ENGINEERING STANDARD

FOR

SEWERAGE AND SURFACE WATER DRAINAGE SYSTEM

0. INTRODUCTION

This Engineering standard " SEWERAGE AND SURFACE WATER DRAINAGE SYSTEM" is prepared to be used as a guidance on the planning and design of collecting net work to convey sanitary sewage and surface water of residential areas preferably in separate, combined or semicombined systems up to a sewage treatment plant or other final place of disposal.

Due to its broad scope, this Standard is presented in two parts:

PART ONE : **SURFACE WATER DRAINAGE**

PART TWO : **SANITARY SEWERAGE COLLECTION AND DISPOSAL**

Note:

Use of metric units is obligatory.

PART ONE

SURFACE WATER DRAINAGE

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1. SCOPE

This engineering Standard sets out general design considerations for predicting the magnitude of peak flows of surface water, generated within an urban area during rainfall plus normal and incidental surface water flows from different sources that finds its way into the surface water drainage* system or surface water sewerage* system.

2. REFERENCES

Throughout this Standard the following standards and codes are referred to. The edition of these standards and codes that are in effect at the time of publication of this Standard shall, to the extent specified herein, form a part of this Standard.

IPS (IRANIAN PETROLEUM STANDARDS)

IPS-E-CE-390	"Rain and Foul Water Drainage of Buildings"
IPS-E-CE-400	"Sewage Treatment"
IPS-E-PI-100	"Plant Piping Systems"
IPS-E-SF-400	"Fire Protection of Stairs, Ladders & Platforms"
IPS-M-SF-325	"Personal Protective Clothing and Anti Acid Clothing"
IPS-M-SF-302	"Masks and Breathing Apparatus"

*** For definitions and field of application of "surface water drainage and surface water sewerage systems " see clauses 3.10 and 3.11 respectively.**

3. DEFINITIONS AND TERMINOLOGY

3.1 Surface Water

Natural rainwater from the ground surface, paved areas and roofs plus occasional courtyard and car washing waste waters and incidental fire fighting water. The cooling water effluent of air-conditioning units, if any, should be included.

3.2 Storm Water

Rainwater discharged from a catchment area as a result of a storm.

3.3 Run - off

That part of rainfall which flows off the surface to reach a surface water sewer (conduit), an open channel or river.

3.4 Catchment Area

The area of a watershed discharging into a surface water drainage system, or water course.

3.5 Flow Attenuation

The process of reducing the peak flow rate in a surface water drainage system by redistributing the same volume of flow over a longer period of time.

Note:

This can be achieved by on or off line storage within the surface water network, or by above ground storage before flows enter surface water sewer or nullah network.

3.6 Nullah

An open channel with invert slab and side slopes of different sizes and degrees built in the residential areas of oil industries in south of Khuzestan.

3.7 Percentile Peakedness

The departure from the mean value of a storm profile, expressed as a percentage.

3.8 Rainfall

Natural precipitation of water in any form such as rain, snow, hail, etc., the rate of which is measured in millimeters of water per hour.

3.9 Roughness Value (K_S)

A measure of the resistance of the surface of a pipe or channel under turbulent flow which is expressed in millimeters.

Note:

This value is based on the diameter of uniformly graded sand grains which would give the same resistance to flow.

3.10 Surface Water Drainage

An open channel drainage systems beyond curtilage that conveys the rainfall of the roofs of buildings and run-off from paved and unpaved areas plus any other used water(not clean but not foul either) into a watercourse, a soakaway or to a storage container.

Note:

The drain pipes conveying the rainwater of roofs out of buildings and houses are prescribed in engineering standard IPS-E-CE-390.

3.11 Surface Water Sewerage

An underground piped system for drainage of surface water i.e. surface water sewers that convey run-off generated within an urban area during rainfall for safe discharge into a receiving watercourse.

3.12 Gully

A gully usually incorporates a trap, or a sump, or both, to retain detritus. The trap should be fitted with either a grating or a sealed cover. Connections should be made below the grating or cover.

3.13 Gully Gratings

Grating and, where provided, frames should be suitable for the gullies with which they are to be used and should be capable of supporting the loadings likely to be encountered in use without excessive deformation or failure. Cast grey and ductile iron and steel road gully gratings and frames should comply with the relevant clauses of IPS-M-CE-345.

4. SURFACE WATER SEWERAGE

Surface water sewers as defined in clause 3.11 are designed mostly to collect and convey run-off generated within an urban area during rainfall in an underground piped network for safe discharge into a receiving watercourse.

For hydraulic design methods and choice of appropriate method refer to BS 8005: part 1 clause 7. But for hydraulic design criteria of surface water sewers refer to clause 8 of part 2 of this engineering standard.

5. SURFACE WATER DRAINAGE

With due consideration to the definition of " surface water drainage" given in clause 3.10 the outfall of a surface water drain pipe beyond curtilage of the houses & building complexes separately or jointly should discharge into a surface water sewer, a combined sewer, an open channel surface water drainage network, or if neither is available, to a soakaway, a watercourse, or to a storage container.

5.1 Drainage to a Sewerage System

When a surface water drain is connected to a surface water sewer or open channel, an intercepting trap should not be provided.

Where the connection is made to a combined or partially separate sewer (which system is not recommended in this Standard) a trap should be provided in order to prevent sewer gas entering the surface water drain.

5.2 Disposal of Surface Water

In the absence of a surface water or combined sewerage system, disposal into soakaway, ditches, land drainage, canals, natural watercourses or other water bodies,e.g. ponds, lakes is desirable, if practical and permissible.

5.3 Drainage to a Soakaway

In the absence of a suitable sewer, and if it is not desired to conserve rainwater, it is often practicable to dispose of surface water to a soakaway. Where ground conditions are suitable and where it is desirable to maintain groundwater levels, soakaways should be considered. They also reduce the hydraulic loading on downstream sewers and watercourses.

A soakaway is a pit to provide soakage and storage capacity for run-off; its base and sides are open-jointed to facilitate the percolation of water into the surrounding subsoil. The pit can be filled with rubble, or a roof slab can be provided. If rubble is used, extra capacity should be provided to allow for this.

When the subsoil conditions are satisfactory, soakaways can be used to take the run-off from new development. Individual soakaways can be provided at each gully, downpipe,etc., or a number of drainage points can be connected to one soakaway. Particular attention should be paid to the provision of oil spillage interception, if any, in order to safeguard groundwater quality and also to protect the performance of the soakaway. In ground with low permeability, it is necessary to provide storage capacity to retain the flows during prolonged or heavy rainfall. A capacity equal to 12 mm of rainfall over the area drained should be adopted. Its effective depth is measured below the invert of the lowest incoming drain. This can be achieved by the provision of one soakaway or a number linked at overflow level by piped seepage trenches. Similar trenches can be used to provide means of overflow from a soakaway.

If drainage to a soakaway is to be adopted, the subsoil and the general level of the ground water should be investigated. A soakaway is not desirable nearer to a building than about 5 m, nor in such a position that the ground below foundations is likely to be adversely affected.

Small pit soakaways may be unlined and filled with hard-core for stability or the soakaway may take the form of seepage trenches following convenient contours. Larger pits may be unfilled but lined, e.g. with brickwork laid dry, jointed honeycomb brickwork, perforated precast concrete rings or segments laid dry, and the lining surrounded with suitable granular material. An unfilled pit should be safely roofed and provided with a manhole cover to give access for maintenance purposes. A soakaway can be used most effectively in pervious subsoils, such as gravel, sand, chalk or fissured rock, and where it is completely above the water table.

Seasonal variations in the water table may necessitate an increase in the storage capacity.

5.4 Drainage to a Watercourse or other Water Body

Where surface water is discharged to a nearby ditch, stream, river,canal, pond or lake (see 5.2), the invert level of the outfall should be approximately 150 mm above the normal water level.

Where periodic backflooding is likely to occur and it is not practicable to discharge at a higher level, a non-return valve should be fitted.

The outfall should be so formed as to avoid, or provide protection against, local erosion. It may be necessary to provide additional protection to the outfall opening to prevent damage or interference.

5.5 Drainage to Storage Containers

Where it is required to conserve it for use, water from paved areas or roofs may be drained to a storage cistern which should be securely covered, with provision for inspection and cleaning, and provided with an overflow to a nuisance-free outfall.

5.6 Flow Balancing

Where high rates of discharge of surface water occur, it may be necessary to provide a retention tank or balancing pond to intercept and hold back temporary peak storm discharges in order to avoid flooding. Suitable arrangements should be made for the maintenance and safety of the tank or pond and the surrounding area.

5.7 Determination of Flow: Surface Water

5.7.1 General

If an area is sewered on the separate system, all foul water should be excluded from the surface water drains. If sewered on the combined or the partially separate system, and the new development is to be similarly served, the appropriate drains should provide capacity for foul and surface water (see 5.7.2 to 5.7.8)

5.7.2 Flat rate of rainfall

The method adopted for the assessment of the peak rate of discharge of surface water from building development will vary according to the area and type of development. For areas which require a main surface water drain up to 200 m in length, a uniform rate of rainfall intensity may be adopted.

If mean rate of rainfall is not readily available, a rate of 50 mm/h can safely be used for small areas. However it is recommended to get the highest recorded rainfall intensity in ten years from meteorological authorities. The whole of the rainfall on impervious areas should be assumed to reach the drains. The impervious areas should include the horizontal projection of the roof areas, paved areas and half the area of the exposed vertical face of tower buildings.

For larger areas the peak rate of discharge should be assessed as described in 5.7.3 to 5.7.8.

5.7.3 Wallingford rational method

When a design based on a uniform rate of rainfall is not appropriate, the use of the, Wallingford Rational’ method for design of storm sewers or open channel canals is recommended for catchment areas of less than 150 hectares where the time of concentration is up to 30 min. and the pipe size is up to 1m.

This method of Wallingford Rational Method is a development from the Lloyd-Davies formula (see clause 7 of Appendix B of BS 8005, part 1).

The Lloyd-Davies formula states that:

$$Q = A_p \times i \times C_r \times C_v \times 2.78$$

Where:

Q	is the rate of run off (in L/s);
A_p	is the contributing impermeable area (in ha);
i	is the mean rate of rainfall (in mm/h);
C_r	is the dimensionless routing coefficient;
C_V	is the volumetric run-off coefficient.

This formula enables the calculation of run-off to reflect the nature of the catchment and substrata since recent research has shown that the volume of run-off is related to soil type as well as impermeable area.

In order to determine the rate of flow in a drainage system it is therefore necessary to know the intensity of rainfall to apply corresponding to the return period chosen, the impermeable area contributing to each part of the system and the appropriate coefficients.

Note:

For larger than 150 hectar catchment areas, the residential land can be divided into zones, specially in flat lands, and separate drainage systems be designed to avoid construction of very large rectangular channels and or " Nullahs".

5.7.4 Intensity of rainfall

It is assumed that:

a) the variation in the rate of rainfall during storm and in the volume of water retained in the drainage system may be neglected;

b) the maximum discharge of stormwater from an area occurs when the duration of the storm is equal to the time of concentration, t, of the area. The time of concentration is the longest time taken for the rain falling on the area to reach the drain plus the time taken to travel to the point under consideration,i.e.:

$$\textbf{t = time of entry} + \frac{\textbf{length of drain}}{\begin{array}{c}\textbf{velocity of flow in main drain}\\ \textbf{pipe or canal when running full}\end{array}}$$

c) the time of entry may be regarded as representing the delay and attenuation of flow over the ground surface.

The following values are recommended:

Return period	Time of entry (minutes)
5 years	3 to 6
2 years	4 to 7
1 year	4 to 8

For each return period the larger times of entry are applicable to large, flat sub-catchments (area greater than 400 m^2, slope less than 1:50) and the smaller values to small, steep sub-catchments(area less than 200 m^2, slope greater than 1:30).

Note:

These values of area and slope refer to the sub-catchments contributing to each pipe or shallow nullah length.

Rates of rainfall for any urban or rural area in Islamic Republic of Iran can be obtained from the Meteorological Organization.

Volume 4 of the Wallingford Procedure contains a manual method of calculation and the computer version of the methods with the rainfall data (see 5.7.3).

5.7.5 Impermeable area

For the purpose of calculations, the surface area contributing to flow in drains is normally taken as the area of paved surfaces connected to the drainage system, and unpaved areas are assumed not to contribute, except areas with clayey stratum like south of khuzestan.

5.7.6 Dimensionless routing coefficient, C_r

The value of the routing coefficient should, theoretically, vary with the shape of the time-area diagram and the shape of the rainfall profile. For use with the ' Wallingford Rational' method (see 5.7.3) a constant value of 1.3 is recommended.

5.7.7 Volumetric run - off coefficient, C_V

The volumetric run-off coefficient may be defined as the proportion of rainfall on the paved areas which appears as surface run-off in the storm drainage system. The coefficient ranges from about 0.6 on catchments with rapidly-draining soils to about 0.9 on catchments with heavy soils. These values reflect the loss of some rainfall from impervious areas through cracks and into depressions and by drainage onto pervious (unpaved) areas. Similarly, any run-off from pervious areas onto the impervious areas is also incorporated.

Alternative methods of determining C_V to take account of specific soil characteristics and regional variations in catchment wetness are described in Volume 1 of the Wallingford Procedure (see 5.7.3).

5.7.8 Method of calculation

The procedure described in items (a) to (g) should be adopted.

a) Prepare a key plan of the proposed drainage network, to identify the trunk and branch lengths of the system. For planning see clause 5.14.

b) Determine the impermeable areas contributing to each length of drain.

c) Select a pipe size or canal size in order to determine the time of flow through the drainage channel or pipe

Note:

For pipe drains the Hydraulics Research Station's Tables for the Hydraulic design of pipes' are recommended. Refer to clauses 10 and 11 of Appendix B of BS 8005, Part 1.

