CHAPTER 15 PIPE DRAINS

15.1 INTRODUCTION

This Chapter provides an introduction and outline of the design requirement and procedures for pipe drain, a subsurface conduit for conveying minor storm flows from impervious surfaces, such as streets, parking lots, buildings, etc. to major stormwater facilities within an urban catchment.

Pipe drain systems are recommended mainly for high density residential and commercial/industrial developments where the use of open drain and swales is not feasible and uneconomical. Since other services are involved, requirement for the locations and alignments of pipe drains will also be provided in the design procedure.

The pipes, differing in lengths and sizes, are connected by appurtenant structures below ground surface (Figure 15.1) using pits, junctions or manholes and other miscellaneous structures such as transitions, bends and branches.

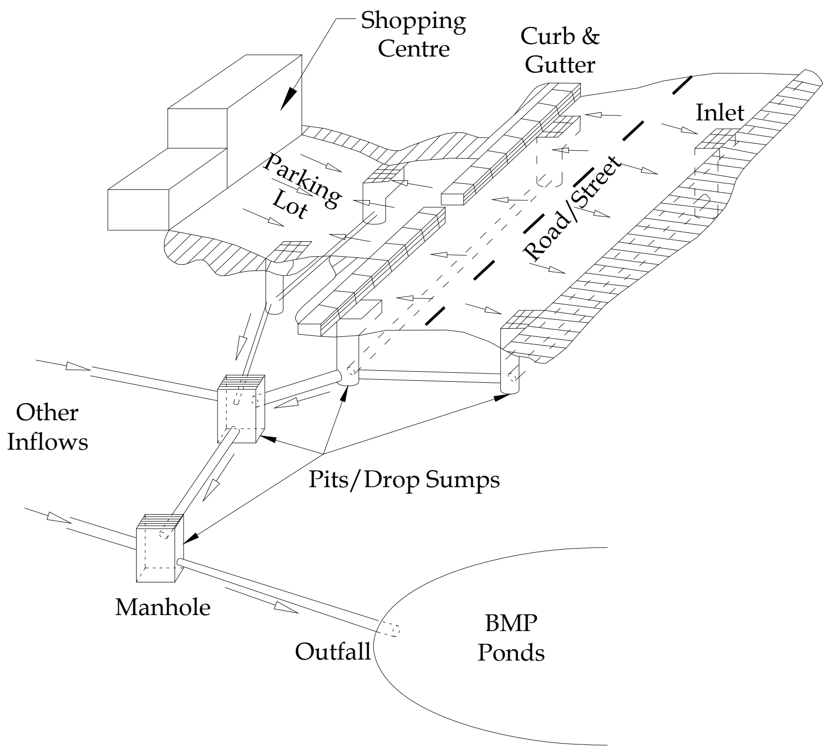

Figure 15.1: Elements of Pipe Drainage System

15.2 LOCATION AND ALIGNMENT

Standardised locations for stormwater pipelines are provided to limit the negotiations needed when other services are involved and permit ready location by maintenance crews.

15.2.1 Roadway Reserves.

Table 15.1 provides typical requirements for location of pipe drains and services within road reserves, however these may be varied for different authorities. The relevant authority should be consulted concerning its standard alignments for services.

In selecting pipeline locations, it is necessary to also consider inlet location preferences as outlined in Chapter 13.

Table 15.1: Alignments within Roadway Reserves

Pipe Diameter (mm)	Alignment
375 to 675	under curb line
750 to 1800	within median strip, or centreline of road

15.2.2 Privately Owned Properties

Stormwater pipelines are often constructed in parallel to sewers and as the sewerage system is usually deeper, pipes connecting to stormwater ties have less problems in crossing over the sewer.

Alignments shall offset with sufficient distance from building lines to allow working space for excavation equipment.

Acceptable centreline offset alignments from property boundaries in residential, commercial, and industrial areas shall be in accordance with Table 15.2.

Table 15.2: Offset Distance within Privately Owned Properties

Pipe Diameter (mm)	Rear Boundary	Side Boundary
375 to 450	1.8m	1.2m (see Note)
525 to 675	1.8m	1.5m (see Note)

Note: Where other hydraulic services or power poles are located on the same side of a property boundary, the centreline of the stormwater pipeline shall be located 1.8m from the property boundary.

15.2.3 Public Open Space

The location of stormwater pipelines within public land such as open space shall be brought to the attention of the operating Authority for consideration. As a guide, unless directed otherwise, stormwater pipelines shall be located not less than 3 m from the nearest property boundary.

15.2.4 Drainage Reserves

A drainage reserve shall be wide enough to contain the service and provide working space on each side of the service for future maintenance activities. Minimum drainage reserve widths shall be in accordance with Table 15.3.

Pipelines up to and including 675mm diameter may be located within privately owned properties if satisfactory arrangements are made for permanent access and maintenance. Larger diameter pipelines shall be located within public open space or outside privately owned properties in separate drainage reserves.

Consideration should be given to the multi-purpose use of drainage reserves such as open space or pedestrian corridors.

Table 15.3: Minimum Drainage Reserve Widths

Pipe Diameter, *D* (mm)	Minimum Reserve Width (m)
Invert < 3.0 m deep	
375 to 450	2.5
525 to 675	3.0
750 to 900	3.5
1050 to 1200	3.5
1350 to 1800	not less than 3 x *D*
Invert 3.0 - 6.0 m deep	
375 to 450	3.5
525 to 675	4.0
750 to 900	4.5
1050 to 1800	not less than 4 x *D*

Note: Where other hydraulic services or electricity services are laid on the same side of the property boundary, the required reserve width shall be increased by 500 mm to provide horizontal clearance between services.

15.2.5 Clearance from Other Services

Minimum clearances between stormwater pipelines and other services shall be in accordance with Table 15.4. The nominated clearance should make due allowance for pipe collars and fittings. Special protection may be provided to protect service crossings by concrete encasing the stormwater pipe for sufficient length to bridge the trench of the other service.

Table 15.4: Minimum Clearances

Service	Clearance (mm)
Horizontal	
All services	600
Vertical	
Sewers	150
Water Mains	75
Telephone	75
High Pressure Gas	300
Low Pressure Gas	75
High Voltage Electricity	300
Low Voltage Electricity	75

15.3 HYDRAULICS FUNDAMENTALS

15.3.1 Flow Type Assumptions

The design procedures presented in this chapter assume that flow within pipe drain is steady and uniform. The discharge and flow depth in each pipe segment are therefore assumed to be constant with respect to time and distance. For prismatic conduits, the average velocity throughout a segment is considered to be constant.

In actual storm water drainage systems, the flow entering at each pit is variable, and flow conditions are not steady or uniform. However, since the usual hydrologic methods employed in storm water drain design are based on computed peak discharges at the beginning of each segment, it is generally a conservative practice to design using the steady uniform flow assumption.

Two design philosophies exist for sizing storm water drains under steady uniform flow assumption. The first is referred to as open channel or gravity flow design. The pipe segments are sized so that the water surface within the conduit remains open to atmospheric pressure and the flow depth throughout the conduit is less than the height of the conduit. The second is the pressure flow design where the flow in the conduit is at a pressure greater than atmospheric. Under this condition, there is no exposed flow surface within conduit. The pressure head will be above the top of the conduit and will not equal the depth of flow in the conduit. In this case, the pressure head rises to a level represented by the Hydraulic Grade Line (HGL).

For a given flow rate, design based on open channel flow requires larger conduit sizes than those sized based on pressure flow but it provides a margin of safety by providing additional headroom in the conduit to accommodate an increase in flow above the design discharge. However, in situation where there is adequate headroom between the conduit and pit/junction elevations to tolerate pressure flow, significant cost savings may be realised.

