}{150} = 5$ m

Adopt 2 rotors of 2·5 m length each

width of ditch = 3m (giving a clearance of 0·25 m on either side)

adopting a depth = 1·5 m

Surface area needed

$$= \frac{750}{1.5} = 500m^2$$

Adopting 2 ditches, surface area of ditch = 250 m²

Length of the ditch

$$= \frac{250}{3} = 83.3 \text{ or } 85 \text{ m}$$

Power requirements for each rotor

$= 1.35 \times 2.5 = 3.375$ kw say 3.5 kw

Energy requirements per year $= 3{\cdot}5 \times 2 \times 24 \times 365 =$ 61,320 kw

A sketch of oxidation ditch is shown in Fig. A-17.1.

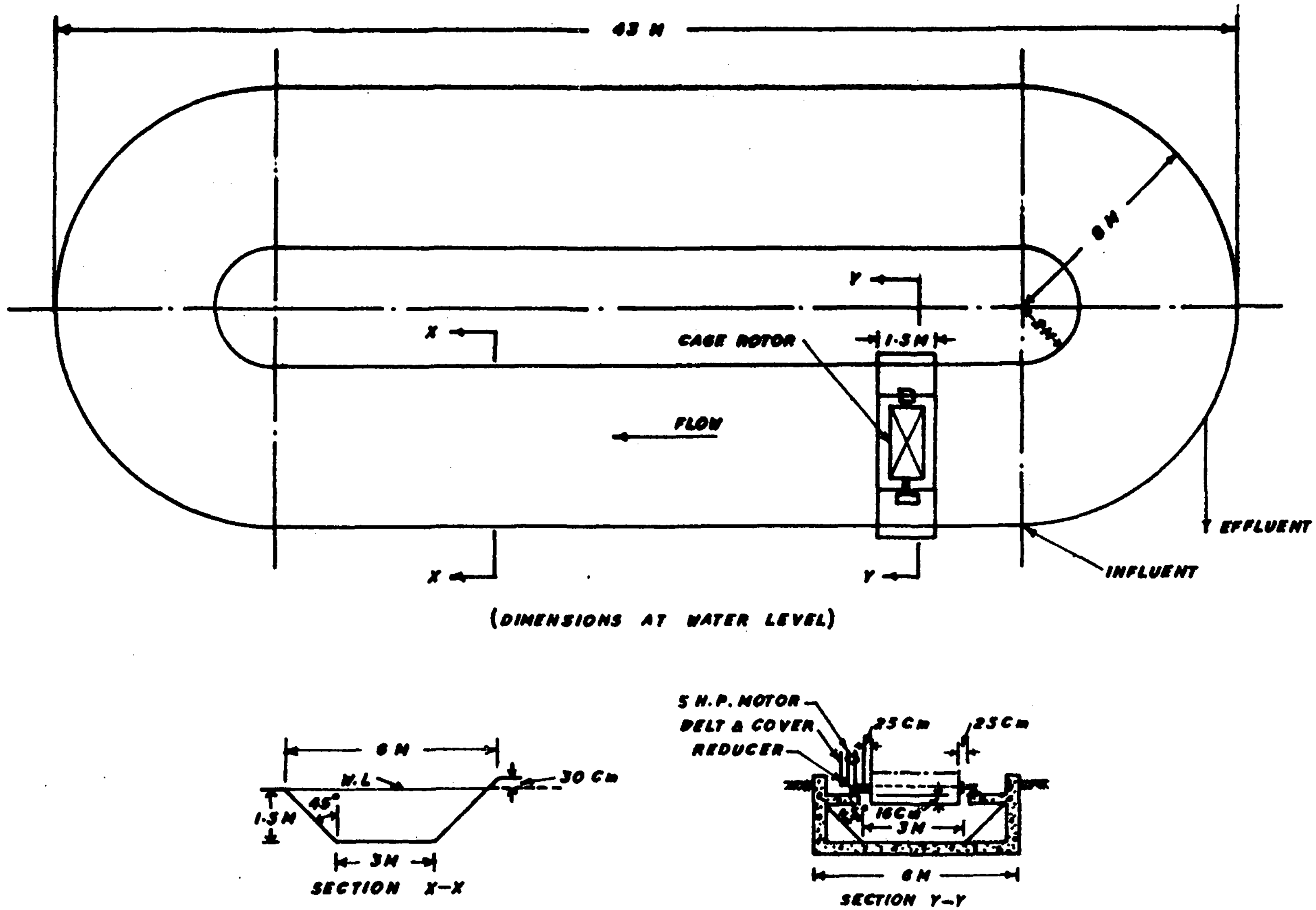


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FIG. A 17·1 SKETCH OF OXIDATION DITCH

APPENDIX 18

Sludge Digestion

Design a digester for 1,50,000 persons for digesting mixed raw and activated sludge.

Solution:

From Table 16—1,

Volumes of mixed raw primary, activated and mixed digested sludges.

% of V.M. in raw mixed sludge

$$= \frac{58}{85} \times 100 = 68.23\%$$

% of non-volatile matter in sludge

$$= \frac{27}{85} \times 100 = 31.77\%$$

$\therefore$ Sp. gr. of dry solids in mixed raw sludge.

$$= \frac{100}{Sd} = \frac{68.23}{1.0} + \frac{31.77}{2.5}$$

$$\therefore \; Sd' = 1.236$$

Sp. gravity of wet solids in mixed raw Pr. + AS.

Percentage solids (4%)

$$\therefore \frac{100}{S_{w1}} = \frac{960}{1} + \frac{4}{1.236} \quad \therefore Sw = 1.008$$

Volume of mixed raw sludge

(Primary + Activated sludge)

$$= 85 \times \frac{100}{4} \times \frac{1}{1.008} \times \frac{1000}{10^6}$$

$= 2{\cdot}11 m^3/1000$ persons/day

Sp. gr. of dry solids in digested mixed Pr. & A.S.