Take the time of concentration as the cumulative time of flow plus a time of entry (see item(c) of 5.7.4)

d) Select the rate of rainfall corresponding to the time of concentration and chosen return period of storm from the appropriate Meteorological Office data.

e) Select the volumetric run-off coefficient.

f) Calculate the expected peak flow using the above formula.

g) Check the chosen pipe size or canal size, change if obviously too large or too small, and repeat steps (c) to (e).

This procedure is applied to each length throughout the system.

A full explanation of the method, including a pipe numbering system and determination of impermeable areas, rates of rainfall and volumetric run-off coefficient is given in Volume 4 of the Wallingford Procedure. For simple calculations see clause 5.8

5.8 Design of Open Drain Channel or Nullah - Simple Method

For the design of open drain channels the simple Chezy formula may be used:

$$V = C \quad R.S \quad or, \quad S = \frac{V_2}{C^2.R} \quad \text{or} \quad V = c\sqrt{RS}$$

Where:

V = velocity (m/s)
C = chezy coefficient in m/s
R = hydraulic radius (hydraulic mean depth) in (m)
S = gradient (slope) of invert slab of channel

The value C in graphs given in Fig. A.1 of Appendix A of IPS-E-CE-380 part 1 is related to the material being used, size of the channel etc. and is based on the Bazin formula:

$$C = \frac{87}{1+\frac{a}{R}}$$

a = a channel wall factor related to the material being used.

The hydraulic radius or hydraulic mean depth (R) is the relationship between the amount of liquid being conveyed and the contact area between this liquid and the inside of the channel.

$$\mathbf{R} = \frac{\textbf{cross sectional area of flow}}{\textbf{wetted perimeter}}$$

For a rectangular drain, R will be:

$$R = \frac{a ¢ b}{2a + b}$$

Complementary typical values of Manning's "n" in:

Q= (1/n) (AR $^{2/3}$ ***S*** $^{1/2}$ ***) and ' Chezy's "C" in V= C*** ^{p} $\overline{RS}$

are given in table A.1, A.2 and A.3 of Appendix A.

5.9 Flow Measurement Methods in Open Channels International Standards

a) Slope-area method-ISO 1070-1973.

The Slope - area method can be used with some degree of accuracy in channels with stable boundaries such as rock (or very cohesive soil) bed and sides, and in channels with relatively coarse bed material.

b) End-depth method for estimation of flow in rectangular channels with a free over fall- ISO 3847:1977

The International Standard ISO 3847 specifies a method for the estimation of sub-critical flow of clear water in smooth, straight, rectangular prismatic open channels with a vertical drop and discharging freely. By using the measured depth at the end, the flow in rectangular channels (horizontal or sloping) with confined nappe and un-confined nappe may be estimated.

c) Velocity-area methods for flow measurement in open channels ISO 748:1979

The International Standard ISO 748 specifies methods for determining the velocity and cross-sectional area of water flowing in open channels (with or without ice cover), and/or computing the discharge therefrom. For the principles of the methods of measurements see ISO 748.

d) Measurement of flow in tidal channels* -ISO 2425: 1974/Amd. 1982

The two different methods generally used for measuring tidal flow in tidal waterways are prescribed in ISO 2425 which can be used for estimating of seasonal flows of tidal rivers such as Arvand and Bahmanshir and to determine the limitations of discharge capacities of any other tidal water-course.

Note:

For full information on the ISO standards related to the subject of liquid flow measurement in open channels refer to ISO Book 16

*** A channel in which the flow is subject to tidal action.**

5.10 Connection of Gullies

A gully may be connected directly to a drain, to a suitably located inspection chamber or manhole, or to a soakaway. In the case of a drain connection, a gully having rodding access should be provided and the connection made by a junction or, if impracticable in the case of an existing drain, by a saddle.

For details of methods of making connections, see relevant standard drawings of Iranian Petroleum Industries.

5.11 Connection of Rainwater Pipes

A rainwater pipe may be connected to a drain or to a suitably located inspection chamber, manhole or gully, or to a soakaway. Where not otherwise available, access should be provided on or at the foot of the pipe in such a manner that rodding is practicable and adequate working space is available.

Where positioning of a rainwater pipe coincident with a structural foundation is unavoidable , the connection should be made with the minimum of changes in direction, and additional access points should be provided if necessary to facilitate effective maintenance.(See relevant standard drawings of IPI*).

5.12 Connections to Public Sewers

An intercepting trap should not be provided for a connection to a surface water sewer unless required by the drainage authority.

5.13 Ventilation

Ventilation can usually be achieved by the direct connection of rainwater pipes or the use of untrapped gullies in suitable positions. (see relevant standard drawing of IPI*).

*** Iranian Petroleum Industries.**

5.14 Planning

Surface water drainage planning, should take account of the lifestyle of the people, limitations of land use, magnitudes of rainfall intensity and permeability of the catchment area.

In terrains with low permeability, the nullahs, apart from the function of draining, provide storage capacity that retain the flows during prolonged or heavy rainfall. However the cost of pedestrian and vehicle bridges built over nullahs would be higher versus those built over rectangular channels.

In ten meter wide streets or less, open drainage channels can be built on one side of the road only, thus reducing the overall costs

5.15 Design Criteria

The surface water drainage system should be able to handle either one of the following combinations of anticipated flows:

COMBINATION No. 1: - WET SEASONS

- The rate of run-off (Q) see clause 5.7.3.

- The rate of outpouring incidental water from burstage of water mains assumed to generate 20 m^3 /hr settled or potable water lasting for 2 hours.

COMBINATION No. 2: - DRY SEASONS

- Fire fighting water at a rate of 8 cubic meter per minute lasting for a duration of ten hours.

- Courtyard washing at a rate of 400 litres per residential house unit or 20 litres per square meter of paved courtyard per day.

- Car washing once a week at a rate of 100 litres per minute lasting for 15 minutes.

Note:

For design criteria of surface water drainage system of oil refineries and petrochemical plants see relevant clauses of Engineering Standard IPS-E-PI-100.

6. STRUCTURAL DESIGN

6.1 General

In the design and choice of the shape of the passage of surface water drainage schemes the following factors should be taken into consideration.

6.1.1 The topography of the land of the catchment area. Whether flat or steep.

6.1.2 The nature of the subsoil, whether corrosive (salty) or non-corrosive (sweet).

6.1.3 The level of the water table in different seasons.

6.1.4 The life style of the residents of the housing area.

6.1.5 The relative cost of the land that can be allocated for construction of surface water drainage passage routes versus other costs.

6.1.6 Whether the roads have trees and bushes along the drainage canals or not.

6.1.7 The ultimate place of disposal of surface water. Either a soakaway, a storage container or a watercourse. If the place of disposal is a watercourse, is it a non-tidal river, a tidal river, or a seasonal river.

6.1.8 Whether the solid rubbish, sweeped daily from the surface of the roads, is permitted to fall into the open drainage channels or should be disposed off together with household garbage separately.

6.2 Choice of Drainage Channels

With due consideration and evaluation of the above mentioned factors, the experienced engineering firms or owner's project managers can choose the most suitable shape of the open drainage channel.

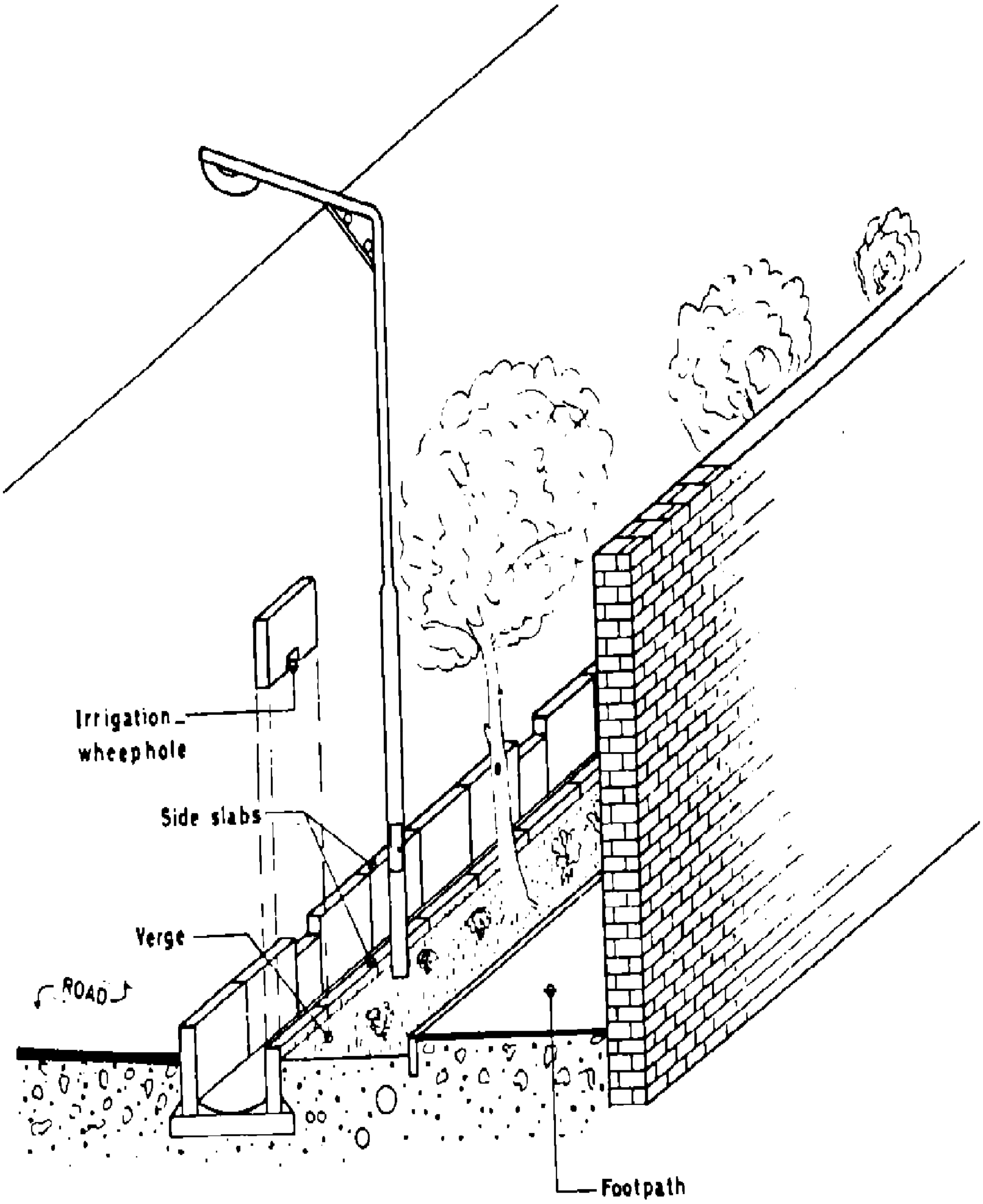
Normally in temperate regions of Iran the cross section of drainage channels should be rectangular in shape with different widths and depths as required, and are constructed in precast concrete slabs with nominal reinforcement or without reinforcement.

The cities of Iran mostly have trees on both side of the streets that are planted in the verge of pedestrian footpaths between the footpath curb or outer edge of the footpath and surface drainage channel.

Hence, the vertical concrete side slabs of the channel near the trees should have irrigation weep holes, provided at the bottom alternatively, as shown in Fig. 1.

Iranian Petroleum Standard Drawing No. S-...... should be used for construction of precast concrete side slabs and invert slabs of rectangular drainage channels applicable to moderate regions.

The cross section of the open surface water drainage channels in flat lands and wherever ample width of footpath verge can be allocated , at reasonable cost, can be trapezoidal in shape.



CROSS SECTION OF SURFACE WATER DRAINAGE CHANNEL (RECTANGULAR) IN A CONVENTIONAL STREET IN MODERATE REGIONS
Fig. 1

6.3 Lift Pumphouses

In flat lands, in order to avoid construction of rather wide & deep surface water drainage channels, installation of low head lift pumps in pumphouses built over major open channels (rectangular or trapezoidal) are recommended to deal with storm run-offs, particularly if the surface water is discharged into tidal rivers. Flap gate non-return valves or pen-stocks may be provided at the mouth of outfall into tidal watercourse.