15.3.2 Hydraulic Capacity

The flowrate through a storm water pipe depends on the upstream and downstream water levels, as well as the pipe characteristics. Several flow friction formulas, however, have been advanced which define the relationship between flow capacity and the pipe characteristics (size, shape, slope and friction resistance). The most common equations are from the Manning formula and Colebrook-White formula. For circular storm water drains flowing full, Manning formula is reduced to represent the followings:

$$V = (0.397/n)\, D^{0.67}\, S_o^{0.5} \qquad (15.1a)$$

$$Q = (0.312/n)\, D^{2.67}\, S_o^{0.5} \qquad (15.1b)$$

$$D = [(Q\, n/(0.312\, S_o^{\,0.5})]^{0.375} \qquad (15.1c)$$

where,

V = Mean velocity (m/s);

Q = Flow rate (m^3/s);

D = Circular pipe diameter (m);

S_o = Slope of HGL; and

n = Manning coefficient.

The Colebrook-White equation, representing the flow mean velocity is

$$V = -\,0.87[\sqrt{(2g.D.S)}]\, \log_e [(k/3.7D) + (2.51\upsilon/D\sqrt{(2g.D.S)}] \qquad (15.2)$$

where,

S = Longitudinal slope of pipe (m/m);

k = Pipe roughness height;

D = Circular pipe diameter (m); and

υ = Kinematic viscosity of water (m^2/s)

Chart 15.A1 and Chart 15.A2 in Appendix 15.A can be used respectively for solution of Manning formula and Colebrook-White formula for flow in circular conduits. The Hydraulic Elements graph Chart 15.A4 can be used to assist in the solution of the Manning's equation for part full flow in storm water drains. Tables for the hydraulic design of pipe, sewer and channel are also provided (Wallingford and Barr, 2006).

15.3.3 The Energy and Hydraulic Grade Lines

The energy grade line (EGL) is a line that represents the total energy along a channel or conduit carrying water. Total energy includes elevation head, velocity head and pressure head.

The hydraulic grade line (HGL) is a line coinciding with the level of water at any point along an open channel. In closed conduits flowing under pressure, the hydraulic grade line is the level to which water would rise in a vertical tube at any point along the pipe. The HGL is used in determining the acceptability of a proposed storm water drainage system by establishing the elevation to which water will rise when the system is operating under design conditions. The HGL can be determined by subtracting the velocity head from the EGL, as shown in Figure 15.2.

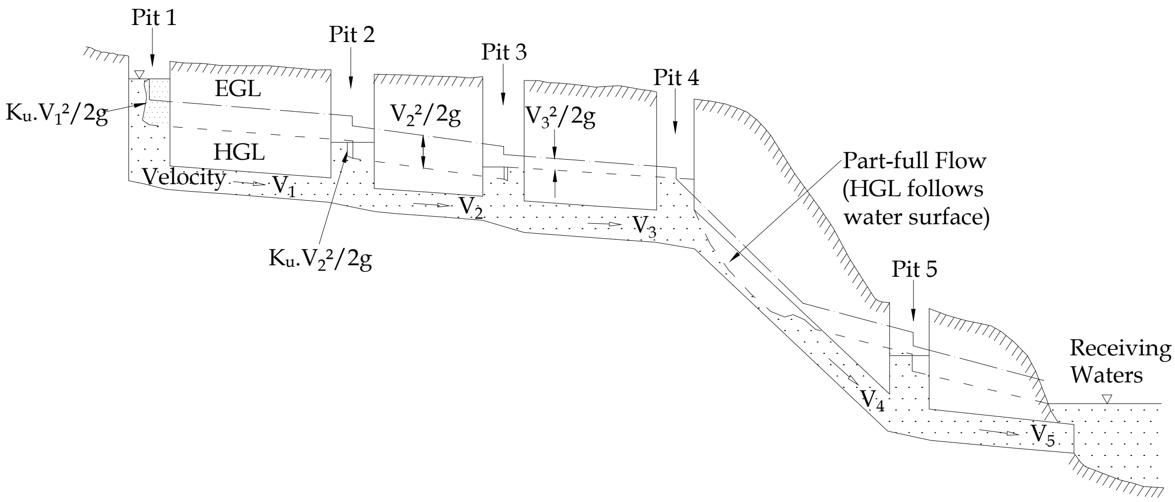

Figure 15.2: Hydraulic Grade Line (HGL) and Energy Grade Line (EGL) of Pipe Drainage
(Inst. Engrs., Aust., 1977)

Pressure head is normally lost in both pipes and pits due to friction and turbulence and the form of the HGL is therefore a series of downward sloping lines over line lengths, with steeper or vertical drops at manholes. In some circumstances there may be a pressure gain and therefore a rise in the HGL at a structure. In these cases the gain should be taken into account in the hydraulic calculations.

When the pipe is not flowing full, such flow is considered as open channel flow and the HGL is at the water surface. When pipe is flowing full under pressure flow, the HGL will be above the obvert of the pipe. Inlet/pit overflow can occur if the HGL rises above the ground surface.

Pipes which flow under pressure may be located at any grade and at any depth below the HGL without altering the velocity and flow in the pipe subject to the grade limitations. Hence, pipe grade may be flattened to provide cover under roads, or clearance under other services, without sacrificing flow capacity, provided sufficient head is available.

The HGL and the Water Surface Elevation (WSE) must be below the surface level at manholes and inlets, or the system will overflow. This depth below the surface is termed the *freeboard*. Minimum freeboard requirements are specified in the design criteria in Section 15.3.5 and 15.4 The level of the HGL for the design storm applied should be calculated at the upstream and downstream side of every inlet or manhole, at points along a pipe reach where obstructions, penetrations or bends occur, or where a branch joints.

It is recommended that designers check that the elevation of the total energy line falls progressively as flow passes down through the drainage system. This is an important check that should be undertaken where the drainage system is complex and where the configuration of pipes and structures does not conform to the structure loss charts available.

15.3.4 Energy Losses

Prior to computing the HGL, all the energy losses in pipe sections and junctions must be estimated. In addition to the principal energy involved in overcoming the friction in each conduit section, energy or head is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions and manholes.

15.3.4.1 Friction Losses

The head loss due to friction in pipes is computed by the relationship Equation 15.3a and for full flow pipe it is given by Equation 15.3b derived from Manning's formula.

$$h_f = S_f L \tag{15.3a}$$

$$Sf = h_f / L = (Qn/0.312\, D^{2.67})^2\, L \tag{15.3b}$$

where,

h_f = Head loss in pipe due to friction (m);
n = Manning's roughness coefficient;
L = Length of the pipe (m);
S_f = Slope of HGL;
D = Diameter of the pipe (m); and
Q = Discharge in the pipe (m^3/s).

The Manning's roughness coefficients *n* and roughness height *k* for some pipe material are given in Table 15.5.

Table 15.5: Pipe Roughness Values

Pipe Material	*n*	*k* (mm)
Cost iron – cement lined	0.011	0.3
Concrete (Monolithic) – Smooth forms	0.012	0.6
Concrete (Monolithic) – rough forms	0.015	0.6
Concrete pipe	0.011	0.3
Plastic Pipe (smooth)	0.011	0.3
Vitrified clay pipe	0.011	0.3

Further sources of information on roughness values can be found elsewhere (French, 1985 and Chow, 1959).

15.3.4.2 Structure Losses

Losses due to obstructions, bends or junctions in pipelines may be expressed as a function of the velocity of flow in the pipe immediately downstream of the obstruction, bend or junction as follows:

$$h_s = K\, V_o^2 / 2g \tag{15.4}$$

where,

h_s = Head loss at structure (m);
K = Pressure change coefficient (dimensionless); and
V_0 = Velocity of flow in the downstream pipe (m/s).