% V. M. in digested sludge

$$= \frac{20}{57} \times 100 = 35.09\%$$

% Non V.M. in digested sludge

$$= \frac{37}{57} \times 100 = 64.91\%$$

$$\therefore \frac{100}{S_d'} = \frac{35.09}{1} + \frac{64.91}{2.5} \quad \therefore S'_d{}' = 1.64$$

Percentage solids in digested sludge $= 7\%$

$\therefore$ Sp. gr. of wet solids

$$= \frac{100}{S'_w} = \frac{93}{1} + \frac{7}{1.64}$$

$$\therefore \; S'_w = 1.028$$

Volume of digested mixed sludge

$$= 57 \times \frac{100}{7} \times \frac{1}{1.028} \times \frac{1000}{10^6}$$

$= 0{\cdot}8 m^3/1000$ persons/day.

Assuming a parabolic reduction of volume and a digestion period of 30 days at 27° C (vide Table 16·2) and 60 days storage in monsoon, capacity of digester required vide equation 16—3

$$V = [V_f - \tfrac{2}{3}(V_f - V_d)]T_1 + V_d T_2$$

$$= [2.11 - \tfrac{2}{3}(2.11 - 0.8)]30 + 0.8 \times 60$$

$$= 37.11 + 48 = 85.11 m^3/1000 \text{ persons.}$$

Vol. of digester needed for 1·50 lakh persons—

$$= \frac{85.11 \times 1{,}50{,}000}{1000} = 12{,}767 m^3 \text{ say } 12{,}800 m^3$$

(with in 08.0—0.15 m^3/capita for combined tludge)

Loading factor :

$$\text{Vol. solids} = \frac{58 \text{ kg/day}}{85.11} = 0.681 \text{kg/day/m}^3$$

(within the prescribed range of 0·3—0·75).

Dimensions of the digester:

Assuming a cylindrical digester, average gas production. $= 0.9 m^3$/kg volatile matter destroyed

Volatile matter destroyed in the combined sludge $=$ 38gms/cap

$$= \frac{38}{1000} \times 1{,}50{,}000 = 5700 \text{kg}$$

Gas produced $= 0{\cdot}9 \times 5700 = 5{,}130 m^3$

Min. area of digester required (to avoid foaming) $= \dfrac{2 \times 5130}{9} = .1140 m^2$

$$\therefore \quad \text{depth} = \frac{12{,}800}{1140} = 11 \text{m}$$

But depth should not exceed 9m. Hence 2 digesters are proposed

Adopting 8 m depths for the 2 tanks

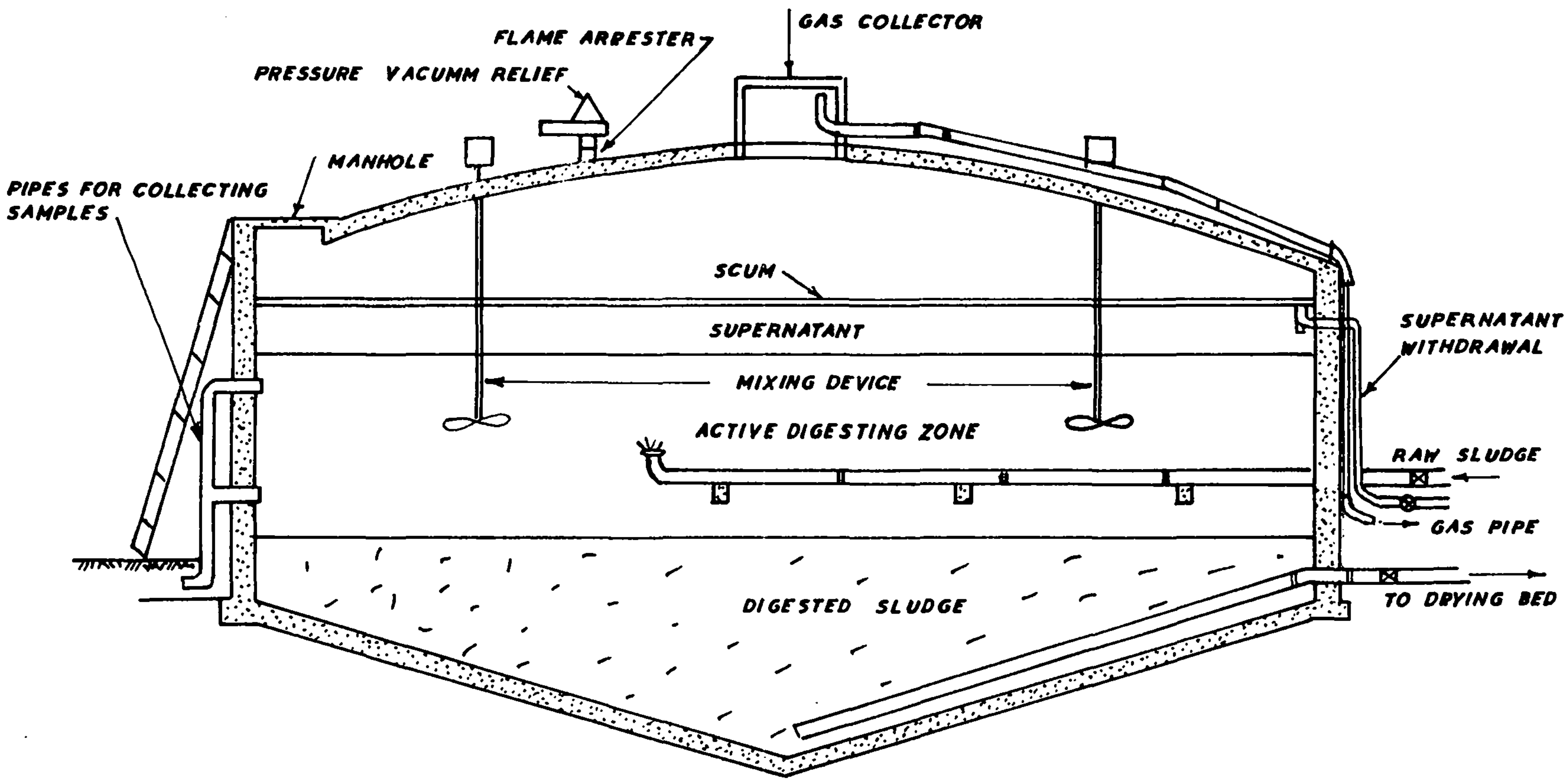
dia. of each digester

$$= \sqrt{\frac{12{,}800 \times 4}{\pi \times 2 \times 8}} = 31.9 \text{ m}$$

Provide a free board $= 0{\cdot}6$ m (for floating cover).