6.4 Screening

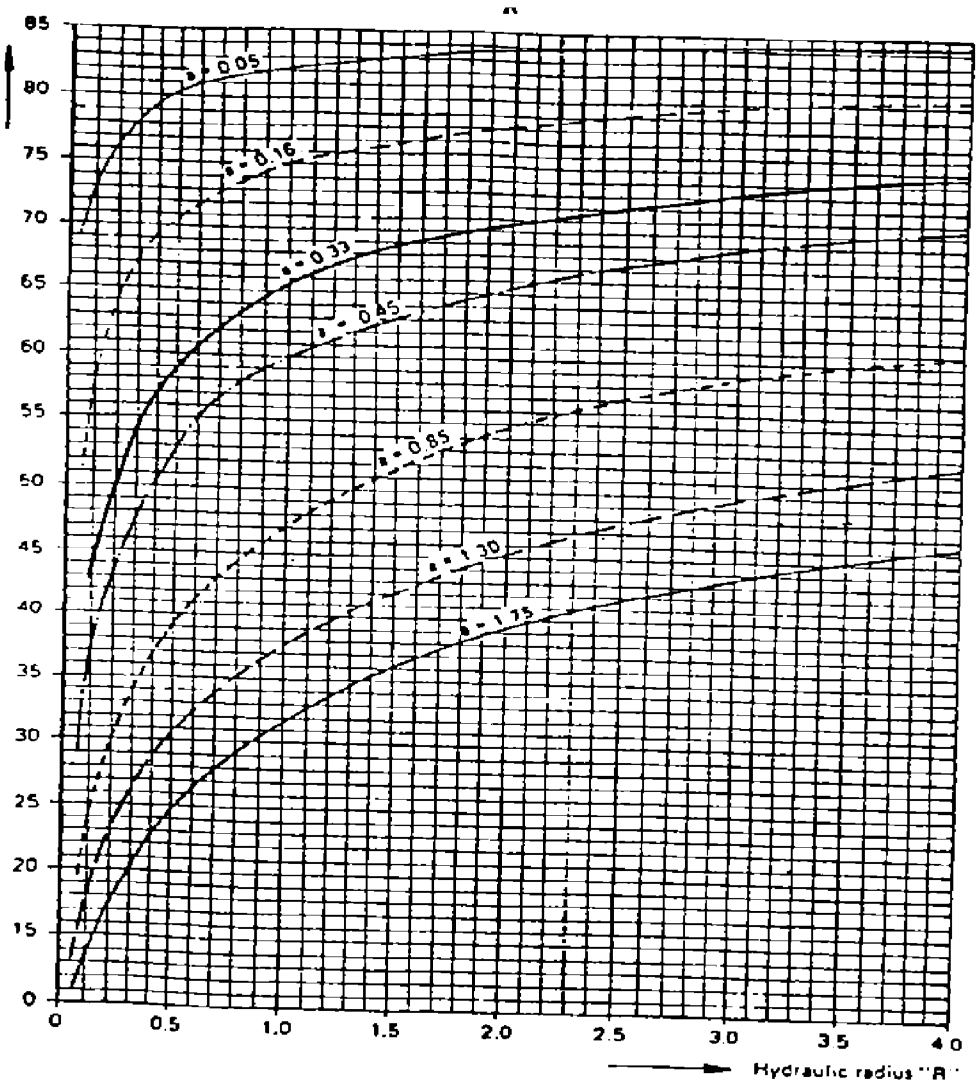
Wherever required, screens together with grabbing and hoisting facilities for removal of floating materials and sludge, should be provided adjacent to lift pumphouses, to prevent entrance of debris etc. into the pumps.

APPENDICES

APPENDIX A

'C' Values in graphical Curves based on Bazin's formula

$$C = \frac{87}{1+\frac{a}{R}}$$



a = 0.05 very smooth cement lining
a = 0.16 cement lining
a = 0.33 average concrete channels
a = 0.45 average masonry
a = 0.85 earth ditches with stone lining
a = 1.30 earth ditches straight and well maintained
a = 1.75 earth ditches average condition

Fig. A.1

TABLE A.1 - TYPICAL VALUES OF MANNING'S n IN

$$Q = (1/_n)(AR^{2/3}S^{1/2}) \text{ and Chezy's } C^* \text{ in } V = C\sqrt{RS}$$

TYPE OF CHANNEL	n	C (SI units)
Smooth timber	0.011	
Cement-asbestos pipes, welded steel	0.012	70-90
Concrete-lined(high-quality formwork)	0.013	60-75
Brickwork well-laid and flush-jointed	0.014	
Concrete and cast iron pipes	0.015	
Rolled earth: brickwork in poor condition	0.018	40-55
Rough-dressed-stone paved, without sharp bends	0.021	30-45
Natural stream channel, flowing smoothly in clean conditions	0.030	19-30
Standard natural stream or river in stable condition	0.035	14-25
River with shallows and meanders and noticeable aquatic growth	0.045	
River or stream with rocks and stones, shallow and weedy	0.060	
Slow flowing meandering river with pools, slight rapids, very weedy and overgrown	0.100	

*** For a full discussion of Chezy's coefficient C see An Introduction to Engineering Fluid Mechanics by J.A. Fox, published by The Macmillan Press, London, second edition, 1977.**

TABLE A . 2 - COEFFICIENTS FOR CHANNELS WITH RELATIVELY COARSE BED MATERIAL AND NOT CHARACTERIZED BY BED FORMATION

TYPE OF BED MATERIAL	SIZE OF BED MATERIAL mm	MANNING'S COEFFICIENT n	CHEZY'S COEFFICIENT C			
			R_n = 1 m	R_n = 2.5 m	R_n = 5 m	R_n = 10 m
Gravel	4 to 8	0.019 to 0.020	53 to 50	61 to 58	69 to 65	77 to 73
	8 to 20	0.020 to 0.022	50 to 45	58 to 53	65 to 59	73 to 67
	20 to 60	0.022 to 0.027	45 to 37	53 to 43	59 to 48	67 to 54
Pebbles and shingle	60 to 110	0.027 to 0.030	37 to 33	43 to 39	48 to 44	54 to 49
	110 to 250	0.030 to 0.035	33 to 29	39 to 33	44 to 37	49 to 42

TABLE A . 3 - COEFFICIENTS FOR CHANNELS OTHER THAN THOSE WITH COARSE BED MATERIAL

TYPE OF CHANNEL AND DESCRIPTION	MANNING'S COEFFICIENT n	CHEZY'S COEFFICIENT C			
		R_n =1 m	R_n = 2.5 m	R_n = 5 m	R_n = 10 m
A EXCAVATED OR DREDGED					
a) Earth, straight and uniform					
1 Clean, recently completed	0.016 to 0.020	63 to 50	72 to 58	81 to 65	91 to 73
2 Clean, after westhering	0.018 to 0.025	55 to 40	64 to 46	72 to 52	81 to 59
3 WITH short grass, few weeds	0.022 to 0.033	45 to 35	63 to 35	59 to 40	67 to 44
b) Rock cuts					
1 Smooth and uniform	0.025 to 0.040	40 to 25	46 to 29	52 to 33	59 to 37
2 Jagged and irregular	0.035 to 0.050	29 to 20	33 to 23	37 to 26	42 to 29
B Natural streams					
B.1 minor streams (too width at flood stage less than 30 m (100 ft.))					
a) Streams on plains clean,straight, full stage, no rifts or deep pools	0.025 to 0.033	40 to 30	46 to 35	52 to 40	59 to 44
B.2 Flood plains					
a) pasture, no brush					
1 Short grass	0.025 to 0.035	40 to 29	46 to 33	52 to 37	59 to 42
2 High grass	0.030 to 0.050	33 to 29	39 to 23	44 to 26	49 to 29
b) Cultivated areas					
1 No crop	0.020 to 0.040	50 to 25	58 to 29	65 to 33	73 to 37
2 Mature row crops	0.025 to 0.045	40 to 22	46 to 26	52 to 29	59 to 33
3 Mature field crops	0.030 to 0.050	33 to 20	39 to 23	44 to 26	49 to 29
c) Brush					
1 Scattered brush. heavy weeds	0.035 to 0.070	29 to 14	33 to 17	37 to 19	42 to 21
2 Light brush and trees (without foliage)	0.035 to 0.060	29 to 17	33 to 19	37 to 22	42 to 24
3 Light brush and trees (with foliage)	0.040 to 0.080	25 to 12	29 to 14	33 to 16	37 to 18
4 Medium to dense brush (without foliage)	0.045 to 0.110	22 to 9	26 to 10.5	29 to 12	33 to 13
5 Medium to dense brush (with foliage)	0.070 to 0.160	14 to 6.5	17 to 7.5	19 to 8	21 to 9
d) Trees					
1 Cleared land with tree stumps, no sprouts	0.030 to 0.050	33 to 20	39 to 23	44 to 26	49 to 29
2 Same as above, but with heavy growth of sprouts	0.050 to 0.080	20 to 12	23 to 14	26 to 16	29 to 18
3 Heavy stand of timber, a few felled trees, little undergrowth, flood-stage below branches	0.080 to 0.120	12 to 8.5	14 to 9.5	16 to 11	18 to 12
4 Same as above. but with flood-stage reaching branches	0.100 to 0.160	10 to 6.5	12 to 7.5	13 to 8	15 to 9
5 Dense willows, in mid-summer	0.110 to 0.200	9 to 5	10.5 to 6	12 to 6.5	13. to 7.5

PART TWO

SANITARY SEWERAGE COLLECTION AND DISPOSAL

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APPENDICES :

1. SCOPE

This Engineering Standard:

1.1 Prescribes the basic principles of different sewerage systems, the factors involved in the choice and layout of the sewerage scheme, the gradient and economical depth of the sewer mains with due consideration to the topography of the site and ground investigations. Guides for the hydraulic design of sewers with references to recognized standards as required.

1.2 Prescribes the components of sewer lines and drains, sewage pumping stations, rising or pumping mains that eventually convey the total crude sewage load of the selected sewerage system, to a sewage purification plant, or after semi-treatment, to outfalls at a sufficient distance from the shore ensuring its self purification by dilution method wherever the codes of practices would permit.

1.3 Sets out general design consideration for drainage of foul water of buildings and houses so as to convey and discharge the sanitary sewage beyond the curtilage of the property.

2. REFERENCES

In preparation of this Standard the following standards and publications have been considered.

- BS 8005 Sewerage parts 0 to 5: 1987

Part:0	"Introduction and Guide to Data Sources and Documentation"
Part:1	"Guide to New Sewerage Construction"
Part:2	"Guide to Pumping Stations and Pumping Mains"
Part:3	"Guide to Planning and Construction of Sewers in Tunnel"
Part:4	"Guide to Design and Construction of Outfalls"
Part:5	"Guide to Rehabilitation of Sewers"

- BS 8110 : 1985	Structural Use of Concrete
- BS 8301 : 1985	Building Drainage
- BS 5400 : Part 2:1978	Specification for Loads
- BS 8010 : 1987	Code of Practice for Pipelines

3. DEFINITIONS AND TERMINOLOGY

3.1 Wastes

The wastes produced by a municipality or self governed residential houses and offices of a corporation consist of sewage and municipal refuse or household garbage; the former is liquid, made up of the used water supply of the community, containing bodily discharges or excreta and industrial wastes; the latter comprises the solid wastes. garbage, rubbish, etc.

3.2 Sewage

Each of the liquid wastes named hereunder, or a combination of them, wholly or partially, are defined as sewage.

a) The liquid wastes drained away from residences and buildings.
b) The liquid wastes conducted away from industrial plants and establishments (trade shops etc.).
c) Such ground, surface and storm water as may be admitted to or finds its way into the conduits that carry the sewage.

3.3 Sewers

Underground conduits designed to carry sewage mostly by gravity and occasionally under pressure.

3.4 Drain

A pipe that takes foul sewage or surface water, or both, from a single building or from any buildings or yards appurtenant to buildings within the same curtilage.

Note:

A drain becomes a sewer when it leaves the curtilage of the property and is connected to the public sewerage system.

3.5 Foul Sewage

Water contaminated by domestic sewage, waste or trade effluent.

3.6 Manhole

A chamber constructed on a sewer so as to provide access thereto for inspection, testing, or the clearance of obstruction. It also receives branch sewers and drains.

3.7 Separate Sewer

A sewer designed to carry either foul sewage or surface water.

3.8 Combined Sewer

A sewer designed to carry both foul sewage and surface water.

3.9 Off - site Sewer

A foul, surface water or combined sewer, usually constructed as a public sewer, which carries the flow from a development site to a point of disposal.

3.10 On - site Sewer

A foul, surface water or combined sewer, usually built as a private sewer to be adopted later, which carries flows within a development site to an off-site sewer.

3.11 Partially Separate Sewer

A sewer designed to carry foul sewage and some storm water run-off usually from the rear roofs of properties.

3.12 Combined System

A drainage system in which both foul and surface waters are conveyed in the same pipe.

3.13 Separate System

A drainage system in which foul and surface water are conveyed by separate pipes.

3.14 Water Table

The level below which the ground is saturated with water.

3.15 Sewer Connection

Connection, that may be a saddle connection, of lateral pipe or drain to a sewer other than by a purpose made or pre-formed junction in the sewer run.

3.16 Sewer Junction

Purpose made or preformed junction pipe built into the sewer usually during construction.

3.17 Sewer Lateral

Pipe connected to a sewer.

3.18 Private Sewer

A pipe that carries foul sewage or surface water, or both, from one or more properties and which has not been adopted by the sewerage authority.

Note:

Private sewers may be owned by private individuals, companies or public organizations.

3.19 Public Sewer

A pipe that carries foul sewage or surface water, or both, from more than one property and that is vested in a statutory sewage authority.

Note:

For example, a regional water authority or municipality.

3.20 Rider Sewer

A sewer line usually laid under footpath parallel to branch sewers or mains that connects into it at convenient points.

3.21 Main Sewer

Increasingly larger diameter sewers accepting the branch sewers along the route toward pumping station.

3.22 Sewerage

A system of sewers and ancillary works to convey sewage from its point of origin to a treatment works or other place of disposal.

3.23 Trade Effluent

A fluid discharge, with or without matter in suspension, resulting wholly or in part from any manufacturing, industrial or commercial process, including farm, research institution, hospital and nursing home effluents.

3.24 Infiltration

Entry of groundwater into sewer.

3.25 Exfiltration

Escape of flow from sewer into surrounding ground.

3.26 Inverted Siphon

A pipe or conduit where the soffit drops below the hydraulic gradient and in which the sewage flows under pressure of gravity.

3.27 Nominal Size (DN)

A numerical designation of the size of a pipe, bend or branch fitting, which is a convenient round number approximately equal to manufactured dimension.

Note:

"Nominal bore" is the approximate internal diameter of a unit as declared by the manufacturer. This quantity is quoted with units (mm) whereas nominal size (DN) is quoted without units.

3.28 Trunk Sewer

Is a gravity sewer that receives the sewage load of a pumping station from the outlet of a rising main at different intervals and while gravitating, from a point at the perimeter of the next watershed area, toward the next pumping station (usually at the center of the catchment or watershed area), receives also branch sewers.

3.29 Dry Weather Flow (DWF)

When the sewage flow is mainly domestic in character, the average daily flow to the treatment works during seven consecutive days without rain (excluding a period which includes public or local holidays) following seven days during which the rainfall did not exceed 0.25 mm on any one day.