Pressure change coefficients K (sometimes referred to as structure loss coefficients) are dependent on many factors, for example:

- Junction, manhole and inlet structure geometry;
- Pipe diameters change;
- Bend radius;
- Angle of change of direction; and
- Relative diameter of obstruction etc.

(a) Losses at Junctions

At a junction with an access structure such as manhole or pit, the static head loss or pressure head change, which is the drop of the HGL at the junction can be calculated from,

$$H = K_u \, V_o^2/2g \qquad (15.5)$$

where,

H = Pressure head change at junction (m);

K_u = Pressure change coefficient (dimensionless); and

V_o = Velocity at the outlet pipe.

The value of the head loss coefficients, K_u for a limited range of configurations are presented by Chart 15.A5 and Chart 15.A6, part of the 'Missouri Charts'. The main difficulty in presenting information on pit losses is the almost infinite number of configurations which can occur. The Missouri Charts (Sangster et al., 1958) or similar information such as the charts in the Queensland Urban Drainage Manual, 2008 are too voluminous to present in this manual and they should be consulted if a great precision calculation is required. The design Chart 15.A7(a) and Chart 15.A7(b) is an attempt to simplify the pit loss calculation procedure by Mills (O'Loughlin (2009)). The data is greatly simplified and therefore is not highly accurate, and it is not intended that the chart be used with great precision. Though it is not highly accurate, the charts provide realistic head loss coefficients for most cases. The input data ($d/D_o, D_u/D_o$) can be determined approximately without the need for iteration to give reasonable value with minimum of effort. For greater precision, the loss coefficient calculated iteratively as layout by Flowchart 15.B1 in Appendix 15.B for design of small reticulation systems where the values obtained are seen not to be critical, the Equation 15.6 can be used to estimate pit loss coefficient K.

$$K = 0.5 + 2(Q_m/Q_o) + 4(Q_g/Q_o) \qquad (15.6)$$

where,

Q_o = Outlet flow rate (m^3/s);

Q_g = Inflow above the water level (pipes are assumed to flow full) (m^3/s); and

Q_m = Misaligned inflows that enters below the pit water level (m^3/s).

Notes;

- Any pipe inflow that is aligned with the outlet within D/4 is assumed not to cause losses;
- Subtract 0.5 to the result of Equation 15.6 if (i) there are deflectors, or (ii) the outlet diameter is larger than the biggest inlet; and
- Add 1.0 for opposed inlet.

Large energy losses and pressure changes can be avoided by attention to simple details in the design and construction supervision of pits and manholes (AR & R, 1998). One principle is to ensure that jets of water emerging from incoming pipes are directed to outlet pipes, rather than impinging on pit walls.

HEC – 22 Manual (USFHWA, 2009) introduced its latest method for estimating losses in access holes (junctions) and inlets (pits) by classifying access holes and their hydraulic conditions in a manner analogous to inlet control and full flow for culverts. The FHWA access hole method follows three fundamental steps:

Step 1: Determine an initial access hole energy level based on inlet control (weir and orifice) or outlet control (partial and full flow) equations.

Step 2: Adjust the initial access hole energy level based on benching, inflow angle (s), and plunging flows to compute the final calculated energy level.

Step 3: Calculate the exit loss from each inflow pipe and estimate the energy gradeline, which will then be used to continue calculations upstream.

For detail account of the method, readers and designers are referred to the publication.

(b) Inlets and Outlets

Where the inlet structures is an endwall (with or without wingwalls) to a pipe or culvert, an allowance for head loss should be made. Table 15.A1 in Appendix 15.A provides loss coefficients K_e to be applied to the velocity head.

$$h_e = K_e V_o^2/2g \tag{15.7}$$

where,

h_e = Head loss at entry or exit (m);

K_e = Entry or exit loss coefficient; and

V_0 = Velocity in pipe (m/s).

(c) Bends

Under certain circumstances it may be permissible to deflect a pipeline (either at the joints or using precast mitred sections) to avoid the cost of junction structures and to satisfy functional requirements. Where pipelines are deflected an allowance for energy loss in the bends should be made. The energy loss is a function of the velocity head and may be expressed as:

$$h_b = K_b V_o^2/2g \tag{15.8}$$

where,

h_b = Head loss through bend (m); and

K_b = Bend loss coefficient = 0.0033 A, where A is the angle of curvature in degrees.

Values of bend loss coefficients for gradual and mitred bends are given in the Design Chart 15.A8 and Table 15.A2 respectively in Appendix 15.A.

(d) Obstructions or Penetrations

An obstruction or penetration in a pipeline may be caused by a transverse (or near transverse) crossing of the pipe by a service or conduit e.g. sewer or water supply. Where possible, such obstructions should be avoided as they are likely sources of blockage by debris and damage to the service. To facilitate the removal of debris, a manhole should be provided at the obstruction or penetration.

The pressure change coefficient K_P at the penetration is a function of the blockage ratio. Design Chart 15.A9 in Appendix 15.A should be used to derive the pressure change coefficient, which is then applied to the velocity head.

$$H = K_p V_0^2/2g \tag{15.9}$$

where,

h_p = Head loss at penetration (m); and

K_p = Pressure change coefficient of the penetration.

Where a junction is provided at an obstruction or penetration it is necessary to add the structure loss and the loss due to the obstruction or penetration based upon the velocity, V in the downstream pipe.

(e) Pipe Branch losses

A pipe branch is the connection of a lateral pipe to a larger trunk pipe without the use of junction (Figure 15.3). The loss for a pipe branch can be estimated by Equation 15.8. Alternatively the formula relating the pressure loss coefficients and velocity head at the branch lines may produce estimate of losses. Design Chart 15.A10 in Appendix 15.A provides the pressure loss coefficients.

$$H_j = \{[(Q_o V_o)-(Q_u V_u)-(Q_L V_L \cos \theta)]/[0.5g(A_o-A_u)]\} + h_u-h_o \quad (15.10)$$

where:

H_j = Branch loss (m);
Q_o, Q_u, Q_L = Outlet, inlet, and lateral flows respectively (m^3/s);
V_o, V_u, V_L = Outlet, inlet, and lateral velocities respectively (m/s);
h_o, h_u = Outlet and inlet velocity heads (m);
A_o, A_u = Outlet and inlet cross sectional areas (m^2); and
θ = Angle between the inflow trunk pipe and inflow lateral pipe.

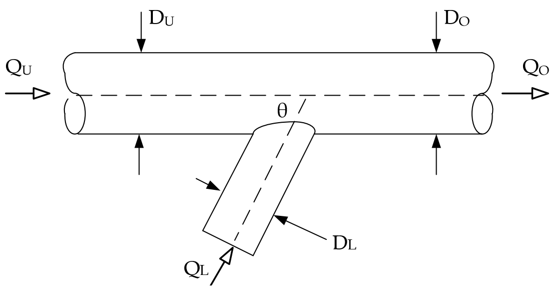

Figure 15.3: Branch /junction Line

(f) Transition losses (Expansion and Contractions)

Sudden expansions or contractions in stormwater pipelines should normally be avoided. They may however need to be installed as part of a temporary arrangement in a system being modified or upgraded, or in a relief drainage scheme.

Expansions and contractions also occur at the outlet and inlet, respectively, of stormwater pipelines The energy loss in expansions or contractions in non-pressure flow can be evaluated using Equation 15.11a for a sudden contraction and Equation 15.11b for sudden expansion,

$$H_c = C_u (V_2^2/2g - V_1^2/2g) \quad (15.11a)$$

$$H_e = C_u (V_1^2/2g - V_2^2/2g) \quad (15.11b)$$

where,

C_u = Expansion or contraction coefficient (Chart 15.A11, Appendix 15.A);
V_1, V_2 = Velocity upstream and downstream respectively (m/s); and
g = Acceleration due to gravity, 9.81 m^3/s.