Adopt 2 numbers of 32 m dia × 8·6 m. height.

Typical details of sludge digester are given in Fig. A-18·1



SECTION

FIG. A 18·1 TYPICAL DETAILS OF SLUDGE DIGESTER

APPENDIX 19

Slude Thickening

Design a gravity thickener for thickening the combined primary and activated sludge from a treatment plant for 150,000 population

Solution :

Per capita settleable suspended solids in primary sludge . . = 54 gm/day

Per capita settleable SS in activated sludge . . . . . = 31 gm/day

$$\text{Wt. of combined sludge} = \frac{85 \times 1,50,000}{1000} = 12,750\text{kg/d}$$

Average solids loading . . =40 $kg/m^2/day$

Hydraulic loading required . =25 $m^3/m^2/day$

Specific gravity of wet mixed sludge =1.008

Assuming solids in combined wet sludge as 3%

Volume of sludge/day

$$= 85 \times \frac{100}{3} \times \frac{1}{1.008} \times \frac{1}{10^6} \times 1,50,000$$

$= 421.8\ m^3/d$ Say 420 m^3/d

$$\text{Area needed} = \frac{12,750}{40} = 318.7\ m^2 \text{ Say } 320 m^2$$

Flow needed for giving hydraulic loading of

$$25m^3/m^2/d = 320 \times 25m^3/day = 8000m^3/d$$

∴ Balance of (8000—420) =7580 m^3/day is made available by blending with primary or secondary effluent.

Assume a side water depth =3 m,

Sludge detention period

$$= \frac{V}{Q} = \frac{320 \times 3 \times 24}{420} = 54.9 \text{ hrs.}$$

(>24 hrs. O.K.)

Assuming a circular sludge blanket type thickener, dia of the tank

$$= \sqrt{\frac{320 \times 4}{\pi}} = 20.19 \text{ or Say } 20 \text{ m}$$

Sludge blanket restricted to 1 m is adopted

Expected solids in the thickened sludge =6%

35—480 M. of W&H/ND/79

APPENDIX 20

Sludge Drying Beds

Design a sludge drying bed for digested sludge from an activated sludge plant serving 150,000 people.

Solution

Solids in digested sludge (from mixed primary and activated) =57 gms/cap/day

Daily solids =1,50,000 × 57 = 85,50,000 gms = 8550 kg.

Adopting a dry solids loading of 100 kg/m²/year,

$$\text{area of bed needed} = \frac{8550 \times 365}{100} = 31210 \text{ m}^2$$

$$\text{Check for per capita area} = \frac{31,210}{150,000} = 0.21 \text{m}^2$$

(within the range of 0.175 to 0.25)

Adopting 8 m wide × 30 m long beds with single point discharge and a bed slope of 0.5%,

$$\text{the number of beds needed} = \frac{31,210}{8 \times 30} = 130$$

Assuming 2 months of rainy season in a year and 3 weeks for drying and one week for preparation and repair of bed, number of cycles per year would come to 10.

Volume of digested sludge (assuming 7% solids and a specific gravity of 1.025)

$$= 8,550 \times \frac{100}{7} \times \frac{1}{1.025} \times \frac{1}{1000}$$

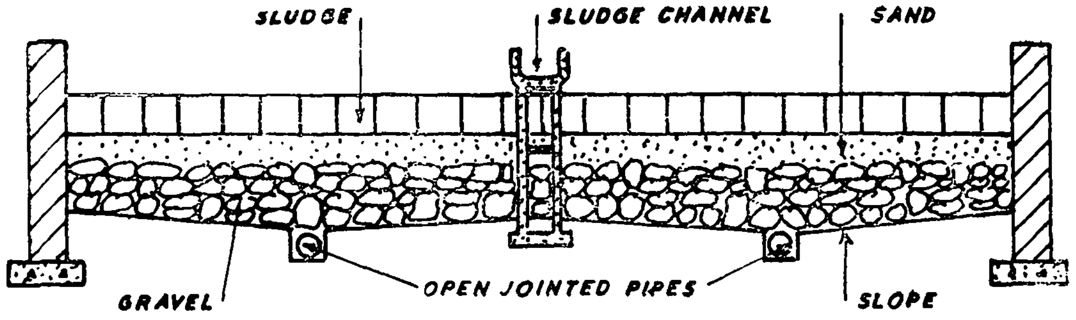
$$= 119.2 \text{ m}^3/\text{day}$$

say 120m³/day

Depth of application of sludge

$$= \frac{120 \times 365}{130 \times 8 \times 30 \times 10} = 0.140 \text{ or } 14 \text{ cm.}$$

Typical details are given in A-20.1



SECTION

FIG. A 20.1 TYPICAL DETAILS OF SLUDGE DRYING BED

APPENDIX 21

Septic Tank

Design a septic tank for a small residential housing colony of 100 users and also design the soil absorption system for disposal of the septic tank effluent. (Assume the percolation rate as 20 minutes per cm and the water table is below 180 cm from the ground level).

Solution :

From the Table 22.3 estimated peak discharge for residential housing colony of 100 users is 240 lpm. Sludge withdrawal will be once a year.

$$\therefore \text{ Surface area of the tank } = \frac{240 \times 0.92}{10}$$

$$= 22.08 \text{ m}^2 \text{ say } 22\text{m}^2$$

From 22.1.2.4. the total volume of the septic tank would be as follows :

1. For sedimentation (assuming 0.3 m depth) $22 \times 0.3 = 6.6 \text{ m}^3$
2. For digestion $0.032 \times 100 = 3.2 \text{ m}^3$
3. For sludge storage$(0.0002 \times 365 \times 100) = 7.3 \text{ m}^3$
4. For free board including provision for seed sludge 22×0.3 6.6 m^3

$$= 23.7 \text{ m}^3$$

$$\therefore \text{ Total depth of tank } = \frac{23.7}{22} = 1.08 \text{ m say } 1.1 \text{ m}$$

Providing length : breadth = 2.5:1

The tank dimensions are $7.5 \text{ m} \times 3.0 \times 1.1 \text{ m}$

Typical details are shown in the Fig A-21.1

Soils absorption system:

The total soil surface area required is

$$Q = \frac{130}{\sqrt{t}}$$

where, Q= maximum rate of effluent application in lpd/m² of leaching surface and t = standard percolation rate in min.