Note:

With domestic sewage from industrial premises the dry weather flow should be based on the flows during five working days if production is limited to that period. Preferably, the flows during two periods in the year, one in the summer and one in the winter, should be averaged to obtain the average dry weather flow.

4. ABBREVIATIONS

BOD	Biochemical Oxygen Demand
COD	Chemical Oxygen Demand
DN	Nominal Size
DWF	Dry Weather Flow
CF	Compaction Fraction

5. PLANNING

5.1 General

Sewerage planning and design should take account of the impact of changing land use and future developments within the catchment area and an appropriate population growth of the community that the sewerage scheme is planned to serve.

In planning and design of improved and extended sewerage systems, one should determine how best to rehabilitate and extend the existing system to serve future needs.

5.2 Design Periods and Sewage Flow Data

As in waterworks design the engineer must adopt periods of design for sewage works and make proper use of flow data.

5.2.1 Design of a sewer system

Period of design is indefinite as the system is designed to care for the maximum development of the area which it serves. It is necessary to estimate maximum population densities expected in various districts and locations of commercial and industrial districts together with maximum rates of sewage flow per second and maximum infiltration per day. For new developments, the estimates used, should be appropriate for a 20 year planning horizon.

5.2.2 Sewage pumping plant

Design period is usually 10 years. Rates of flow required are average daily, peak, and minimum flow rates, including infiltration.

5.2.3 Sewage treatment plant

Design period is 15 to 20 years. Flow rates required are average and peak rates, both including infiltration, as indicated in IPS-E-CE- 400.

5.3 Industrial Wastes - plant Drainage

Industrial liquid wastes and surface water contaminated with oil and strong solutions of organic and inorganic chemicals or suspensions of solid matter and or essentially clean but warm water, shall be discharged through separate system as prescribed in Iranian Petroleum Standard IPS-E-PI-100.

5.4 System Planning

5.4.1 General

System planning within a catchment area or river basin should consider the sewerage network, storm sewage overflows, pumping installations and the receiving sewage treatment works, and the effects of their discharges on receiving water courses.

Hydraulic performance of the whole system has to be considered to ensure that additions or modifications to the system do not result in overloading and/or premature operation of overflows. Where sewage is pumped consideration should be given to the effects of the pump discharge rates on the downstream parts of the system.

5.4.2 New developments (see also BS 8005 : Part 0)

When consideration is given to the allocation of land for development, the authority responsible for sewerage should be consulted regarding the constraints which may be imposed.

Site plans should include areas reserved for future routes for sewers including any necessary pumping stations and treatment plant, allowing sufficient room and access for construction and subsequent operation and maintenance. Sewers to serve new development should be constructed if possible within the right of way of the highways and roads.

Good public relations are important in the design, construction and operation of sewerage systems. Attention should be drawn to the necessity for close liaison between developers, land users, local authorities, statutory undertakers and others who may be affected by new sewerage schemes. Persons and organizations affected by the proposals should be consulted as early as possible in the planning stage and such liaison should continue for the duration of the work. Property owners affected should be made aware of the implications of the works proposed and of their own rights.

5.4.3 Sewerage rehabilitation

Where, a very high proportion of the population is served by existing public sewerage systems the objective has to be to optimize the existing system and to determine what adaptations or improvements are necessary to meet future needs.

BS 8005: Part 5, provides guidance on the procedures of rehabilitation of sewers, from the planning of the initial investigations to final completion, specially when in the upgrading of the structural and hydraulic performance of existing sewerage system, various options are available for appraisal.

6. BASIC DESIGN PRINCIPLES

6.1 General

The design of a sewerage system is influenced by the local topography, the extent and character of development, the existing and future flows from adjacent catchments, together with the suitability of the point of discharge of any proposed sewer, and the location and adequacy (both hydraulic and structural) of any existing sewerage system.

6.2 Sewerage Systems

6.2.1 The different systems

There are three different systems as follows.

a) The separate system (sometimes known as the totally separate system) carries foul sewage (domestic sewage and trade effluents) by an individual system of sewers to a sewage treatment works or outfall point.

Surface water from roads, footpaths, roofs and other paved areas is collected in an independent system of surface water sewers or open channels and discharged into convenient natural watercourses or soakaways, except in industrial areas where there is a high risk of pollution.

In separate domestic sewerage system of Iranian Petroleum Industries in south of khuzestan, the trade effluents permitted to enter into separate sewerage system are limited only to sanitary drains of hospitals and medicine centers. Hence trade effluents of any process plant or machine shop is categorized as industrial wastes.

b) The partially separate system is one in which foul sewage, together with some surface water, usually but not necessarily from roof or yard drainage, is collected by one sewer and the balance of the surface water is collected in another sewer or discharged direct to a watercourse or soakaway.

c) The combined system is a one pipe system which carries all the foul sewage and surface water to a sewage treatment works or outfall point

6.2.2 Choice of system

The sewerage system to be used depends on both technical and financial considerations, and on the requirement of the owner. In the Iranian Petroleum Industries, the separate system should be adopted in all new developments.

During the redevelopment of urban areas, or when sewers are being reconstructed, the opportunity may arise to construct a separate system to replace an existing system of combined or partially separate sewers, and this may be more economical than the provision of additional combined sewerage capacity.

The choice of a sewerage system depends on a number of factors for which refer to BS 8005, Part 1.

6.3 Site Investigations

6.3.1 General

Particular attention should be paid to the topography and the nature of the subsoil and water table. Special investigations are necessary if there is a possibility of the land to slide during deep pipe laying or other ground instability.

For more information refer to IPS-E-CE-110.

6.3.2 Existing services

The positions of all existing services should be ascertained as accurately as possible. The records of the relevant public and private bodies should be examined and consideration given to their possible inaccuracy. Exploratory holes should be excavated where necessary. The details of proposed roadworks and other development on the route of the new sewers should also be examined. For typical cross section arrangement of service lines such as mains and rider sewers, waterlines, cables etc., laid under roads, refer to Std. Dwg. No. S ......

6.3.3 Existing surface water drainage

The lines and levels of all existing drains, ditches and watercourses should be ascertained. When these are to be drained into a new surface water sewerage system, their drainage areas should be determined and surveyed. The relevant drainage authority should be consulted.

6.3.4 Ground water

The existence of ground water will be of particular importance and steps should be taken to identify ground water levels, including seasonal variations. Sampling and testing should be carried out to identify conditions which may be corrosive to materials used.

6.3.5 Infiltration

When the water table of the site is always high, the infiltration quantities become significant.

It may be possible to control infiltration by maintenance, renovation or elimination of wrong connections. This could alleviate the need for, or reduce the capacity of, additional sewers.

Notwithstanding the advances in pipe materials and jointing systems, some allowance for infiltration into completely new systems should be made where pipelines are laid below the ground water table.

6.4 Financial and Engineering Assessment

6.4.1 General

The financial and economic aspects of the various options open to the designer should be considered alongside the technical, operational, manpower, social, energy conservation and other factors before reaching a decision as to the preferred solution.

Both capital investment and operational costs should be evaluated on the basis of the best information available and options should be compared using cash flow discounting or other techniques as may be appropriate. Due consideration should be given to the accuracy of the various assumptions made in developing the scheme and the sensitivity of the future, including the effects of possible differential inflation and low operational cost.

6.4.2 Staging of schemes

Staged construction should be considered particularly when sewers can be laid at minimum depth or where lengths of pumping main are needed.

6.5 Layout of Sewerage Systems

6.5.1 Economy in the design of sewers and sewer connections

Where main sewers are laid at considerable depths or under main highways having expensive foundations and surfaces, it may be cheaper or more convenient to lay shallow rider sewers to receive the local house connections, and to connect the riders at a small number of convenient points into the main or branch sewer.

6.5.2 Gradients of sewers

The most economical design of sewers is obtained when they follow the natural falls of the ground. Sewers should, however, be laid at such gradients as will produce velocities sufficiently high to prevent the deposition of solid matter in the invert. For surface water sewers this can usually be achieved by ensuring that gradients are sufficient to give full bore velocities of at least 0.75 m/s. For foul and combined sewers this minimum velocity should be exceeded daily, which can be achieved by laying the sewers to a gradient which will give a velocity of 1.0 m/s at full bore flow. For maximum velocity see clause 8.6.

6.5.3 Pumping stations

Circumstances which may make the pumping of foul or surface water sewage either necessary or advisable include the following:

a) avoidance of excessive depths of sewer; specially when the land is flat and water table is high, necessitating design and construction of multiple pumping lift stations along the route of sewerage scheme toward final point of disposal.

b) the drainage of low lying parts of an area;

c) the development of area not capable of gravitational discharge to an adjoining sewerage system, a sewage treatment works or an outfall;

d) overcoming an obstacle,e.g. a ridge, a watercourse, a railway, or for avoiding an inverted siphon;

e) the provision of sufficient head for operation at a treatment works;

These points should be considered alongside the long-term energy commitments and the capital cost.

The output from a pumping station is normally irregular, particularly during the early life of the station when the flow in the sewers may be substantially below the design flow. The effects of this irregular flow should be taken into account in relation to the flow in downstream sewers and at any treatment works, the premature operation of overflows and the possibility of septicity of the sewage in pumping mains.

The discharge from a pumping station may, however, provide a beneficial flushing effect in the sewers. Site selection should take account of the possibility of flooding, limiting nuisance or damage caused by overflow in the event of mechanical or power failure and the effect on the general amenity of an area from noise or smell. Pumping operations may be concentrated at one point or occur at a number of strategic points on a drainage system. Alternatives should be considered and cost comparisons made with due regard given to the capital investment and operational costs which pumping will entail.

6.5.4 Vacuum sewerage

In a vacuum sewerage system a vacuum is induced and maintained in the pipe system by means of central vacuum pumps and a reservoir. The vacuum pulls sewage through the system to a central collection chamber where it is disposed of using gravity or conventional pumping. Pipes are of plastics and can vary in size from 75 mm to 200 mm in diameter.

6.5.5 Location of sewers

To allow for adequate access to a sewer for maintenance and to keep maintenance costs to a minimum, the following factors should be considered:

a) the position of other existing or proposed services;

b) the proximity of existing buildings and their foundations and the nature of the road construction, in relation to the depth and size of the proposed sewer;

c) the impact of the construction of the sewer and subsequent maintenance activities upon road users.

Where practicable, if the majority of development is along one side of a road, the sewers should be laid along that side. The duplication of sewers and the cost of laying these in the verge on each side of a main road should be compared with the cost of a single sewer with individual property connections from either side laid under the carriageway. Where sewers are located under the carriageway, this will leave verges and footpaths available for services which are more likely to require repairs.

However, if the sewer is at sufficient depth, it can be constructed in heading or tunnel to pass under existing services and this position will then only be affected by the requirements to locate manholes to avoid those services. In the case of new development, where the routing of sewers along roads may be difficult or produce an uneconomic layout, it may be possible to lay them under public footpaths or pedestrian areas linking different parts of the development.

When areas are being improved or redeveloped and the existing sewerage system is combined or partially separate, the possibility of changing the system should be considered, together with the possible replacement of old sewers, or their relocation under new or improved paved areas.

6.5.6 Septicity

Septicity caused by the prolonged retention of sewage under anaerobic conditions should be avoided by limiting the time of retention in rising mains and siphons and by the provision of self-cleaning velocities in all sewers. At temperatures of about 20°C sewage will change from fresh to stale in 2 to 6 hours, commencement of septic condition, with the time depending primarily upon the concentration of organic matter. The latter varies with per capita water consumption and infiltration.

Septicity of sewage leads to offensive smells and difficulties in treatment. It is mainly caused by the retention of organic solids for long periods in an environment where there is a shortage of free oxygen. It may occur in lengthy sewerage systems, and in siphons and pumping mains where sewage may be retained for many hours particularly during the night.

Septic sewage may produce gases such as methane and hydrogen sulphide. When mixed with air in certain proportions methane is highly explosive. Hydrogen sulphide is toxic in high concentrations and in part-full pipes may be converted by micro-organisms on the damp crown of the pipe into sulphuric acid. This acid can cause serious corrosion of work constructed with Portland cement.

Hydrogen sulphide is formed under anaerobic conditions at low velocity of flow and warm temperatures. The rate of release is increased at points of high turbulence such as backdrops and the outlets of inverted siphons and rising mains. Corrosion can occur to susceptible materials at these points and downstream of them.

Septicity in pumping or rising mains can be controlled by addition of oxidizing chemicals such as hydrogen peroxide but provision of air release valves or pipe stacks as vents is mandatory (see 11.6).

7. DESIGN OF GRAVITY SEWERS

7.1 General

7.1.1 Water consumption statistics

Existing water supply statistics should be used to derive future water supply consumption and hence sewage flows.

They will be invaluable in deriving flow patterns for daily consumption and indicating anticipated variations between different types of development. Figures for consumer wastage and distribution leakage are of particular importance in assessing sewage flows from such statistics. The overall capacity of the existing water supply system, together with projected additions, should be correlated with the capacity of planned sewerage schemes. Private water supplies obtained from boreholes, rivers, etc. that are likely to contribute to the sewerage systems should also be investigated.

7.1.2 Design flows: foul sewers

For foul sewers the dry wheather flow rates are usually based on either population and a rate of flow per head or, for new developments where such data may not be available, on the number of dwellings (see also clause 5.2.1).

The rate of flow per head may be based on local water supply statistics allowing for distribution losses and consumption that does not result in discharge to the sewers. Typical discharge figures for similar developments to those under consideration may also be used. In the absence of such data a figure of 220L (based on 200 L plus 10% may be assumed which multiplied by the population gives the average flow [dry weather flow (DWF)].

Foul sewers are frequently designed to carry 4 to 6 × DWF,the larger figure relating to sub-catchments and the smaller to the trunk sewers. This takes account of daily peaks and the daily and seasonal fluctuations in water consumption, together with an allowance for extraneous flows such as infiltration.