The pressure change due to the expansion or contraction can be derived using the energy loss coefficients determined from the Design Charts 15.A11, in Appendix 15.A. The entrance loss coefficient should be applied to the absolute value of the difference between the two velocity heads.

15.3.5 Freeboard at Inlets and Junctions

For the design of underground drainage systems a freeboard should be provided above the calculated water surface elevation (WSE) to prevent surcharging and to ensure that unimpeded inflow can occur at gully inlets.

The maximum permitted WSE should allow for the head loss resulting from surface inflow through grates etc. into the structure being considered.

Where an appropriate chart is not available it is recommended that the WSE be arbitrarily adopted at the height above the calculated HGL (as described in Section 15.5) in accordance with Equation 15.12:

$$WSE - HGL = 0.3\ V_u^2/2g \tag{15.12}$$

where,

V_u = Upstream velocity (m/s).

The freeboard recommendations should be applied as detailed in Table 15.6.

Table 15.6: Minimum Freeboard Recommendations for Inlets and Junctions

Pit Types	Freeboard
Gully pit on Grade	0.15m below invert of curb and channel. (See Notes 1 and 2).
Gully pit in Sag	0.15m below invert of curb and channel. (See Note 1)
Field pit	0.15m below top of grate or lip of pit
Junction Structure (See Note 3)	0.15m below top of lid.

Notes:

1. Where the channel is depressed at a gully inlet the freeboard should be measured from the theoretical or projected invert of the channel.
2. Where an inlet is located on grade the freeboard should be measured at the centreline of the gully inlet chamber.
3. Where it is necessary for the HGL to be above the top of a manhole or junction structure, a bolt-down lid should be provided. This will, of course, prevent the use of the manhole as an inlet.

15.4 DESIGN CRITERIA

The design of pipe drainage system should conform to the following criteria:

- Pipe are designed by a 'Hydraulic Grade Line' (HGL) method using appropriate pipe friction loss and structure head loss;
- If the potential water surface elevation (in junction/pit) exceeds 0.15m below ground elevation for the design flow, the top of the pipe, or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the hydraulic grade line;
- The maximum hydraulic gradient should not produce a velocity that exceeds 6m/s and the minimum desirable pipe slope should be 1.0% to provide self cleansing and free from accumulation of silt;
- Minimum diameter for pipe draining a stormwater inlet and crossing a footpath alignment shall be no less than 300mm;
- To allow for passage of debris, the minimum diameter shall not be less than 381mm;

- For a non-self draining underpass, the pipe shall be sized for 10 year ARI and shall not be less than 450mm;
- The maximum pipe diameter to be used depends on the availability of pipe from manufacturers. Culverts or multiple pipes should be used in situation where large pipe interfering with clearance for other services;
- Curved stormwater pipelines are only permitted for diameters 1200mm and above; and
- Stormwater pipelines shall be designed for a minimum effective service life of 50 years.
- The most commonly used design ARI values for minor drainage systems are provided in Table 1.1.

The HGL is used to determine the acceptability of a proposed stormwater drainage system by establishing the elevation to which water level will rise when the system is operating under design conditions.

15.5 DESIGN PROCEDURE

The steps generally followed in the design of pipe drains are listed as follows:

Step 1: Prepare a tentative layout plan for the stormwater pipe drainage system which covers:

- Location of stormwater drains and inlets or pits;
- Direction of flow;
- Location of manholes or junction box; and
- Location of existing facilities such as water, gas, or underground cables.

The slope and the length of pipe segments are determined. The pipe lengths for each segment are entered in Column 2 in Table 15.C3 Hydraulic Design Sheet.

Step 2: Determine drainage areas, and by using the Rational Method the pit inlet capacities and the design discharges in each pipe in the system are established and entered into Column 3 in Table 15.C3.

Assume a trial pipe diameter and calculate the full pipe velocity as in Column 5 Table 15.C3. Calculate the friction loss in pipe segment and the pressure/head loss in the pit (Column 10 Table 15.C3).

The procedure of obtaining the pit inlet capacities have been addressed in Chapter 13 and the Rational Method have been described in Chapter 2.

Step 3: Carry out the HGL evaluation procedure (Section 15.6) for the system starting at a point where the HGL may be readily determined, either working upstream from the outlet or proceeding downstream from a starting permissible water level depending on the flow conditions of the system.

Step 4: Check whether the design criteria in Section 15.4 (minimum velocity/slope and freeboard) are complied with, and carry out the necessary adjustments.

Step 5: It is a good practice to test the adopted configuration and pipe sizing with discharge of higher ARI. The risk factor is therefore defined.

15.6 HGL CALCULATION PROCEDURE

A step-by- step procedure for manual calculation of the HGL using the energy loss method is presented in this section. Hydraulic Design Sheet Table 15.C3 in Appendix 15.C is used in the organisation of data and calculations. A single line on the computation sheet is used for each junction or structure and its associated outlet pipe.

Columns 1 to 6 in Table 15.C3 present the basic design information, while Columns 7 to 16 are for calculation of HGL position. The remaining columns are used to determine pipe invert levels, allowing for hydraulic considerations, cover and positions of upstream pipes. Pipes slope are calculated to check for sedimentation problem. The table ends with remarks and actions taken.

Here the storm drain systems are designed to function in a subcritical flow regime. In subcritical flow, pipe and the junction losses are summed to determine the upstream HGL levels or subtracted to determine the downstream HGL depending where we begin computation.

If supercritical flow occurs, pipe and junction losses are not carried upstream. When a storm drain section is identified as being supercritical, the designer should advance to the next upstream pipe section to determine its flow regime. The process continues until the storm drain system returns to a subcritical flow regime (Hec-22, 2009). The flow is supercritical when the Froude Number of the flow is greater than one ($Fr>1$) or when the normal depth (y_o) of flow is less than its critical depth (y_c). For illustration, the HGL computations begin at the outfall and are worked upstream taking each junction into consideration. The HGL at the outfall, in most cases are readily determined. Furthermore, the head losses in pits and junction are expressed as a function of the velocity in the downstream pipe.

The HGL computation for pipe flow is in accordance with the followings:

Step 1: Identify the tailwater elevation (*TW*) at the downstream storm drain outlet. If the outlet is submerged, the HGL will correspond to the water level at the outlet. If the outlet is not submerged, the HGL are computed as follows:

- If the TW at the pipe outlet is gr eater than $(d_c + D)/2$, the *TW* elevation taken as the HGL; and
- If the TW at the pipe outlet is less than $(d_c + D)/2$, use $(d_c + D)/2$ plus the invert elevation as HGL.

Where d_c is the critical depth determine from Chart 15.A3 for circular conduit.

Step 2: Calculate full flow velocity (V), and hence the velocity head ($V^2/2g$).

Step 3: Compute the friction slope (S_f) for the pipe using Equation 15.2b.

Step 4: Compute the friction loss (H_f) by multiplying the length (L) by the friction slope.

Compute other losses along the pipe run such as bend losses (h_b), transition Contraction (H_c) and expansion (H_e), and junction losses (H_j) using Equations 15.5 to 15.11, or use Charts 15.A9 to 15.A11 in Appendix 15.A.

Step 5: Compute the EGL at the upstream end of the outlet pipe as the HGL plus the total pipe losses plus the velocity head.

Step 6: Estimate the depth of water in the access hole as the depth from the outlet pipe invert to HGL in pipe at outlet. It also can be computed as EGL minus the pipe velocity head minus the pipe invert.

Step 7: If the inflow storm drain invert is submerged by the water level in the access hole, compute the access hole losses using Equation 15.3 and 15.4. The value of pressure change coefficients (K_u) for various configurations can be obtained from Charts 15.A5 to 15.A7 or Equation 15.5. Iterative calculation may be required in high precision case to obtain the pressure change coefficient, since pressure loss depends on depth, Flowchart 15.B1 in Appendix 15.B.