Since t = 20 min;

$$Q = \frac{130}{\sqrt{20}} = 29.07 \text{ say } 29 \text{ lpd/m}^2$$

Assuming 150 lpcd,

Total flow per day $= 150 \times 100 = 15,000$ lpd

Total area of trench required $= 15000/29 = 517.2\text{m}^2$ say 520 m²

Adopting a trench width of 1m, and a separation between trenches of 2m, total land area required $= 3 \times 520 = 1560$ m²

Provide 20 trenches, each 26 m long and 1 m depth Typical soil absorption system with dispersion trenches for above design is given in Fig. A-21.2

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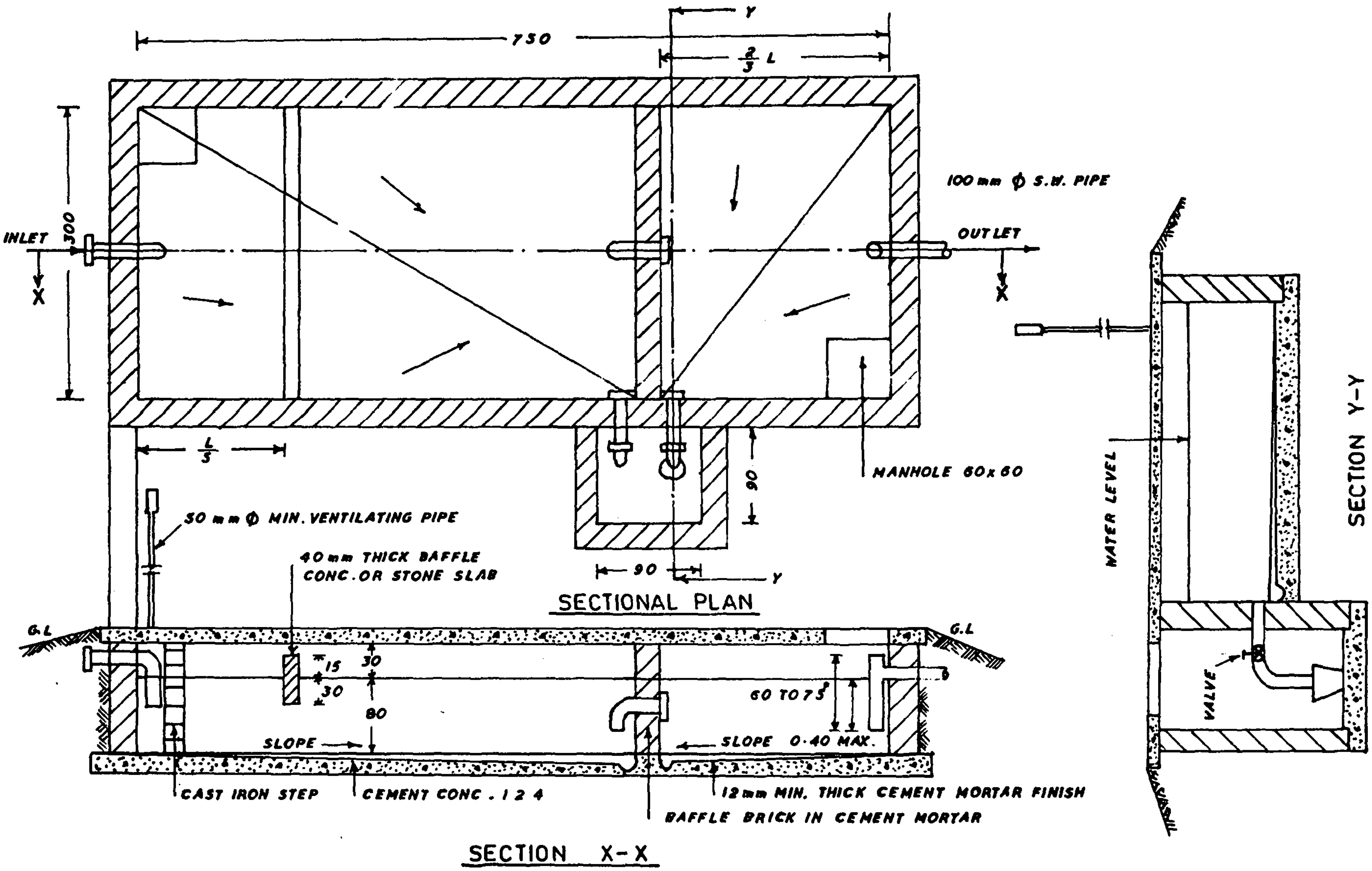
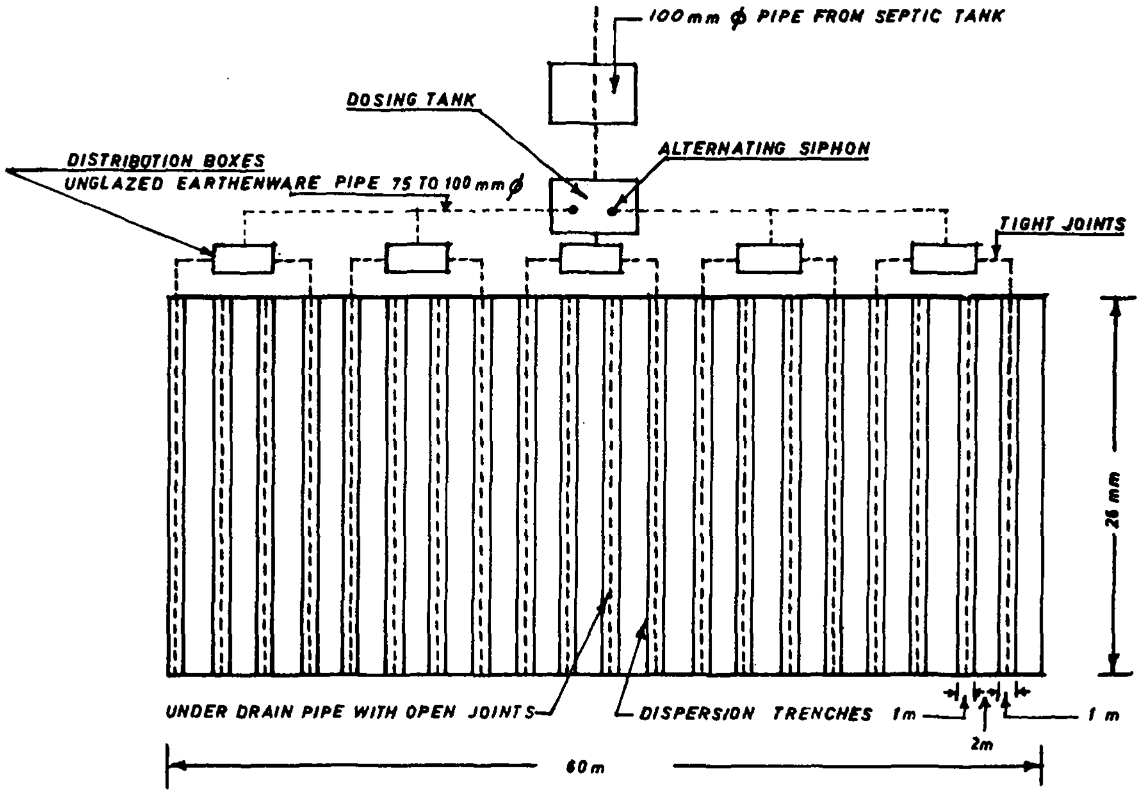
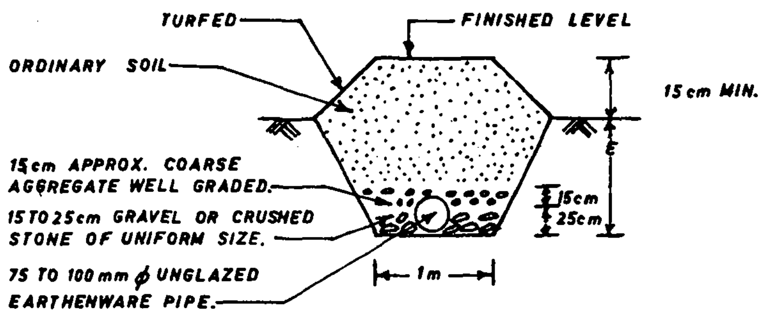