A figure of 4000 L per dwelling has been recommended for new development as an approximation to 6 × DWF. See Appendix B.3 of BS 8005, Part 1.

Where a scheme is to be developed in phases, consideration should be given to the likely flows following the initial stages of construction so that self-cleansing velocities are attained at times of peak flow each day (see 6.5.2).

Flow rates calculated from the DWF should not be used to design sewers that serve small and individual groups of buildings where flushes from individual appliances will give relatively high flows of an intermittent and irregular nature. In such circumstances the likely peak rate of flow should be derived from the probability study on the number and type of appliances connected. See IPS-E-CE-390, "Rain and Foul Water Drainage of Buildings".

7.1.3 Design flows: combined and partially separate sewers

For combined and partially separate sewers, the design flow rate is made up of the storm flow, which is by far the predominant component, plus an allowance for foul sewage flows. For more information refer to BS 8005, part 1.

7.1.4 Storm sewer overflows

The purpose of storm sewage overflows is to limit the quantity of storm sewage carried to sewage treatment works and to allow the discharge of diluted sewage to watercourses without causing undue pollution to the receiving water.

For further information see clause 9.4 of BS 8005: part one, section three.

7.1.5 Pumped discharge

Where a sewer receives a pumped discharge through a rising main it should be capable of receiving the output from all the pumps in the station discharging together at the total head that would apply if the wet well were filled to ground level. This excessive flow is one that can occur either:

a) upon power supply after a mains failure; or,
b) after repairs to the station if the attendant should inadvertently switch on all pumps whilst the wet well is full.

This is generally a problem with pumps that have a steep characteristic curve, as is normal with submersible type pumps.

7.2 Carrying Capacity of Sewer

Although the flow quantity of sewage in a gravity sewer running half full or full is the same (the maximum taking place at about 0.9 of the depth of its section), it is preferable to size the sewer for design flows of running half full to provide a safety factor in critical moments when run-off from surface drainage system may incidentally enter the separate sewerage system. However, foul water level in a sewer at low discharge periods should not drop below 0.3 of the sewer diameter.

8. HYDRAULIC DESIGN

8.1 Routing of Flow in Sewers

The flow in a sewer usually varies with time, particularly in a storm water sewer or in a foul sewer at the daily peaks. A discharge hydrograph (i. e. a discharge-time record taken at such a time would show that:

a) the discharge rises from a base flow to a peak and then decreases to the base flow;

b) assuming no lateral inflow, the peak discharge is less at successive locations along the sewer (this feature is termed attenuation);

c) the time for which the discharge is greater than the base flow increases at successive locations along the sewer.

The computation of the changes in the discharge hydrograph along the sewer is called "routing": the most suitable method for routing flow along sewers is that of Muskingum-Cunge which is used in the Wallingford Procedure of Appendix B.7 of BS 8005, Part 1.

8.2 Recommended Velocities in Sewers

8.2.1 Self cleansing velocities

Sewers should be laid at gradients that will produce velocities sufficient to prevent permanent deposits of solids. A velocity of 0.75 m/s occurring sufficiently frequently is usually enough to maintain self-cleansing conditions to avoid long-term deposition of solids.

8.2.2 Maximum velocities

The erosive effects of high velocities are not as serious as formerly had been thought. As per BS 8005, part 1 there is no need on this account to place an upper limit on the velocity in a sewer. Nevertheless, with due consideration of other circumstances it is recommended to limit the sewer velocities to 2 m/s and a maximum of 2.5 m/s be permitted for short runs only. (see also 8.6).

8.3 Hydraulic Roughness Value of Sewer Pipes and Pressure Mains

8.3.1 Velocity equation

Various equations have been developed in attempts to provide a scientific basis for the assessment of velocities of flow in channels and pipes.

The formula in use today include the following:

a) The Colebrook-White equation. This is most commonly used in the design of sewers, that provides an accurate basis of design, with experimental confirmation over a wide range of flow conditions. It is recommended as the first choice for design of sewers of the new developments.

b) The Manning equation and the Crimp and Bruges equation (Manning with a constant coefficient). The Manning equation is widely used because of its simplicity. A special form of this, the Crimp and Bruges equation, is also still widely used, but incorporates a constant hydraulic roughness value. Manning and Crimp equations would be described as empirical and their use is recommended for design of extensions to existing sewers.

c) Hydraulic Research Paper No. 4 equations. This publication recommends use of the Colebrook-White equation and provides tables to simplify application of the equation to the range of sewer pipe sizes, gradients and roughness commonly encountered. The equation is presented in engineering terms for both pipe-full and open channel flow; factors to allow its application to non-circular pipes are provided. Methods for evaluation of the composite roughness of surfaces of dissimilar texture are also described.

Although the Colebrook-White equation is complex, design charts[10] and tables [11] are available refer to Appendix B of BS 8005, part 1. The exponential equations (Manning and Crimp and Bruges) do not have any theoretical basis and only apply to a limited range of conditions. They can be used where the flow is turbulent and where circumstances require an algebraic analysis; their simpler form gives them an advantage over the Colebrook-White equation. When using exponential equations, the appropriate coefficient and constant have to be selected see Hydraulic research paper No. 1 of Appendix, B [12] of BS 8005, Part 1.

8.3.2 Roughness of sewers

In order to use a velocity equation, it is necessary to estimate the roughness of the sewer. Whilst in theory the amount of friction or resistance that must be overcome varies directly with the roughness of the inner surface of the pipe and hydraulic radius i.e. area of foul water stream divided by the wetted perimeter, in practice it is also influenced by other factors such as the straightness of the pipe, discontinuities at the joints, slime growths around the inner surface and sediment deposits on the invert.

In pipes carrying foul sewage, the roughness will be influenced to some extent by the pipe material, but will be primarily dependent on the slime that grows on the pipe surface below the water level corresponding to the maximum daily discharge.

A surface water sewer is unlikely to slime to any significant extent, but it is likely to contain deposits of grit on the invert. The roughness will depend on these and on the pipe material (see BS 8005, Part 1).

Recommended roughness values (K) for sewers are given in table 4.

TABLE 4 - ROUGHNESS VALUES FOR NEW, WELL - ALIGNED PIPES FREE FROM DEPOSITS

Material	Recommended values of k
	mm
Brick work	
Well pointed	1.5
Concrete	
Spun concrete pipes with "O" ring joints	0.15
Precast concrete pipes without "O" ring	0.30
Joints or having rough internal finish	
In situ lined concrete tunnels	0.60
Asbestos- cement	
With "O" ring joints, long pipe sections	0.06
Clayware (glazed or unglazed)	
With sleeve joints and "O" ring seals	0.03
With spigot and socket joints	0.06
Polymeric materials (GRP, PE, unplasticized PVC)	
With chemically cemented joints	0.03
With sleeve joints and "o" ring seals	

8.3.3 Roughness of pressure mains

Although a pressure main carrying sewage will slime, the amount and pattern of sliming will be different from that occurring in a gravity foul sewer. For hydraulics calculations, see Clause 11.4.

8.4 Energy Losses at Manholes and Bends

In addition to the energy losses that are induced by friction on the surface of the pipe, other losses will occur at manholes and bends as a result of sudden changes in velocity and in direction. These additional losses are usually small in relation to the frictional losses, and are not normally considered (see Appendix A of part Two).

The equation for the energy loss is usually of the form:

$$energy\ loss = \frac{kv^2}{2g}$$

Where:

- ***k*** is the energy loss coefficient;
- ***v*** is the mean velocity in sewer (in m/s);
- ***g*** is the gravitational constant (in m/s2).

8.5 Surcharging

A surcharged sewer is one where the sewage flows under pressure. This situation will arise when the incoming flow is greater than the capacity of the pipe when just full, or when the tailwater level is raised sufficiently to cause the sewer to run full when it would normally run only part full. The carrying capacity of a surcharged sewer depends on the hydraulic gradient. If the hydraulic gradient is steeper than the invert gradient, the capacity will be greater than the capacity when just full. Surcharging of inadequately sized sewers in order to increase their carrying capacity is undesirable specially in systems carrying foul sewage and in surface water systems where there is a danger of flooding. Also, it should be borne in mind that frequent surcharge of defective sewers will cause soil loss which could result in structural damage.

8.6 Sewers at Steep Gradients

Although there is criterion for limiting maximum sewage velocities (see Clause 8.2.2), however in steeply sloping ground, considerable saving in the cost of a sewerage scheme might result from laying the sewers at the prevailing ground slope. Smaller pipes can be used, and backdrop manholes can be eliminated.

For design guidances refer to clause 8.6 of BS 8005, Part 1.

8.7 Backdrop and Ramp Manholes

Where a sewer connects with another at a materially lower level, a backdrop or ramp manhole may be constructed to drop the higher incoming flow to the level of the lower sewer and a common outlet (see 13.2.5).

The choice between constructing a backdrop manhole or laying a sewer more steeply should be decided on economic grounds.

8.8 Inverted Siphons

Inverted shiphons pose a potential maintenance problem and whenever there is a practical alternative, sewerage designs should be arranged to avoid them. They are necessary, however, on occasions to carry sewage flows under obstructions such as rivers, canals, major roads and railways. Because there is normally more grit in suspension (and on the invert) in a combined or partially separate system than in a separate foul system, to achieve self-cleansing conditions the design of siphons on those sewers is more critical than for siphons on separate foul systems.

For design guidances refer to clause 8.9 of BS 8005, part 1.

8.9 Miscellaneous

8.9.1 Ventilation arrangements

Sewers should be ventilated to release sewer gases. Unventilated sewers may give rise to air locks or it may empty domestic siphon traps when the sewers are running full.

Sewers should generally be ventilated as follows:

a) at the head manholes in a gravity sewer system;
b) at about every 500 m on main sewers;
c) at pumping stations;
d) at manholes where pumping mains discharge.

Columns should rise to at least 900 mm above the top of the highest opening window and should be at least 3m away horizontally and preferably much more. The top of each column should be equipped with a wire cage.

8.9.2 Sewer flushing

Wherever possible, sewers should be designed to achieve selfcleansing conditions. Frequent sewer cleaning may be necessary on flat sewers where the gradient is insufficient to achieve selfcleansing velocities. Portable high pressure jetting equipment is found to be more convenient than flushing and better able to lift deposited grit.

9. DESIGN OF PUMPING STATIONS

9.1 General

The type and size of pumping stations and pumps depends on the pumping duties, the locations, whether the station will be attended, and the preferences of the user and designer.

The conventional pumping station has a dry well for pumps and other plant and a separate wet well, usually housing some of the control equipment. The roof of the dry well, which may extend partly or wholly over the wet well, should be above ground and flood level and serves as the floor of the superstructure building for motors and electrical equipment.

The building can include facilities for operators such as a toilet, messroom and store. At large stations it can also have a workshop and garage. Because of possible smell and noise problems it is not usually advisable to locate offices or amenity buildings at pumping stations.

It may be possible and necessary to construct a pumping station partly or wholly underground, for instance to deter vandalism, but this calls for special precautions in designing the substructure and in observing health and safety requirements.

A screw pumping station is used to discharge into a channel or gravity sewer and not into a pumping main. The motors and control gear should be housed and, if the screws are in the open, they should be provided with removable safety covers.

Small pumping devices, such as ejectors, may have an integral reception chamber and can therefore be installed in basements rather than in separate structures.

9.2 Health, Safety and Welfare Design Features

It is essential when designing sewage pumping stations and pumping mains to incorporate necessary health, safety and welfare features to comply with statutory requirements.

The scale of provision will depend on the numbers of staff and the frequency of visits to the station.

Typical hazards are as follows:

- **a)** falls of persons from heights, and into liquids or on to moving machinery;
- **b)** tripping or slipping on stairways, walkways or other means of access;
- **c)** falling or other traveling objects;
- **d)** inadequate levels of ventilation, particularly in confined spaces (see clause 9.9);
- **e)** combustion and explosion of flammable gases;
- **f)** electrical shocks and burns;
- **g)** faults in the installation and guarding of machinery (see BS 5304);
- **h)** excessive noise, vibration or fumes (see clause 9.9),

The following equipment should be provided, as appropriate:

- **1)** first aid and rescue equipment;
- **2)** emergency equipment and alarms;
- **3)** telephone and/or radio communication;
- **4)** toilet and washing facilities;
- **5)** facilities for the changing and storage of clothes and for the storage of tools and equipment;
- **6)** meal and office facilities.

For small pumping stations the design could provide for the use of a specially equipped vehicle to incorporate some of the above facilities.

9.3 Maximum and Minimum Pumping Rates

The maximum discharge rate from a pumping station, when all the duty pumps and pumping mains are in use, should be equal to, or preferably greater than, the maximum design rate of flow to the station. The minimum pumping rate should achieve a self-cleansing rate of flow in the pumping main(s). At a large station the minimum pumping rate may be governed by an assumed minimum flow to the station.

For a small station, with one constant speed duty pump, the pumping rate will be intermittent and may be unrelated to the rate of flow to the station.

Pumping will also be intermittent at multi-pumps stations whenever the flow to the station is less than the minimum pumping rate.

At medium and large stations, the station discharge can be kept approximately equal to the rate of flow of the incoming sewage by the adoption of variable speed pumps. This is not possible at small stations with constant speed pumps.

9.4 Pumping Heads

For a selected pumping rate total pumping head (or pressure) comprises the static lift, the friction in the pumping main, the friction through the pumps and station pipework and valves and the entry and exit head losses.

9.5 Number and Size of Pumpsets

The selection of the type of pumps, and their sizes and numbers depends, among other things, on the desired maximum and minimum pumping rates and on the need, or otherwise, to control the variations in the rate of discharge from the station.

A station with one constant speed duty pump should normally have a second pump to provide 100 % standby. This may be the most economical arrangement as far as pumping plant and electrical power is concerned, but it will result in intermittent discharge.