If the inflow storm drain invert is not submerged by the water level in the access hole, the head in the manhole is computed using culvert techniques, depending whether the outflow pipe under outlet or inlet control. For outlet control, coefficient obtained from Table 15.A12 in Appendix 15.A.

Step 8: Compute the EGL at the structure by adding the structure losses to EGL at the upstream end of the outlet pipe (step 5).

Step 9: Compute the HGL at the structure by subtracting the velocity head from the EGL (Step 8).

Table 15.C3 in Appendix 15.C summarised the calculation steps.

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APPENDIX 15.A DESIGN CHART, NOMOGRAPHS AND TABLES

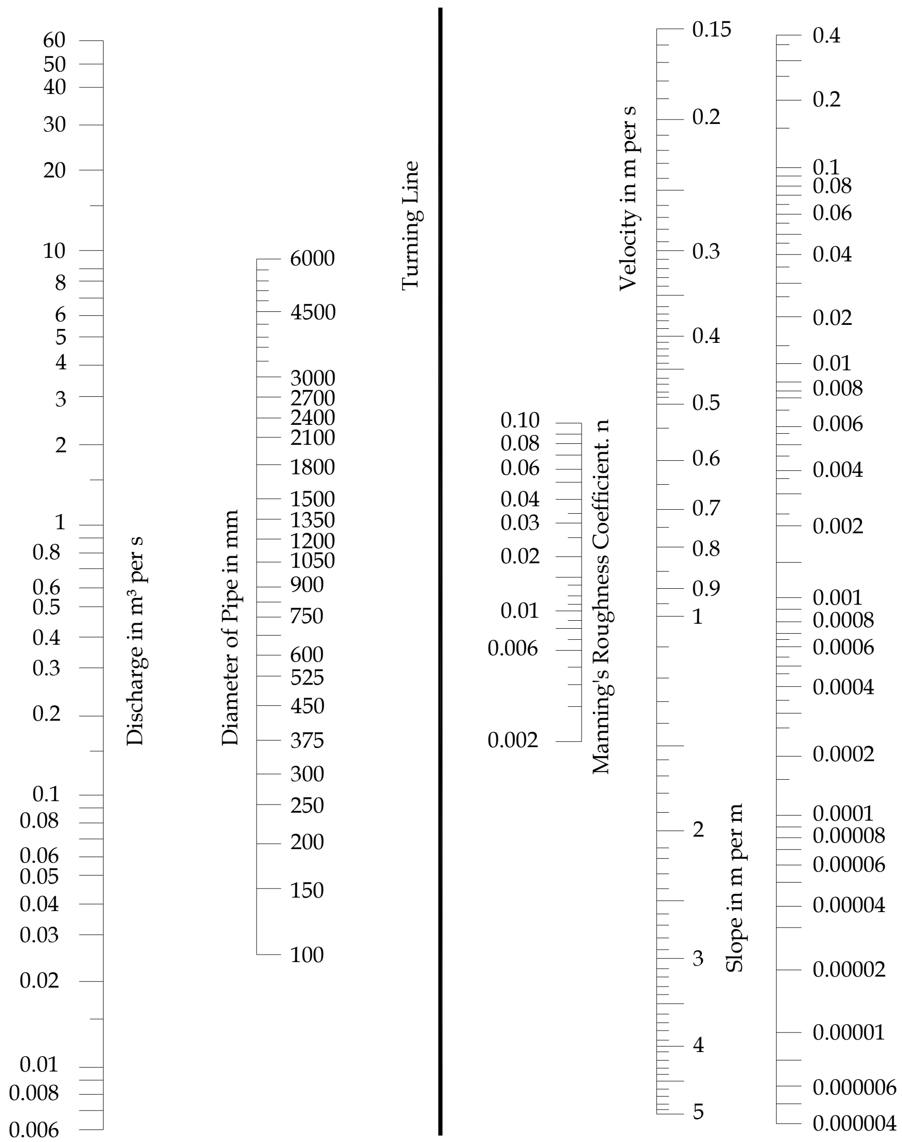


Chart 15.A1: Hydraulic Design of Pipe- Manning Formula

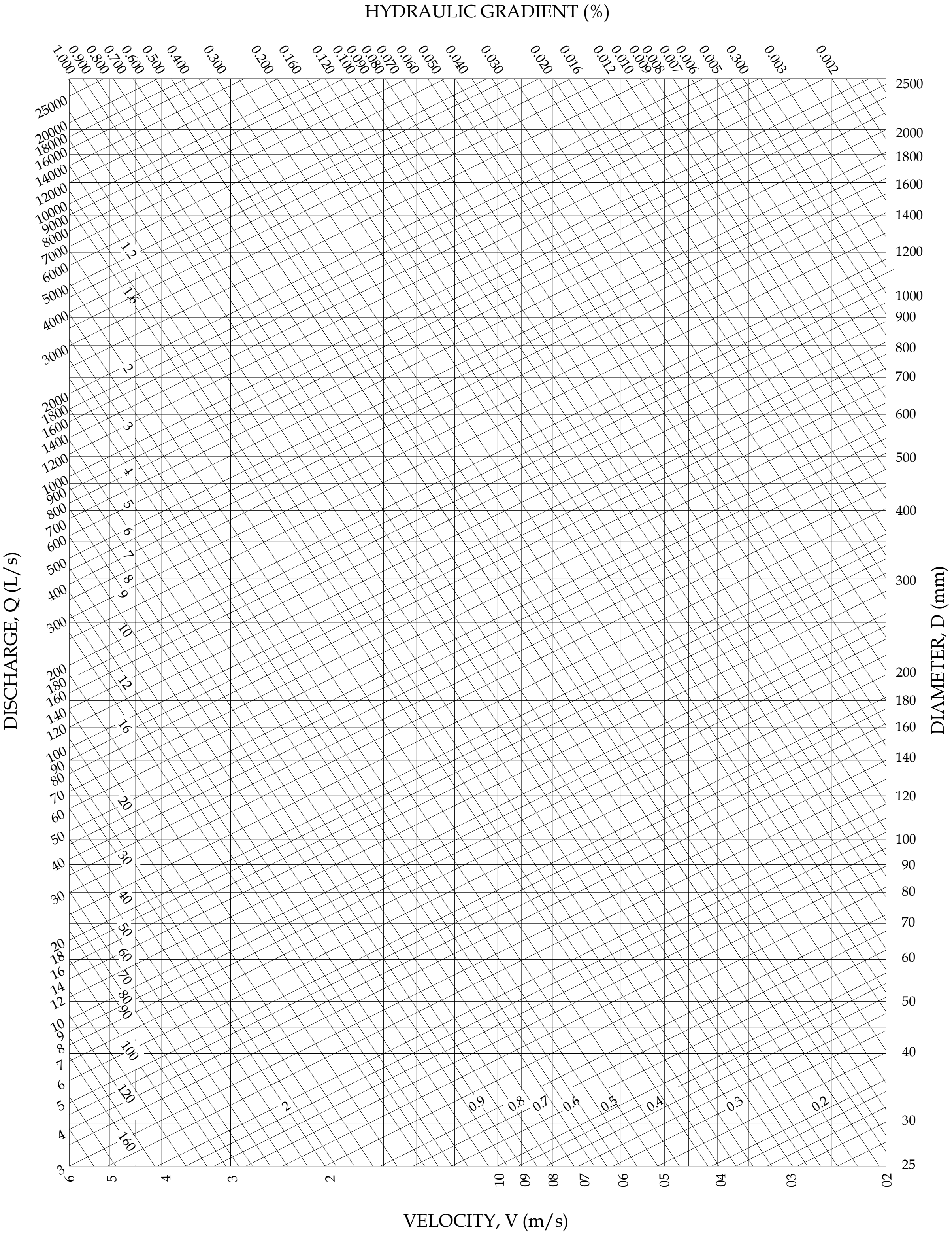


Chart 15.A2: Hydraulic Design of Pipes – Colebrook-White Formula – *k* = 0.30 mm

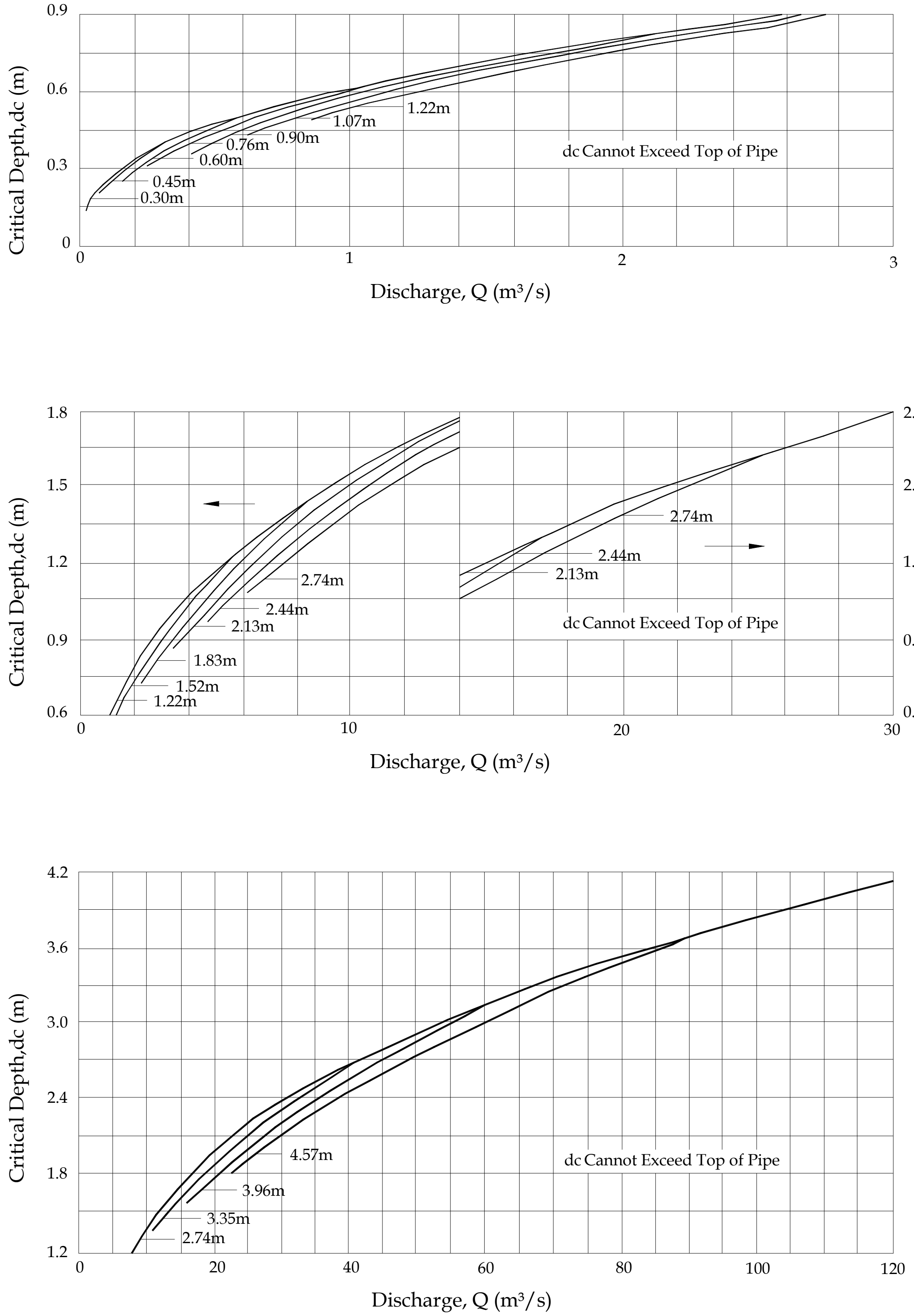


Chart 15.A3: Critical Depth in Circular Pipe

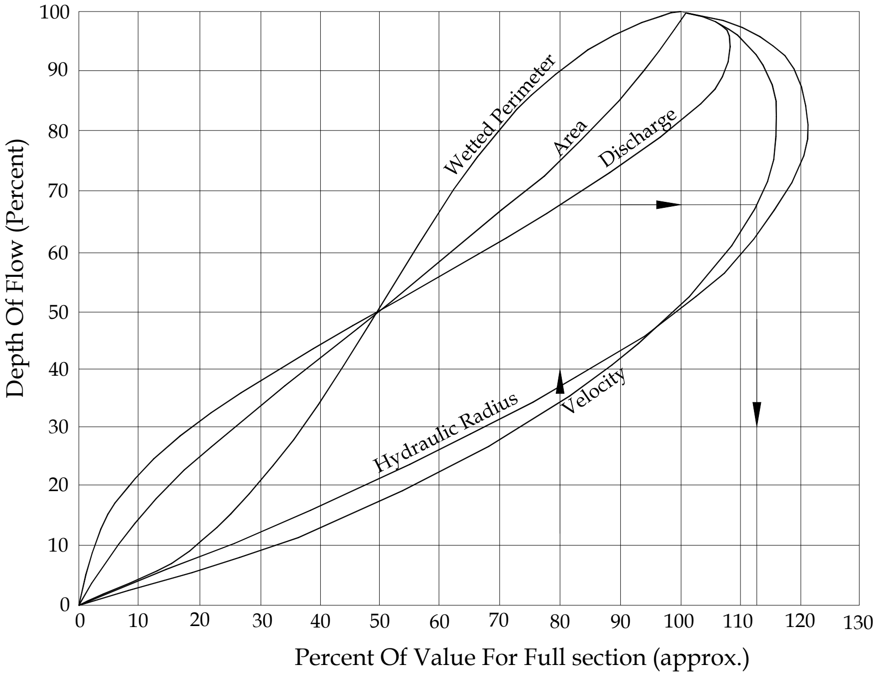
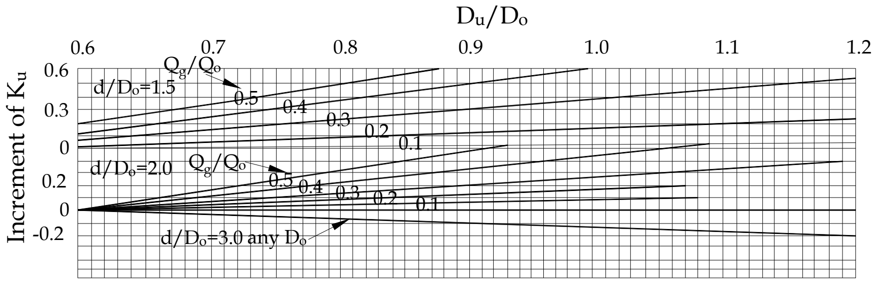


Chart 15.A4: Hydraulic Element Chart



Supplementary Chart for Modification of K_u
for Depth in Inlet other than $2.5D_o$