750
Y
2/3 L
100 mm ϕ S.W. PIPE
INLET
300
OUTLET
X
X
L/3
MANHOLE 60 x 60
90
50 mm ϕ MIN. VENTILATING PIPE
40 mm THICK BAFFLE
CONC. OR STONE SLAB
90
Y
SECTIONAL PLAN
G.L
G.L
15
30
30
80
60 TO 75
SLOPE
SLOPE 0.40 MAX.
CAST IRON STEP
CEMENT CONC. 1 2 4
12 mm MIN. THICK CEMENT MORTAR FINISH
BAFFLE BRICK IN CEMENT MORTAR
SECTION X-X
WATER LEVEL
VALVE
SECTION Y-Y

FIG. A-21-1



TYPICAL SOIL ABSORPTION SYSTEM WITH DISPERSION TRENCHES



ENLARGED SECTION THROUGH FILLED DISPERSION TRENCH

FIGURE: A-21.2

APPENDIX 22

Soil Percolation Tests

To design a suitable soil absorption system for disposal of effluent from septic tanks, percolation tests shall be carried out, on the proposed site for location of the absorption system, in the following manner.

Six or more test holes spaced uniformly over the proposed absorption field shall be made.

A square or circular hole with side width or diameter of 10 cm to 30 cm and vertical sides shall be dug or bored to the depth of the proposed absorption trench. The bottom and sides of the holes shall be carefully scratched with a sharp-pointed instrument to remove any smeared soil surfaces and to provide a natural soil interface into which water may percolate. The holes shall be filled for a depth of about 5 cm with loose material to protect the bottom from scouring and settling.

Before the actual readings for percolation tests are taken, it is necessary to ensure that the soil is given ample opportunity to swell and approach the condition it will be in during the wettest season of the year. This is done by pouring water in the hole upto a minimum depth. of 30 cm over the gravel and allowed to soak for 24 hours. If the water remains in the tets hole after the overnight swelling period, the depth of water shall be adjusted to 15 cm over the gravel. Then from a fixed reference point, the drop in water level shall be noted over a 30 min period. This drop shall be used to calculate the percolation rate.

If no water remains in the hole, at the end of 30 min period, water shall be added to bring the depth of the water in hole 15 cm over the gravel. From a fixed reference point, the drop in water level shall be measured at 30 min intervals for 4 hours, refilling to 15 cm level over the gravel as necessary. The drop that occurs during the final 30 min period shall be used to calculate the percolation rate. The drops during the earlier periods provide information for the possible modification of the procedure to suit local circumstances.

In sandy soils or other porous soils in which the first 15 cm of water seeps away in less than 30 minutes after overnigt swelling period, the time interval between measurements shall be taken as 10 minutes and the test run for one hour. The drop that occurs during the final 10 minutes shall be used to calculate the percolation rate.

Based on the final drop, the percolation rate, which is the time in minutes required for water to fall 1cm, shall be calculated.