If the pumping main velocities are satisfactory, a station can have one variable or one two-speed duty pump and a similar standby pump. This would reduce the flow fluctuation but the electrical plant would cost more; it would be less efficient electrically than a constant speed installation.

To maintain acceptable velocities and reasonable friction losses, the individual suction and delivery pipe legs are, in many cases, larger than the pumps. The taper piece required on the delivery side should be included immediately at the pump branch before the non-return valve. Tapering on the suction side should be fitted between the sluice valve and the pump and should be of level soffit pattern. Tapers should be selected to give good velocity profiles particularly at inlet, and any bends should, where possible, be of long radius.

If the friction in a pumping main is significant, no more than two similar pumps should discharge simultaneously into a single pumping main. The additional output from a third pump into the same main could be quite small. If greater flexibility of discharge is desired, two sets of two duty pumps and one standby, each with its own delivery pumping main, might be appropriate. When the amount of storm water is significant one set might use larger pumps than the other.

When two pumps discharge to a pumping main where the friction head is significant the maximum duty is their combined discharge. When one pump is operating (at the same speed) it will deliver more than half of this discharge. Hence head/output calculations (using the pump characteristic curves) are needed before the duty of a single pump can be assessed.

Commercially available standard pumps should be selected, and familiarity with the range of duties of typical pumps is therefore necessary. The friction head calculations involve several assumptions and cannot be precise. The selected installation may have a capacity that is different (often greater) from that intended. If this is likely to be a serious impediment to the scheme, arrangements can be made for adjusting the pump impellers after a trial period of operation. Even in very small stations it is usually prudent to provide standby plant to operate automatically when a duty pump fails. Standby pumps can also be used during maintenance and repair of other pumps.

The number of standby units which should be provided will depend on the station layout and the possible consequences of pumps failing at a time of maximum incoming flow. It should not be overlooked that one pumpset may be undergoing maintenance when this situation arises.

Provision of an emergency pumping inlet at any station is always a safeguard against mains failure, especially if the failure affects a wide area and there are insufficient mobile generators to serve all stations. Permanent provision also eliminates the hardest and most accident prone task, of inserting the suction pipes.

The need for standby electricity supply depends on the importance of continued operation during a possible period of electricity failure.

9.6 Layout of Pumpsets, Pipework, Control Equipment and Ancillary Plant

Many small and medium sized wet and dry well pumping stations with rotodynamic pumps have comparable layouts. The pumps should be in a line with their vertical spindles passing through the roof slab to the motors on the floor above.

For design considerations refer to clause 15 of BS 8005, Part 2.

9.7 Substructure Design

The form of substructure should suit the types and layout of the pumps and other plant. If alternatives are being considered it will probably be found that submerged pumpsets require the smallest substructures, vertical pumpsets the next larger and horizontal pumpsets the largest. For design guidances and structural recommendations refer to Clause 16 of BS 8005, Part 2.

9.8 Wet Wells

9.8.1 Capacity

The size of a wet well should be related to the pumping rates as, except at large stations, it provides storage for intermittent pumping. At large stations the incoming sewers can provide some of the wet well capacity. For small and medium stations the size of the wet well should be such that the pumps will not start and stop too frequently (six to 12 starts per hour is a guide).

9.8.2 Design

The lower part of the wet well is the sump, which should be shaped to suit the pump suctions. An inefficient arrangement can result in a significant reduction in pump output due to air entrainment. It should also be shaped to prevent deposition of grit and sewage solids which rapidly occurs when the sewage ceases to move.

It is advisable to provide means of stopping the inflow to the wet well for maintenance purposes.

Incoming sewers and sump design should be arranged to avoid sewers dropping into the wet well, as this can also cause air entrainment into the pump suctions. The sewers can backdrop externally into the wet well. If possible, the pump cut-in levels should be below the level of the incoming sewers, to prevent backing up except at large stations(see 9.8.1). Backdrops cause problems of:

a) Turbulence as a result of their discharging below normal water level introducing air direct to the suction pipe; and

b) blockage in the backdrops themselves.

Sump design should attempt to prevent air entrainment and subsequent cavitation in a pump.

As the efficient operation of a station will depend on both the pumps and the design of the sump, the pump supplier should approve the design of the sump and the suction pipework. At small stations it is usually sufficient if the pump suctions are not physically restricted and are well submerged when pumping commences, but for large stations it may be prudent to have hydraulic model tests to achieve an efficient design for the composite arrangement. There is considerable divergence of views on the detailed design of suction pipework.

9.8.3 Operation

A build-up of scum and grease at the sewage surface in a wet well can affect the operation of control equipment and access should be provided for cleaning the control equipment and, if necessary, for removing the scum. The part of the wet well in which the pump control equipment is located should have a sewage surface which is always reasonably tranquil.

Adequate ventilation should be provided as a safeguard against the accumulation of dangerous gases or vapors.

9.9 Ventilation, Smell and Noise

Careful consideration should be given to the question of adequate and safe ventilation of the buildings, and of any confined spaces.

For engineering and operational attentions refer to Clause 18 of BS 8005, Part 2.

9.10 Lifting Facilities

At every pumping station appropriate and suitable lifting equipment should be provided, maintained in a serviceable condition and used. This could take the form of a simple pulley block, or in a large station an overhead gantry crane. The type, rating and range of operation of cranes and other lifting equipment will vary widely depending on the pumps and ancillary equipment which have to be installed and maintained. For larger installations, permanently installed gantry cranes covering the whole area of the pumphouse are convenient. Multi-purpose lifting appliances such as lorry mounted cranes, fork-lift trucks and small hydraulic excavators are in common use in the vicinity of pumping stations. Particularly for mobile plant, consideration should be given to the question of adequate headroom, the proximity of overhead power cables, turning circle and surface wheel bearing capacity.

Slings, chains, ropes and other lifting gear should be suitable for the particular lifting operation.

9.11 Superstructure

The superstructure of a sewage pumping station will have to suit the substructure in providing accommodation for pumping units, equipment and operators. The design of the actual building requires special consideration in respect of size, type and appearance. For design guidances refer to Clause 20 of BS 8005, Part 2.

9.12 Environment and Access

Sewage pumping stations are normally situated in the outskirts of residential and industrial areas or in the rural countryside. Good access is essential for vehicles and plant for maintenance and emergency circumstances, whatever the weather conditions.

Fencing and warning signs are advisable in hazardous or vulnerable locations.

Access roads and parking areas should be designed with suitability, durability and maintenance requirements in mind. Similar consideration should be taken in deciding areas to be grassed and trees and hedges(or fences) to be provided. Landscaping can be hastened by the use of quick-growing trees and shrubs, but this involves extra trimming and the risk of excessive root growth entering into sewer tanks and pipes, and undermining foundations. Power failures and flooding due to weather or burst pipes present hazards to be met, particularly in riverside and remote areas. These can lead to inconvenience to the public from flooding, pollution and smell unless such emergencies are taken into account in general environmental considerations.

10. RANGE OF COMPONENTS AND APPLIANCES

10.1 General

Components and appliances are required to be reliable, robust, easy to maintain and appropriate for pumping water and other liquids. In addition, the aggressive nature of sewage, with its variable solid content and possibility of toxicity and explosive gases, calls for a high degree of caution and the adoption of the latest safeguards to meet all possible hazards.

10.2 Pumps

Pumps for handling sewage should be unchokeable and wear resisting. They may be divided broadly into four groups: rotodynamic; reciprocating; pneumatic and Archimedean screw.

10.2.1 Rotodynamic pumps

Rotodynamic pumps are relatively cheap to buy, of small overall dimensions in relation to capacity, light in weight and can be arranged vertically or horizontally. They may vary from moderate to high efficiency according to the size of the pump type of impeller and the head/quantity characteristic of the duty to be performed. All types of rotodynamic pumps afford a high degree of flexibility. Both quantity and head can be varied by changing the speed and/or diameter of the impeller.

When two or more pumps are required to discharge in parallel to a common rising main the head/quantity characteristics should be studied in order to obtain stable conditions and a good overall efficiency.

This important group of pumps is divided into three types, the centrifugal, the mixed flow and the axial flow or propeller-type pumps.

For their characteristics refer to Clause 5.2 of BS 8005, Part 2.

10.2.2 Reciprocating pumps

The reciprocating pump is heavy and of large dimensions in relation to its capacity. It is reliable, efficient when first installed, and is capable of operating with a high suction lift and of discharging against very high heads. It is susceptible to choking, heavy wear and tear, and loss of efficiency through wear and valve jamming. The reciprocating pump may be either of single-acting or double-acting type. Reciprocating pumps are more expensive in first cost than other types. They are expensive to maintain and therefore are rarely used for pumping crude sewage.

For more information refer to Clause 5.3 of BS 8005, Part 2.

10.2.3 Pneumatic pumps

The pneumatic ejector, whether of the automatically filled vessel or the air lift type, is suitable where reliability and ease of maintenance are of greater importance than overall efficiency, and where a small quantity of sewage is to be pumped against a relatively small head. For more information refer to Clause 5.4 of BS 8005, Part 2.

10.2.4 Archimedean screw pumps

Archimedean screw pumps are basically screws revolving at a fixed speed. They provide a steady rate of pumping and high efficiency over a wide range of flows and are also effective in pumping varying flows. They are suitable for lifting large volumes of unscreened sewage or storm water against low heads.

The actual volume lifted for any particular diameter is dependent on the speed of rotation and on the angle of inclination; the greater the angle the less the rate of discharge. The angle of inclination varies from a minimum of 27° to the horizontal to a maximum of 40°. The preferred angle is 38°. For further information refer to Clause 5.5 of BS 8005, Part 2.

10.3 Prime Movers, Drives and Other Equipment

For different types of driving motors, controls and electrical equipment and pipework and valves see BS 8005 Part 2, Section two.

11. DESIGN OF PUMPING MAINS (RISING MAINS)

11.1 Velocities of Flow

To avoid sedimentation, the minimum recommended velocity in pumping mains is about 0.75 m/s, but if there is a velocity of about 1.2 m/s for several hours each day, the minimum velocity can be as low as 0.5 m/s. The maximum velocity should normally not be above 3 m/s. Power considerations usually impose this limit.

11.2 Diameter

The diameter of a pumping main should be determined by an economic analysis of the pipeline and pumping costs and by an assessment of the engineering factors which may sometimes override the economic analysis. Alternative diameters should be examined which produce minimum and maximum velocities within the acceptable limits, and pumping costs should be estimated taking account of the normal rate of pumping(not necessarily the peak rate). The most economical scheme will be the one which involves the lowest overall annual cost, including repayment of capital cost, running and maintenance.

If septicity of the sewage is likely to be a problem the retention period in the pumping main should be reduced by adopting a smaller diameter and accepting a higher velocity of flow, even though this may mean higher power consumption.

The minimum diameter of a main is usually 100 mm but sometimes smaller mains may be considered to maintain a minimum velocity and avoid septicity.

11.3 Pressures

The maximum friction head (pressure) arises at the maximum velocity. It should be calculated by one of the recognized hydraulic systems for friction losses in a pipeline flowing full. It should be remembered that friction factors and viscosity of the liquid are likely to change when air or oxygen injection is employed to control septicity.

The possibility of positive and negative pressures due to surge (water hammer) should be considered. They are more likely to be significant in long mains or where high velocities arise. Surge analysis is complicated and is usually only undertaken when surge pressures are expected to be important.

For various means of alleviating surge pressures, see BS 8005, Part 2 (Clause 25).

Surge pressures could influence the selection of material and class of pipe of a pumping main for which recommendations are given in BS CP 312: Part 2 and BS 8010 sections 2.1,2.4. The pumping main should be so designed as to be capable of withstanding a hydraulic test pressure of not less than 1.5 times the maximum working pressure or not less than 1.5 times the maximum surge pressure, whichever is the greater, subject to the recommendations in the above codes.

11.4 Valves

The arrangement and location of isolating, air release, washout and non-return valves should be planned together.

On long mains, valves should be included to allow for sections to be isolated and emptied within a reasonable time.

Special consideration should be given to crossings of major roads, railways, watercourses and hazardous locations, but otherwise the sections might be up to about 1 km long. An isolating valve should be provided just inside or outside a pumping station so that the station pipework can be dismantled without emptying the main.

Where a section of a main is to be emptied through a washout valve, provision should be made for the removal of the contents.

Air release valves suitable for sewerage systems should be provided at summits:

a) to release air when the main is being filled;
b) to release air and gas which arise during normal pumping;
c) to mitigate the effects of surge;
d) to permit air to enter when the main is being emptied,

In the vicinity of an air release valve a main should rise to the valve at a gradient preferably not flatter than 1 in 500 and fall away at a gradient not flatter than 1 in 300 for a significant length each side of the summit.

Air release valves can make considerable noise when operating and they should be regularly maintained. If a chamber is provided it should be adequately ventilated to release the volume of air from the main and to prevent the accumulation of malodorous, toxic or flammable gases. In some situations air can adequately be released manually through a vertical pipe with a cock.

Care should be taken not to aggravate possible surge problems(see clause 11.4) by the siting or the use of incorrect types of air release valves. In certain situations it may be necessary to restrict the rate at which a pumping main is filled.

Non-return valves are used to prevent backflow after pumping has stopped and should be provided at the pumps. In special situations they may have to be sited on a pumping main; they should have extended spindles and lever arms so that they can be manually opened for emptying the main but the size of the non-return valve and static head dictate whether this is practicable. A bypass may be necessary in some cases.

11.5 Profiles

Where possible, a pumping main should be laid with continuous uphill grade and with gentle curves on both horizontal and vertical planes. When a continuous uphill grade is not possible, air release valves should be incorporated at high points and as the profile of the main dictates. Washout valves should be installed at low points.