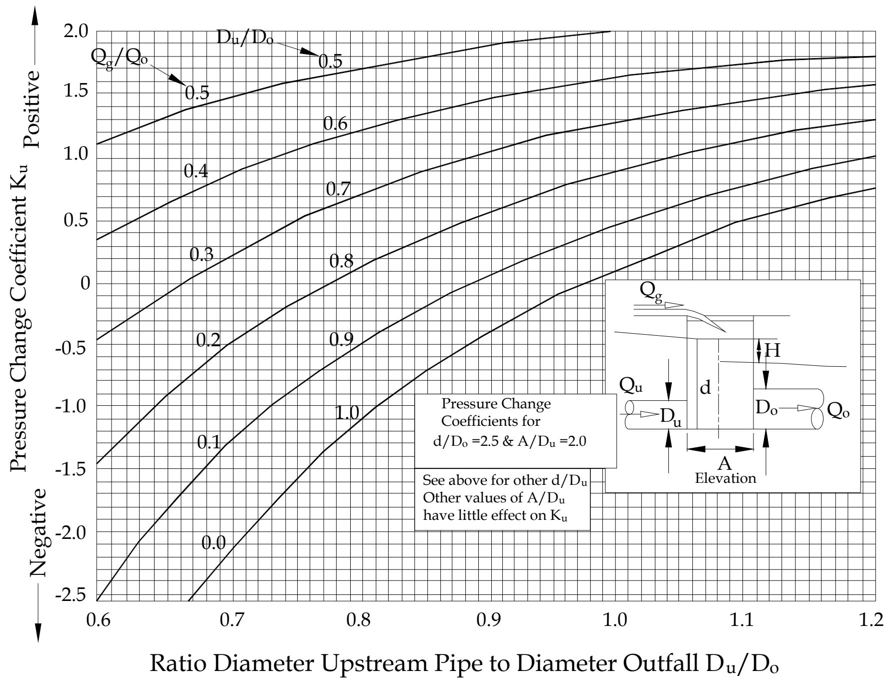
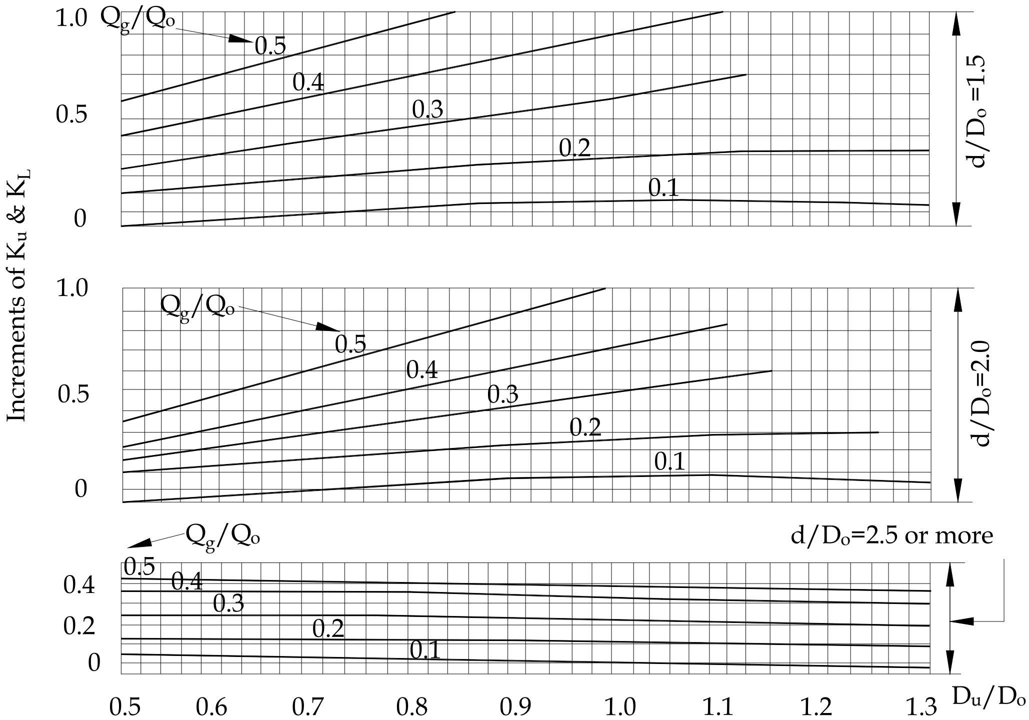


Chart 15.A5: Pressure Change Coefficient for Rectangular Inlet with Through Pipe and Grate Flow
(Sangster et al, 1958)



Supplementary Chart for Modification of K_u & K_L for Grate Flow

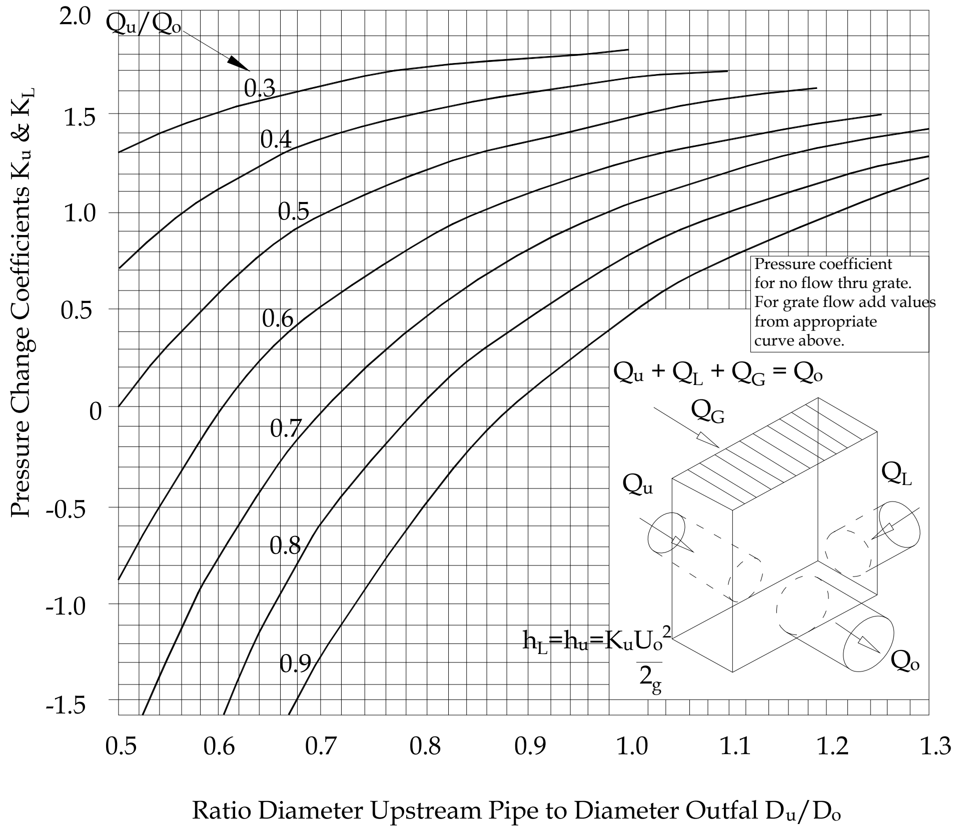


Chart 15.A6: Pressure Change Coefficients for Rectangular Inlets with In-line Upstream Main and 90° Lateral Pipe (with or without grate flow), (Sangster et al, 1958)