37—480 M. of W&H/ND/79

APPENDIX 23

Operation Troubles in Sewage Treatment Plants

Signs & Symptoms	Possible Causes	Suggested Action
(1)	(2)	(3)
Pretreatment		
Unusual or excessive screenings.	Increase in domestic sewage or industrial waste.	Clean screens more often and report.
Excessive grit . .	Roadwashings, ashes or material from building site.	Report and get them diverted.
Excessive organic matter in grit.	Velocity is too low and detention period too long.	Reduce the cross-sectional area of the channel occupied by flowing sewage. Install planks, bricks or tile along sides of channel or reshape or repair outlet weir to proportionally reduce depth of flow for all normal present flow rates; or decrease the number of channels used; or reduce length of channel by moving outlet weir.
Carryover of grit . .	Velocity is too high and detention too short.	Remove grit more frequently or increase number of channels;or increase cross sectional area of channels.
Sedimentation Tank		
Floating sludge—in all tanks.	Accumulated sludge decomposing in the tank and buoyed to the surface.	Remove sludge more completly and more often.
Floating sludge—not in all tanks.	Affected tanks receiving too much sewage.	Reduce flow to affected tanks.
Bubbles rising in tanks .	Septic conditions.	Report and empty tank completely as soon as possible.
Contents black and odorous.	Septic sewage or strong digester supernatant.	Take action to eliminate septicity by improving hydraulics of sewer system, preaeration of organic industrial wastes admitted to the system etc.; or improve digester operation so as to have improved quality supernatant; or reduce flow into settling tank or bypass completely supernatant to lagoons etc. till situation improves.
Excessive settling in Inlet channels.	Velocity too low	Reduce cross-sectional area by installing inner wall of suitable material along one wall of channel; or agitate with air, water or otherwise, to prevent deposition.
Excessive suspended matter in effluent —all tanks	Accumulated sludge	Clean tanks more often.
	Flow through tanks too fast (over-loading).	Report and get the loading reduced.
	Humus sludge or under-drainage returned too fast.	Reduce pumping rate.
—not all tanks . .	Some tanks receiving too much sewage.	Reduce flow to affected tanks.
Execssive floating matter in the effluent.	Defective scum boards or none	Repair scum boards or install new ones.
Sludge pipes choke .	Sludge too thick	Clean more often.
	Sludge contains grit	Clean grit chamber more often; if chokage persists report. Change sludge piping, if necessary.
Intermittent surging of flow.	High intermittent pumping rates.	Adjust pumping rates to keep close to rates of flow or install or adjust baffling to reduce inlet velocity and to have effective flow dis tribution across the width of tank.
Sludge hard to remove from hopper	High content of grit and/or clay	Reduce grit content; or reduce clay-content; or rod the clogged lines.
	Low velocity in withdrawal line	Pump sludge more often; or change sludge piping.
Trickling Filters		
Filter ponding . .	Rock or other media too small or not sufficiently uniform in size. Organic loading excessive.	Rake or fork the rocks on film surface with light equipment; wash the filter surface with a stream of water under high pressure; or stop the distributor over the ponded area and allow continuous flow of sewage; or dose the filter with heavy application of chlorine (5 mg/l Cl_2 in filter influent) for several hours at weekly intervals or take the filter out of use for

(1)	(2)	(3)
		one day or longer to allow it to dry out or replace filter media if above methods do not succeed.
Filter Flies . . .	Develop most frequently in an alternate wet and dry environment.	Dose filter continuously not intermittently; or remove excessive biological growth; or flood the filter for 24 hours at weekly or biweekly intervals (it should be done at intervals frequent enough to prevent the fly completing its life cycle between floodings); or wash vigorously the inside of the exposed filter walls; or chlorinate the sewage (3 to 5 mg/1) for several hours at frequent intervals of 1 or 2 weeks; or apply DDT or other insecticides.
Odours . . .	Anaerobic decomposition of sewage sludge or biological growths	Maintain aerobic conditions in all units including sewer system or reduce accumulation of slime and biological growth; or chlorinate filter influent for short periods when flow is low or reduce unusually heavy organic loadings as from milk wastes.
Icing of Filter Surface .	Air temp. at or below 0° C; or progressive lowering of temperature of applied sewage by recirculation; or uneven distribution of sewage on filter.	Decrease number of times sewage is recirculated; or where two stage filters are used, operate filters in parallel with little or no recirculation; or adjust orifices etc., to improve uniformity of distribution over filter and to reduce spray effect; or erect a wind screen at the filter in the path of prevailing winds; or break up to remove ice frequently.
Activated sludge		
Change in sludge volume index	High soluble organic loads in sewage	Decrease aeration liquor suspended solids; or bulking of activated sludge may be controlled by proper application of chlorine to return sludge; or control sludge index by converting digested sludge to activated sludge.
Rising sludge (in settling tanks)	Due to excessive nitrification	Increase the rate of return of activated sludge from the final settling tank; or decrease the rate of flow of aeration liquor into the tank; or increase the speed of sludge collecting mechanism in the final settling tank to increase the rate of removal of sludge; or decrease nitrification by reducing aeration or lowering the detention period.
Frothing . . .	Synthetic detergents cause frothing. The froth increases with decrease in aeration liquor suspended solids or increase in aeration; or increase in degree of purification of sewage; or increase in atmospheric temperature.	Use water, effluent or clarified sewage sprays in the frothing areas; or apply defoamants in small quantities to tank surface (repeated dosing is necessary) or increase aeration liquor SS concentration.
Sludge Digestion		
Fluctuations in sludge temperature		Pump large quantities of thin sludge at high rate for cooling it.
Temperature drops in unit with hot water coils	Sludge solids adhering to coils forming a thick insulating layer preventing heat transfer to digester.	Clean the surfaces of coils or replace this form of heating with an external heat exchanger.
Temperature constant, gas production drops.	Increase in scum accumulation; or increase in grit accumulation; or excessive acid production or acid condition due to (a) organic over-loading (b) acid wastes (c) toxic metals, Cu, Ni, Cr & Zn.	Control scum; or control grit; or prevent excessive acid conditions by reducing organic overloads; or reduce acid wastes by pretreatment; or eliminate toxic metals or add lime to keep pH between 6·8 and 7·2; or proper quantity of over digested sludge should be withdrawn from digester.
Foaming . . .	Insufficient amount of well buffered sludge in the digester; or excessive additions of raw sludge (with high volatile content); or poor mixing of digester contents; or temperature too low for prolonged periods followed by rise in temperature of digester contents; or withdrawal of too much digested sludge; or excessive scum or grit accumulations.	Temporarily reduce or stop raw sludge additions; or add lime to keep pH between 6·8 and 7·2 while other corrective measures are undertaken; or restore good mixing within digester; or raise temperature to normal range; or breakup and remove excessive scum layer; or if large quantities of oil or grit are present, empty digester.
Sludge Drying Beds		
Sludge dries more slowly than usual	Sludge layer too thick Second dose applied too late. Standing water Bed surface clogged Broken or clogged drains	Put on less sludge. Do not apply second dose if first has started to dry off. Decant water. Rake over, skim if necessary and redress the surface. Set them right.