11.6 Discharge Arrangements

The discharge of a pumping main should be arranged to avoid turbulence or splashing. It is preferable to avoid a vertical drop pipe and to arrange that the end of the pumping main is always full. If this is not possible, and the sewage may be septic, the surfaces of the structure at the discharge should be protected against corrosion.

Chambers into which pumping mains discharge should be well ventilated. (see Clause 9.9).

11.7 Anchorages

Pumping mains require anchorages to resist the thrusts developed at changes of direction, tapers, tees, valves and blank ends. Anchorages should not impede flexibility or expansion and , as far as possible, they should allow for possible replacement of fittings in the pipeline. The maximum thrusts usually occur when the pipelines are being tested.

In situ concrete blocks should be provided for buried pipelines. For horizontal mains they should take the form of a cradle wedged against the undisturbed trench side; the design should be based on the safe bearing pressure of the ground. Vertical or inclined fittings should be clamped with metal straps to concrete blocks beneath them. Inclined pipe-lines, steeper than 1 in 6 should be anchored by concrete blocks cast across the pipes and set into undisturbed ground. For design calculation of pipe restraints (anchor blocks) see IPS-E-CE-390.

In situations where it may be impractical to provide an anchor block to resist thrust, the use of flanged joints, self-restraining flexible couplings, or special harness assemblies across joints, may be considered.

These should transfer the thrust along the pipeline until either it is possible to provide a concrete anchor block, or sufficient frictional resistance is developed between the pipes and the refilled ground to overcome the thrust.

11.8 Design of Tidal Outfalls

The design objective and function of a tidal outfall is to discharge sewage to the sea (occasionally into very large rivers), usually after grit removal and adequate treatment, at a sufficient distance from the shore to ensure that the processes of dilution, dispersion and natural purification reduce the concentrations of harmful organisms and chemical, if any, present in sewage to acceptable levels in inshore waters, particularly those used for fisheries or for recreational purposes. For acceptable limits of discharge of raw sewage (treatment by dilution) or final effluents into large bodies of water see Clauses 5.1 and 5.2 of IPS-E-CE-400.

For types of outfall, principles of design and outfall hydraulic design refer to BS 8005 : Part 4.

12. STRUCTURAL DESIGN

12.1 General

To provide against the effects of settlement and soil movement, sewers should be provided with flexible joints. Sewers shall be laid below freezing level and shall have minimum cover of 1.0 to 1.2 meter.

Sewers laid within and under roads or wherever possibility of vehicles passing over exist, the depth of its crown from the ground surface shall not be less than 1.2 meters.

12.2 Recommended Design Method for Rigid Pipes (Asbestos Cement, Clay, Concrete or Grey Iron)

The design involves the computation of the total effective external load on the pipe caused by the worst combination of fill load and surface loads likely to be imposed simultaneously on any particular length of sewer, and the selection of a rigid pipe having a crushing test strength which, together with the class of bedding to be used, will provide sufficient supporting strength to carry the maximum load with an adequate factor of safety.

Simplified tables and design charts have been published see clause 10.2 of BS 8005, Part one, which indicate certain standard methods of construction.

Various standardized methods of bedding are used, and it is recommended that the bedding factors set out in Figs. 2 and 3 should normally be adopted.

The design of bedding for pipelines is based on the principle that the ability of a pipe to carry a load may be increased by the provision of suitable bedding. A rigid pipe has inherent strength but by providing a degree of encasement higher loads may be carried. A flexible pipe on the other hand will deform under the application of loads and requires support from surrounding material, and thus from the sides of the pipe trench, in order to avoid excessive deformation of the pipe.

The load on a pipeline depends on the diameter of the pipe, the depth at which it is laid, the trench width, the traffic or other superimposed loading and the prevailing site conditions.

The limits of cover for rigid pipes up to DN 300 have been computed from the loads produced by different surface loading conditions, plus backfill, for the various bedding constructions (see fig. 2 and table 5). Wide trenches, which produce the highest loading conditions, have been assumed and a factor of safety has been applied.

Bedding arrangements for flexible pipes in both narrow and wide trenches are given in Fig. 3.

12.2.1 Pipes at shallow depths

Where pipes are to be laid at less than 1.2 m below the wearing surfaces of roads or less than 1 m below road formation level or less than 0.9 m in fields and gardens, protection should be provided against loads other than final backfill and wheel loading or impact, e.g. site construction traffic, the possibility of subsequent building works or agricultural activities, the erection of fences, or from other mechanical damage and frost.

Rigid pipes of less than DN 150 laid with less than 0.3 m of cover, or of DN 150 or greater with less than 0.6 m of cover should, where necessary, be protected by surrounding them in concrete. At depths greater than these, the bedding conditions given in table 5 apply.

Ductile iron, although classed as flexible, has high inherent strength and can usually be laid without special embedment. Where laid with less than 1.0 m cover, except under roads, ductile iron pipes need no special protection. Under roads a granular surround should be provided where they are laid with 1.0 m to 0.3 m of cover, and where less than 0.3 m of cover is provided, a concrete surround may be necessary. Flexible pipes laid at depths less than 0.6 m, not under a road, should where necessary be protected against risk of damage, for example by placing over them a layer of concrete paving slabs with at least a 75 mm layer of granular material between pipes and slabs.

Pipes laid at a depth less than 0.9 m below the finished surface of a road should be suitably protected with a reinforced concrete surround or by means of reinforced bridging slabs of adequate strength.

12.2.2 Special site conditions

In some situations, for example where a drain passes beneath or near to a foundation of a structure, the load expected to be carried by the pipeline cannot readily be assessed and a completely rigid system of protection such as a concrete encasement or a 180° concrete cradle may be appropriate.

Any vertical construction or movement joint in concrete protection should be provided over the full width and depth of the concrete and located at the leading edge of a socket of a flexible joint or a sleeve coupling.

It is desirable to seek expert advice where pipes are laid on piles or beams, or where more than one pipe is laid in a trench. Unstable soils such as soft clay, silt, or fine sand may provide diminished support for pipes and bedding materials, especially if there is high water table.

12.2.3 Pipe strength

The crushing strength of the pipe may be obtained from the pipe manufacturer.

12.2.4 Soil density

A soil density of 2000 kg/m^3 has been assumed in compiling table 5 and fig. 2.

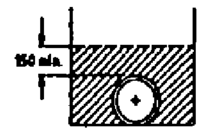
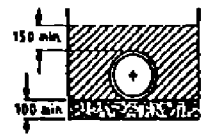
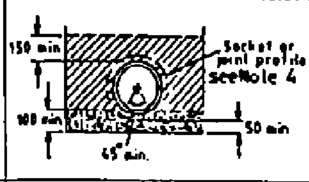
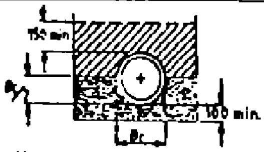
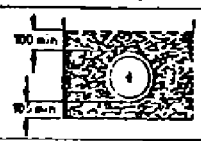
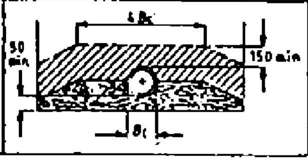
12.2.5 Traffic loads

The following static wheel loads and impact factors have been assumed in compiling table 5 and fig. 2. The main road traffic load includes as impact factor (see BS 5400 : Part 2).

	Wheel load	**No. of wheels**	**Impact factor**
Main road	112.5 kN	8	included
Other road	70 kN static	2	1.5
Field and garden	30 kN static	2	2.0

12.2.6 Total design load

The limits of cover have been calculated for the worst combination of fill and traffic loads occurring over the length of drain. If any other surcharge load is likely to be imposed its effect should be calculated and the pipeline and bedding designed accordingly.

Figure	Bedding class		Bedding factor*	Comments
2a	D		1.1	Pipe laid on trimmed trench bottom.
2b	N		1.1	Pipe laid on a flat layer of granular material with CF** not greater than 0.3.
2c	F		1.5	Pipe laid on a flat layer of granular material with CF** not greater than 0.2. Illustrated after settlement.
2d	B		1.9	Pipe laid on granular material to half diameter with CF** not greater than 0.2.
2e	S		2.2	Pipe fully surrounded by granular material with CF not greater than 0.2.
2f	B (example)		1.9 (example)	Construction as in figures 2b 2c 2d and 2e except that when the trench width exceeds four times the outside diameter of the pipe barrel, the granular material may be sloped down from that width to the trench formation.

All dimensions are in millimetres.

KEY.

 Selected fill see note 2

 Granular material see Note 1

Bc is the external pipe diameter.

Notes:

1) For granular material for bedding, see the relevant clause of IPS M-CE-105.

2) For selected material for sidefill and initial backfill, see the relevant clause of IPS M-CE-105.

3) Where there are sockets, these should be not less than 50 mm above the floor of the trench.

*** Bedding factor = The ratio of the strength of the pipeline on the specified bedding to the specified crushing strength of the pipe.**

**** For compaction fraction test for suitability of bedding material refer to Appendix D of BS 8005, part 1.**

BEDDINGS FOR RIGID PIPES
Fig. 2

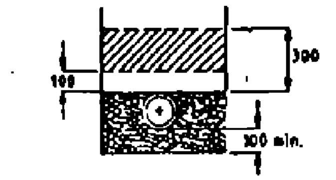
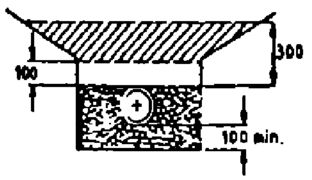
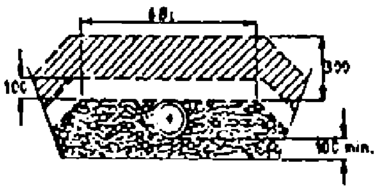
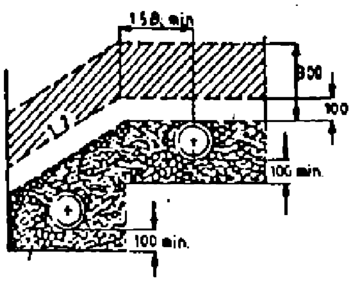
TABLE 5* - LIMITS OF COVER FOR RIGID PIPES LAID IN TRENCHES OF ANY WIDTH (SEWER AND DRAIN PIPES)

Nominal bore	Class of bedding construction	Crushing strength	Main traffic roads		Other roads		Fields and gardens	
			Min.	Max.	Min.	Max.	Min.	Max.
mm		kN/m	m	m	m	m	m	m
100	D and N	20	0.7	3.7	0.7	4.1	0.4	4.2
		22	0.6	4.3	0.6	4.6	0.4	4.7
		28	0.4	5.7	0.5	6.0	0.3	6.0
		38	0.3	8.1	0.4	8.2	0.2	8.2
	F	20	0.5	6.5	0.6	5.8	0.3	5.8
		22	0.4	6.3	0.5	6.4	0.3	6.4
		28	0.3	8.1	0.4	8.2	0.2	8.2
		38	0.2	10.0	0.3	10.0	0.2	10.0
	B	20	0.4	7.2	0.4	7.4	0.3	7.4
		22	0.3	8.0	0.4	8.2	0.2	8.2
		28	0.3	10.0	0.3	10.0	0.2	10.0
		38	0.2	10.0	0.2	10.0	0.3	10.0
150	D and N	20	–		1.1	2.5	0.8	2.7
		22	1.1	2.0	0.9	2.9	0.8	3.1
		25	0.8	2.8	0.8	3.4	0.8	3.8
		28	0.7	3.4	0.7	3.9	0.6	4.0
		38	0.6	5.2	0.6	5.5	0.6	5.8
	F	20	0.7	3.3	0.7	3.8	0.8	3.9
		22	0.8	3.8	0.6	4.3	0.6	4.3
		25	0.6	4.8	0.6	4.9	0.6	5.0
		28	0.6	5.3	0.6	5.6	0.6	5.6
		38	0.6	7.5	0.6	7.8	0.6	7.7
	B	20	0.6	4.6	0.6	5.0	0.6	6.0
		22	0.6	5.2	0.6	5.5	0.6	5.6
		25	0.6	6.1	0.6	6.3	0.6	6.3
		28	0.6	6.9	0.6	7.1	0.6	7.1
		38	0.6	9.6	0.6	9.7	0.6	9.7
225	D and N	20	–		–		–	
		25	–		–		0.7	2.2
		28	–		1.1	2.2	0.6	2.6
		38	0.8	3.0	0.7	3.6	0.6	3.7
	F	20	–		1.2	2.1	0.6	2.5
		25	0.9	2.3	0.9	3.1	0.6	3.3
		28	0.8	3.0	0.7	3.6	0.6	3.7
		38	0.6	4.8	0.6	5.1	0.6	5.1
	B	20	0.8	2.4	0.8	3.2	0.6	3.3
		25	0.8	3.7	0.7	4.2	0.6	4.7
		28	0.6	4.3	0.6	4.7	0.6	4.8
		38	0.6	6.3	0.6	6.5	0.6	6.8
300	D and N	22	–		–		–	
		25	–		–		–	
		28	–		–		1.0	1.8
		34	–		1.4	1.8	0.7	2.3
		35	–		1.4	1.8	0.7	2.3
		47	0.9	2.4	0.8	3.2	0.8	3.3
	F	22	–		–		0.9	1.8
		25	–		–		0.7	2.2
		28	–		1.1	2.5	0.6	2.7
		34	0.9	2.5	0.8	3.3	0.8	3.4
		35	0.9	2.5	0.8	3.3	0.8	3.4
		47	0.6	4.2	0.8	4.8	0.8	4.7
	B	22	–		1.1	2.4	0.8	2.6
		25	1.1	1.9	0.9	2.8	0.6	3.0
		28	0.8	2.8	0.8	3.5	0.8	3.6
		34	0.8	3.8	0.8	4.3	0.8	4.4
		35	0.6	3.9	0.6	4.3	0.6	4.4
		47	0.6	5.7	0.8	5.9	0.8	6.0

*** See 12.2.1 before using the above table, which has been compiled using the following:**

a) soil density of 2000 kg/m^3 ;
b) factor of safety of 1.25 assuming average site supervision and workmanship;
c) wide trench condition;
d) traffic loads as set out in 12.2.5;
e) the bedding factors shown in fig. 2.

Where the cover is less than 1 m, special factors apply.

Figure	Type of trench		Comments
3a	Typical		Pipe surrounded in granular material except above the crown where selected fill may be suitable.
3b	Vee		A sub-trench should be dug as shown, with construction as in figure 3a
3c	Wide		Construction as in figure 3a except that when the trench exceeds six times the outside diameter of the pipe barrel, the granular material may be sloped down from that width to the trench formation.
3d	Stepped dual		Construction as in figure 3a for top pipe.

All dimensions are in millimetres.

KEY.

Selected fill see Note 2

Granular material See Note 1

Granular material or selected fill containing no stones larger than 40 mm.

Bc is the external pipe diameter.

Notes:

1) For granular material for bedding, see the relevant clause of IPS-M-CE-105.
2) For selected material for initial backfill, see the relevant clause of IPS-M-CE-105.
3) Detail pipelaying procedures for beddings, see the relevant clause of IPS-C-CE-382.
4) Where there are sockets, these should be not less than 50 mm above the floor of the trench.

BEDDINGS FOR FLEXIBLE PIPES

Fig. 3

12.2.7 Pipe settlement

A pipe will normally settle under the bedding arrangements shown in Fig. 2. The extent of this settlement will depend upon the total load and the shape, grading, physical properties, and degree of compaction of the bedding material, but in no case should it exceed that illustrated for class F bedding.

12.2.8 The use of table 5

Table 5 shows the relation between pipe crushing strength, bedding class, range of cover and surface conditions, and is applicable to asbestos cement, clay and concrete pipes.

If the trench formation is capable of being hand trimmed to receive the pipeline directly upon it and if the actual cover is included in the range for class D, this class of bedding may be used. However if the cover is within the range of class D but it is considered to be impracticable to trim the trench bottom, then bedding of class N can be used.

In all cases if the cover requirements for a particular pipe strength and traffic condition preclude the use of class D or N bedding, the bedding construction required for that cover, as given in table 5, should be used.

For intermediate pipe strengths not shown in this table, linear interpolation will give the permissible depth range for a given bedding.

Generally, for any length of drain between points of access, the pipe strength and class of bedding selected for the worst condition should be used for the whole of that length of drain.

Pipe strength and bedding for the majority of building drainage work may be determined using this simplified method. Full calculation based on Marston theory allowing for the narrowest practical trench width and the actual soil density plus the appropriate traffic wheel load may result in a more economic design.

12.2.9 Higher strength pipes

For pipes of higher strength than those shown in table 5, laying depths and bedding requirements may be obtained from the pipe manufacturers.

12.3 Design Method for Flexible Pipes (ductile iron, GRP, steel and unplasticized PVC)

12.3.1 General

A flexible pipe may be defined as one that, under soil and surcharge loads, is capable of deflecting to a significant extent without any sign of structural distress.

Flexible pipes support loads by soil/pipe interaction, and so the surrounding fill is an essential part of the support structure and proper regard should be paid to its selection, placing and compaction.

Pipes up to about DN 600 are usually laid according to empirical rules. The wall thicknesses for such pipes for use for underground non-pressure applications are chosen on the basis of experience with the assumption that pipes will be installed so as to limit the amount of ovality that will develop in service to a (usually) 5% decrease in vertical diameter. For concrete-lined steel pipes a limit of 2% is desirable.

Since the consistent control of compaction of soil around pipes, particularly in the smaller sizes, is not easy to ensure, it is preferable to employ a granular surround of material requiring the minimum of tamping to achieve a satisfactory density (see figs. 2 and 3).

Care is needed to identify situations where the pipes may be subjected to considerable external hydrostatic head or where the soil may be rendered unstable by water, since the lateral support afforded to the selected surround material by the adjacent soil may then be inadequate. In cases of doubt, pipes should be used having a wall thickness (stiffness) capable of resisting the long-term external hydrostatic pressure without buckling in the absence of soil restraint.

Computed load design methods may also be employed particularly for pipes over about DN 600. These methods are directed towards the following:

a) the determination of the maximum external loading;and
b) the selection of a suitable pipe wall thickness (stiffness).

The latter, taken in conjunction with a specified degree of compaction of the surrounding bedding and adjacent soil material, should maintain the degree of pipe deformation within prescribed limits without risk of wall buckling or of causing unsafe strains in the pipe wall.

Ductile iron pipes are flexible and are commonly used for pumping mains and in cases where extra strength is required. This may be to resist high external loads, such as traffic loads, where there is no alternative but to place the sewer near to the road surface.

12.4 Factor of Safety Methods

This method uses a factor of safety which differs from usual practice in that it is applied not to the calculated loads based on the properties of the soils to be encountered, but to predicted loads derived from experimental work. Further, the value of ground support utilized in the design method may be less than that which occurs in practice. Where the calculation of supporting strength is based on the crushing strength specified in the IPS-M-CE-125 (Standard for pipes of the various materials) and where the maintenance of this strength is supported during manufacture by a properly controlled quality assurance system, it is recommended that a factor of safety of 2 should be applied to the calculated design load.

12.5 Design Method for Box Culverts

Box culverts are required to withstand the worst combination of the effects of fill and surcharge loads. They are individually designed to the requirements of the relevant highway, rail or water authority and are not manufactured to standard class strengths as in the case of pipes. Surcharge loading specification is to BS 5400: Part 2 and structure design to either BS 8110 or BS 5400: Part 4.

12.6 Particular Cases

12.6.1 Sewers passing below and through structures

The routing of sewers below or through a building or structure should be avoided.

The walls of manholes and other structures generally will be subject to settlement differing from that of the pipe. Differential settlement can be accommodated by means of flexible joints. The risk of shear fractures is considerabley reduced by the provision of a flexible joint, located as close to the face of the structure as is feasible, compatible with the satisfactory completion and subsequent movement of the joint.

In cases where differential settlement is expected, the length of the next pipe(rocker pipe) away from the structure should be kept short. The length of rocker pipe should be approximately 0.5 m to 0.75 m for pipes up to DN 450 and 1 m for pipes up to DN 750.

Where considerable differential settlement is expected, several rocker pipes should be laid and the designed gradient may need to be increased locally so as to reduce the likelihood of a backfall developing.

12.6.2 Two sewers in one trench

Where two sewers are laid at different levels in the same trench the bedding material should be well compacted over the full trench width above the lower pipe to provide a stable bed for the upper pipe. Side fills of the upper pipe should also be well compacted to ensure full and continuing support under the haunches.

12.6.3 Sewers on piles, piers or walls

Where there is no alternative to laying sewers on beams supported either on piles, piers or walls the loads will be considerably higher than usual due to subsequent differential settlement of the fill. It will usually be necessary to support pipes with reinforced concrete bedding or surround designed as a beam to carry the sum of the highest possible loads over the spans between supports. A flat-top T-beam should not be used without the addition of pipe bedding. If differential settlement of the supports is possible, flexible joints in the sewer, extending through all concrete work, should be formed over the edges of each support.

13. MANHOLES

13.1 General

13.1.1 Structural design of manholes

Manholes and other pipeline structures should be designed to carry the worst combination of superimposed and ground loading. Foundations should be designed to carry all the imposed loads. The chamber walls and base should be designed to take into account also any lateral loading and/or hydrostatic upthrust.

13.1.2 Spacing of manholes on sewers not more than 1.0 m in diameter

As smaller sewers cannot easily be entered for cleaning or inspection a manhole should be built at every change of alignment or gradient, at the head of all sewers or branches, at every junction of two or more sewers, and wherever there is a change in size of sewer. In exceptional difficulties in routing, to accommodate angles less than 15° a slow bend may be sited on the sewer immediately upstream of a manhole. Manholes should not be positioned further apart than 100 m, beyond which it is not practicable to use drain rods or some form of pipe scraper.

Preferred distance between manholes is around 50 meters.

13.1.3 Spacing of manholes on sewers more than 1.0 m in diameter

Where a man can enter a sewer for inspection it is not necessary to have a manhole at every change in alignment.

For spacing of manholes on sewers larger than one meter diameter refer to clause 11.1.3 of BS 8005, Part 1.

13.2 Types of Manhole

13.2.1 General

At a change in diameter of sewers, and where conditions permit, the soffits or crowns of the two sewers should be at the same level.

13.2.2 Straight - through manholes

The simplest type of manhole is that built on a straight run of sewer with no side junctions.

13.2.3 Junction manholes

A manhole should be built at every junction of two or more sewers, and the curved portions of the inverts of tributary sewers should be, wherever possible, within the manhole. To achieve this with the best economy of space, the chamber may be built either rectangular or circular. To avoid surcharging of a connecting sewer, its soffit should be at same level as that of the main sewer, wherever conditions permit. However, this may restrict the gradient of the connecting sewer and prove to be uneconomical. The flow from branch connections should merge with the main flow at the smallest angle practicable. To achieve this, bends may be necessary.

13.2.4 Deep manholes

Deep manholes should be provided with rest chambers, or platforms not more than 6 m apart vertically. Access openings in successive landings should preferably be staggered to limit the height of an accidental fall from the access ladder but separate openings should be provided, suitably protected by trap doors directly below each other for the movement of materials and tools.

The landings and platforms and access openings should be of adequate size to accommodate at least two men with safety harness; and to minimize the risk of falling they should be provided with handrails where possible. Refer to Standard Drawing No. IPS-D-CE-250, 251, 252.

13.2.5 Backdrop manholes

a) General

Where a sewer connects with another sewer at a significantly lower level, a manhole may be built on the lower sewer incorporating a vertical or nearly vertical drop pipe from the higher sewer to the lower one. This pipe may be outside the chamber and encased in concrete, or supported inside the chamber, which should be suitably enlarged. Access should be provided to the higher sewer and to the vertical drop pipe for men to inspect and to maintain the sewer.

If the drop pipe is outside, a continuation of the sewer should be built through the chamber wall and the vertical drop pipe continued up to an access cover at ground level, and encased in concrete. If the drop is inside, it should have adequate and accessible means for rodding. The diameter of the backdrop should be more than sufficient to discharge the maximum flow from the incoming pipe.

b) Small sewers (about DN 300 or less)

The backdrop on small sewers pipe should terminate at its lower end with a plain or rest bend turned horizontally so as to discharge its flow 45°or less to the direction of the flow in the main sewer. An external pipe should be surrounded with 150 mm of concrete.

Where the difference in level is less than 1.8 m a ramp is often preferable; a rodding eye should be provided as for a backdrop.

For backdrops on larger sewers and vortex backdrops refer to section three of BS 8005, part 1.

13.2.6 Hydrogen sulphide in manholes

Hydrogen sulphide will be released from sewage at points of high turbulence,e.g. backdrops. Adequate ventilation should be provided and the materials of construction chosen with care.

13.2.7 Catchpits

Catchpits are chambers on surface water sewers installed to retain silt and constructed as manholes but without benchings. A floor level below the lowest pipe invert is used to form a sump, which will require periodic cleaning.

13.3 Detailed Design of Manholes

13.3.1 General

For design guidances refer to clause 11.3 of BS 8005, Part 1 and IPS Standard Drawing No. IPS-D-CE-250, 251, 252.

13.3.2 Sizes of manholes

For sizes of manholes and their accessories, i.e. manhole covers, ladders etc. see Iranian Petroleum Standard manhole drawings of different sizes. For information only, BS 8005, Part 1 can also be referred.

APPENDICES

APPENDIX A

ENERGY LOSSES AT MANHOLES AND BENDS

A.1 MANHOLES

Table 6 gives values of energy loss coefficient,k, derived from experiments on manholes where the sewer is surcharged. The energy losses when the sewer is only just full(i.e. with the flow confined by the manhole benching) will be less than those obtained using these coefficients. When the manhole incorporates a junction, the energy losses will be increased and will depend on the geometry of the junction and on the flows in the branches.

TABLE 6 - ENERGY LOSS COEFFICIENT, K, FOR MANHOLES

PLAN SHAPE OF MANHOLE	TYPE OF MANHOLE		
	STRAIGHT THROUGH	30° BEND	60° BEND
RECTANGULAR CIRCULAR	0.10 0.15	0.40 0.50	0.85 0.95

A.2 CIRCULAR BENDS

Table 7 gives values of k for 90° circular bends, flowing full, for various ratios of bend radius, R, to nominal pipe bore, D.

TABLE 7 - ENERGY LOSS COEFFICIENT, K, AT BENDS

BEND RADIUS/PIPE DIAMETER R/D	K
0.5	1.0
1.0	0.25
1.5	0.18
2.0	0.16
5.0	0.18
10.0	0.24

The values given in table 6 apply when the straight length of pipe downstream from the bend is greater than 30 pipe diameters.

A.3 MITRE BENDS

The energy loss coefficient,K, for a single mitre bend is given by:

$$K = 1.4 \frac{\theta^2}{90}$$

Where:

θ is the bend angle (in degrees).

Table 7 gives the loss coefficient for a 90 lobster-back bend comprising 4/22.5, 3/30 , or 2/45 mitre bends.

TABLE 7 - ENERGY LOSS COEFFICIENT, K, FOR LOBSTER - BACK BENDS

1/D	4/22.5	3/30	2/45
	LOSS COEFFICIENT K		
0.5	0.40	0.45	0.55
1.5	0.25	0.30	0.40
3.0	0.32	0.35	0.48
6.0	0.32	0.37	0.50

Notes:

1) 1 is the centerline length of the individual short pieces (which are all of equal length) from which the bend is made, D is nominal pipe bore.

2) The values given are for a rough pipe: the loss coefficients for a smooth pipe will be approximately 75% of these values.