APPENDIX 24

Minimum Equipment needed for Tests

	Type of plant Small	Large		Type of plant Small	Large
1. Turbidimeter	—	√	18. Magnetic stirrer	√	√
2. Hot air oven	√	√	19. Sedgwick rafter funnel	—	√
3. Water bath with 6 to 8 concentric holes and discs, electrically heated	√	√	20. Foerst Centrifuge or equivalent	—	√
4. Muffle furnace	√	√	21. Microscope, binocular with oil immersion with movable stage	—	√
5. pH comparator	√	√	22. Counting cell for Microscope	—	√
6. pH meter-electrometric with glass, platinum and calomel electrodes	—	√	23. Membrane filter assembly	—	√
7. Photoelectric colorimeter	—	√	24. Colony counters	—	√
8. Orsat gas analysis apparatus or equivalent	—	√	25. 37 °C incubator	—	√
9. BOD Incubator	√	√	26. Autoclave	—	√
10. Refrigerator	√	√	27. Thermostatic water bath	—	√
11. Demineraliser	√	√	28. Vacuum pump	√	√
12. Chlorine comparator	√	√	29. Dissolved oxygen sampler	√	√
13. Kjeldahl digestion unit	√	√	30. Sludge sampler	—	√
14. Soxhlet extraction unit	—	√	31. Gas cylinder if gas supply is not available	√	√
15. Fume cupboard	√	√	*32. Gas Liquid Chromotograph	—	√
16. Analytical balance	√	√	*33. Atomic absorption spectrophotometer	—	√
17. Hot plates	√	√	* for special requirements		

APPENDIX 25

Tests

	Type of plant	
	Small	Large
1. Turbidity	—	√
2. Temperature	√	√
3. Colour	—	√
4. Alkalinity	√	√
5. pH	√	√
6. BOD	√	√
7. COD	√	√
8. Relative stability	√	√
9. Solids —total, suspended, dissolved —fixed and volatile.	√	√
10. Nitrogen —Kjeldahl, free and saline ammonia, albuminoid ammonia, nitrites, nitrates	√	√
11. Dissolved oxygen	√	√
12. Oxidation reduction potential	—	√
13. Chlorine demand and residual chlorine	√	√
14. Sludge volume Index	—	√
15. Volatile acids in sludge	—	√
16. Gas analysis	—	√
17. Total algal count	—	√
18. Microscopic analysis—qualitative for protozoa and rotifers	—	√
19. Bacteriological analysis—presumptive coliform, total bacterial count.	—	√
20. Special tests for oils and grease, copper, cyanide, zinc, chromium, lead, other heavy metals, sulphides, MBAS (Methylene Blue Active substances)	—	√

APPENDIX 26

Geometric Elements For Circular Channel Sections

$\frac{y}{d_0}$	$\frac{a}{d_0^2}$	$\frac{p}{d_0}$	$\frac{r}{d_0}$	$\frac{ar^{2/3}}{d_0^{8/3}}$
0·01	0·0013	0·2003	0·0066	0·0000
0·02	0·0037	0·2838	0·0132	0·0002
0·03	0·0069	0·3482	0·0197	0·0005
0·04	0·0105	0·4027	0·0262	0·0009
0·05	0·0147	0·4510	0·0326	0·0015
0·06	0·0192	0·4949	0·0389	0·0022
0·07	0·0242	0·5355	0·0451	0·0031
0·08	0·0294	0·5735	0·0513	0·0040
0·09	0·0350	0·6094	0·0574	0·0052
0·10	0·0409	0·6435	0·0635	0·0065
0·11	0·0470	0·6761	0·0695	0·0079
0·12	0·0534	0·7075	0·0754	0·0095
0·13	0·0600	0·7377	0·0813	0·0113
0·14	0·0668	0·7670	0·0871	0·0131
0·15	0·0739	0·7954	0·0929	0·0152
0·16	0·0811	0·8230	0·0986	0·0173
0·17	0·0885	0·8500	0·1042	0·0196
0·18	0·0961	0·8763	0·1097	0·0220
0·19	0·1039	0·9020	0·1152	0·0247
0·20	0·1118	0·9273	0·1206	0·1273
0·21	0·1199	0·9521	0·1259	0·0301
0·22	0·1281	0·9764	0·1312	0·0333
0·23	0·1365	1·0003	0·1364	0·0359
0·24	0·1449	1·0239	0·1416	0·0394
0·25	0·1535	1·0472	0·1466	0·0427
0·26	0·1623	1·0701	0·1516	0·0464
0·27	0·1711	1·0928	0·1566	0·0497
0·28	0·1800	1·1152	0·1614	0·0536
0·29	0·1890	1·1373	0·1662	0·0571
0·30	0·1982	1·1593	0·1709	0·0610
0·31	0·2074	1·1810	0·1755	0·0650
0·32	0·2167	1·2025	0·1801	0·0690
0·33	0·2260	1·2239	0·1848	0·0736
0·34	0·2355	1·2451	0·1891	0·0776
0·35	0·2450	1·2661	0·1935	0·0820
0·36	0·2546	1·2870	0·1978	0·0864
0·37	0·2642	1·3078	0·2020	0·0909
0·38	0·2739	1·3284	0·2061	0·0955
0·39	0·2836	1·3490	0·2102	0·1020
0·40	0·2934	1·3694	0·2142	0·1050
0·41	0·3032	1·3898	0·2181	0·1100
0·42	0·3132	1·4101	0·2220	0·1147
0·43	0·3229	1·4303	0·2257	0·1196
0·44	0·3328	1·4505	0·2294	0·1245
0·45	0·3428	1·4706	0·2331	0·1298
0·46	0·3527	1·4907	0·2366	0·1348
0·47	0·3627	1·5108	0·2400	0·1401
0·48	0·3727	1·5308	0·2434	0·1452
0·49	0·3827	1·5508	0·2467	0·1505
0·50	0·3927	1·5708	0·2500	0·1558
0·51	0·4027	1·5908	0·2531	0·1610
0·52	0·4127	1·6108	0·2561	0·1664
0·53	0·4227	1·6308	0·2591	0·1715
0·54	0·4327	1·6509	0·2620	0·1772
0·55	0·4426	1·6710	0·2649	0·1825
0·56	0·4526	1·6911	0·2676	0·1878
0·57	0·4625	1·7113	0·2703	0·1933
0·58	0·4723	1·7315	0·2728	0·1987
0·59	0·4822	1·7518	0·2753	0·2041
0·60	0·4920	1·7722	0·2776	0·2092
0·61	0·5018	1·7926	0·2797	0·2146
0·62	0·5115	1·8132	0·2818	0·2199
0·63	0·5212	1·8338	0·2839	0·2252
0·64	0·5308	1·8546	0·2860	0·2302
0·65	0·5404	1·8755	0·2881	0·2358
0·66	0·5499	1·8965	0·2899	0·2407
0·67	0·5594	1·9177	0·2917	0·2460
0·68	0·5687	1·9391	0·2935	0·2510
0·69	0·5780	1·9606	0·2950	0·2560
0·70	0·5872	1·9823	0·2962	0·2608
0·71	0·5964	2·0042	0·2973	0·2653
0·72	0·6054	2·0264	0·2984	0·2702
0·73	0·6143	2·0488	0·2995	0·2751
0·74	0·6231	2·0714	0·3006	0·2794
0·75	0·6318	2·0944	0·3017	0·2840
0·76	0·6404	2·1176	0·3025	0·2888
0·77	0·6489	2·1412	0·3032	0·2930
0·78	0·6573	2·1652	0·3037	0·2969
0·79	0·6655	2·1895	0·3040	0·3008
0·80	0·6736	2·2143	0·3042	0·3045
0·81	0·6815	2·2395	0·3044	0·3082
0·82	0·6893	2·2653	0·3043	0·3118
0·83	0·6969	2·2916	0·3041	0·3151
0·84	0·7043	2·3186	0·3038	0·3182
0·85	0·7115	2·3462	0·3033	0·3212
0·86	0·7186	2·3746	0·3026	0·3240
0·87	0·7254	2·4038	0·3017	0·3264
0·88	0·7320	2·4341	0·3008	0·3286
0·89	0·7380	2·4655	0·2996	0·3307
0·90	0·7445	2·4981	0·2980	0·3324
0·91	0·7504	2·5322	0·2963	0·3336
0·92	0·7560	2·5681	0·2944	0·3345
0·93	0·7612	2·6061	0·2922	0·3350
0·94	0·7662	2·6467	0·2896	0·3353
0·95	0·7707	2·6906	0·2864	0·3349
0·96	0·7749	2·7389	0·2830	0·3340
0·97	0·7785	2·7934	0·2787	0·3322
0·98	0·7816	2·8578	0·2735	0·3291
0·99	0·7841	2·9412	0·2665	0·3248
1·00	0·7854	3·1416	0·2500	0·3117

d_0 = diameter
y = depth of flow
a = water area
p = wetted perimeter
r = hydraulic radius.

APPENDIX 27

Bibliography

1. "Water and Wastewater Engineering" 1968 Vol. I—Water Supply and Wastewater Removal, Vol. II—Water Purification and Wastewater Treatment and Disposal, Fair G.M., Geyer J. C. and Okun D.A.

2. "Sewerage and Sewage Treatment" First Indian Edition 1960, Babbitt H.E. and Baumann E.R.

3. "Wastewater Engineering" 1972 Metcalf and Eddy Inc.

4. "Public Health Engineering Practice" Fourth Edition 1972, Vol. II—Sewerage and Sewage Disposal, Escritt L.B.

5. "Public Health Engineering—Design in Metric" 1970, Sewerage, Bartlett R.E.

6. "Design and Construction of Sanitary and Storm Sewers", Revised Edition 1970, Manual of Practice No. 9, Water Pollution Control Federation, Washington, D.C. U.S.A.

7. "Sewage Treatment Plant Design" 1959, Manual of Practice No. 8, Water Pollution Control Federation, Washington, D.C. U.S.A.

8. "Disposal of Sewage and other Water-Borne Wastes", Second Edition 1971, Imhoff K.M., Muller W.J. and Thistlewayte D.K.B.

9. "Wastewater] Systems Engineering" 1975, Parker H.W.

10. "Advanced Wastewater Treatment" 1971, Culp R.L. and Culp G.L.

11. "Sewage Treatment—Basic Principles and Trends", Second Edition 1972, Bolton R.L. and Klein L.

12. "The Disposal of Sewage" Third Edition 1956, Veal T.H.P.

13. "Sewerage and Sewage Treatment" Third Edition 1956, Hardenbergh W.A.

14. "Unit Operations of Sanitary Engineering" 1961, Rich L.G.

15. "Environmental Engineering and Sanitation", Second Edition 1971, Salvato J.A.Jr.

16. "The Design and Operation of Small Sewage Works" 1976, Barnes D. and Wilson F.

17. "Environmental Engineering Handbook" 1974, Vol. I—Water Pollution, Liptak B.G.

18. "Hand book of Applied Hydraulics" Third Edition 1969, Davies C.V. and Sorensen K.E.

19. "Open Channel Hydraulics" 1959, Ven Te Chow.

20. "Biological Treatment of Sewage and Industrial Wastes" 1957, McCabe J. and Eckenfelder W.W.

21. "Practical House Drainage" Revised Edition 1971. Southhall C.S. and Prentice H.O.

22. "Chemistry for Sanitary Engineers" 1967, Sawyer C.N. and McCarty P.L.

23. "Microbiology for Sanitary Engineers" 1962, Mckinney R.E.

24. "Waste Stabilisation Ponds" 1970, Design, Construction and Operation in India, Central Public Health Engineering Research Institute, Nagpur.

25. "Methods for the examination of water, sewage and Industrial Wastes" 1963, Indian Council of Medical Research.

26. "Sewage Treatment for Small Communities—Developments" 1977. Indian Association for Water Pollution Control. Technical Annual No. 4 (convention number), V. Raman.

27. "Upflow Anaerobic Filter—A simple sewage Treatment Device" 1978., International Conference on Water Pollution Control for Developing Countries, Bangkok, V. Raman and A.N. Khan.

28. "Process Design Manual for Upgrading Existing Wastewater Treatment Plants" 1974, Technology Transfer, U.S. Environmental Protection Agency.

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