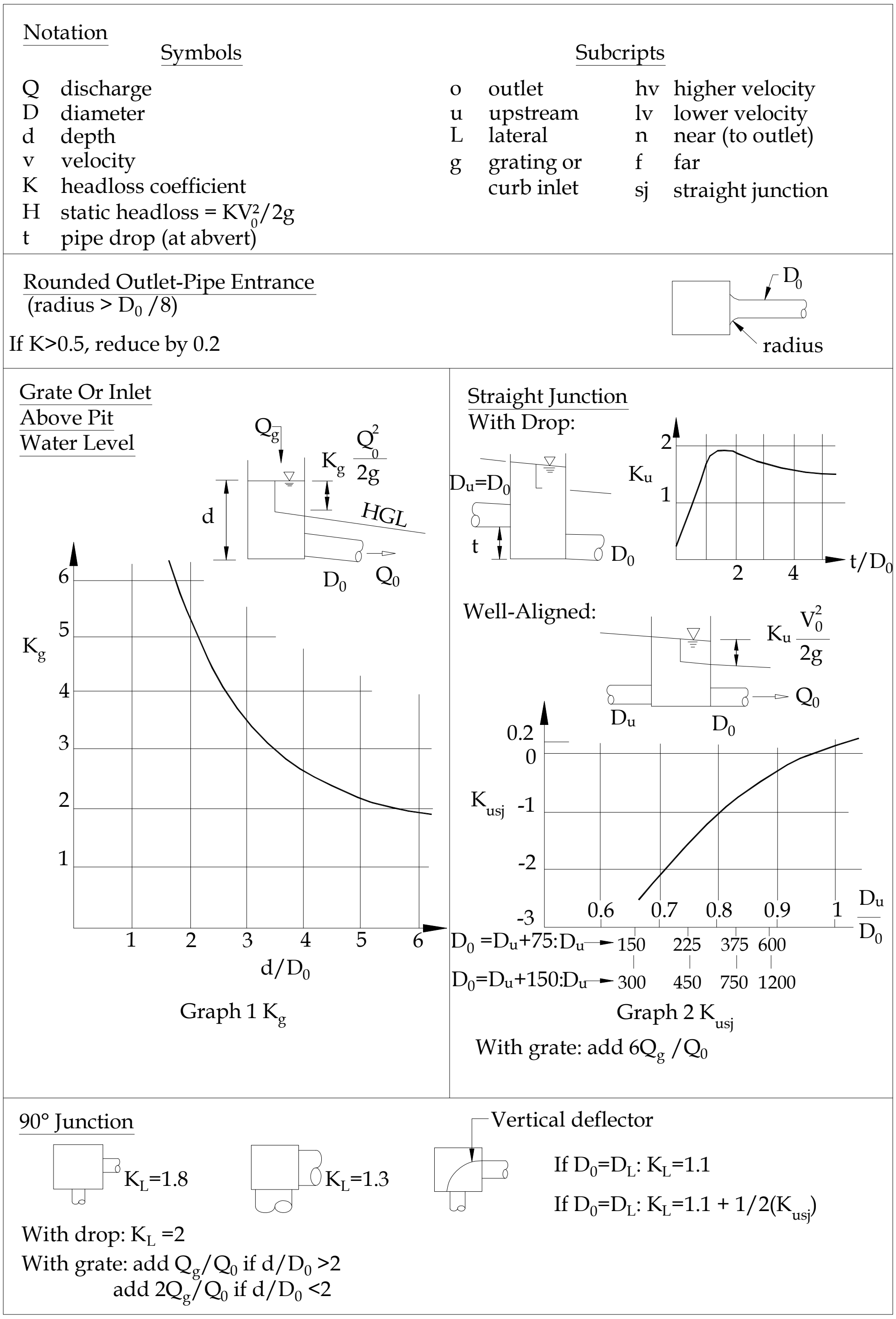
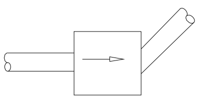
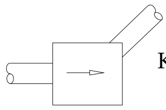


Chart 15.A7(a): Pit Static Head Loss Coefficients

45° Junction



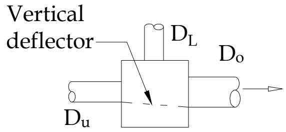
$K_u = 0.6 + \frac{1}{2}K_{usj}$



$K_u = 2.1$

With grate: add Q_g/Q_o if $d/D_o>2$
add $2Q_g/Q_o$ if $d/D_o<2$

Straight Junction With 90° Lateral

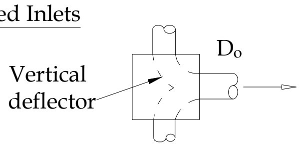


Q_u/Q_o		.3	.6	.9	1.0
$Q_o=Q_u$	no deflector: $K_u=K_L$ =	1.8	1.3	.5	.2
	with deflector: $K_u=K_L$ =	1.4	1.0	.4	.2

If $D_o>D_u$: add $\frac{1}{2}(Q_u/Q_o)(K_{usj})$ (actually reduces K)

With grate: add Q_g/Q_o if $d/D_o>2$
add $2Q_g/Q_o$ if $d/D_o<2$

Opposed Inlets

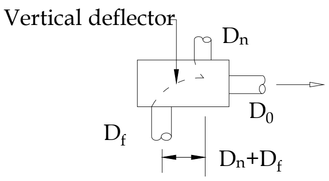


K_{hv} =1.8

Q_{hv}/Q_o	.3	.6	.9
$D_o=D_{hv}$:K_{1v}=1.5		2.0	3.0
$D_o>D_{hv}$:K_{1v}=1.5		3.0	4.5

With grate : no change.
With deflector : subtract 0.3, and $K_{1v} \ngtr 2$

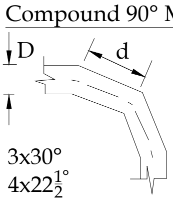
Opposed Offset Inlets

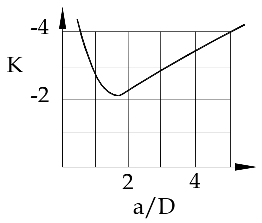


	K_n	K_f
Q_n is Q_{hv} :	1.6	1.9
Q_n is Q_{1v} :	1.6	2.4

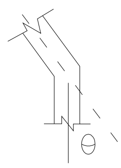
With grate : add 0.2
With deflector : subtract 0.3

Compound 90° Mitre Bend (Splayed)





Mitre Bend



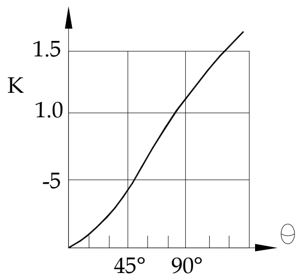


Chart 15.A7(b): Pit Static Head Loss Coefficient

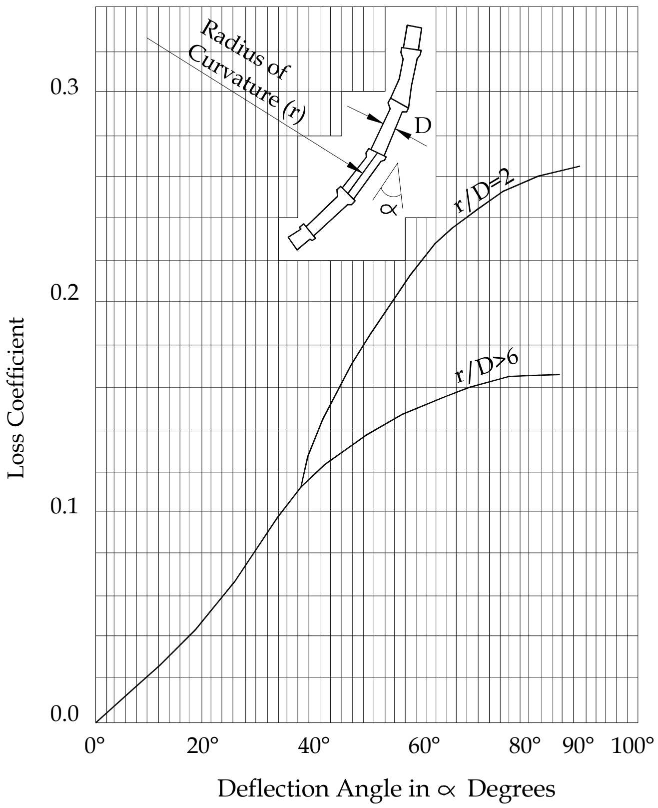
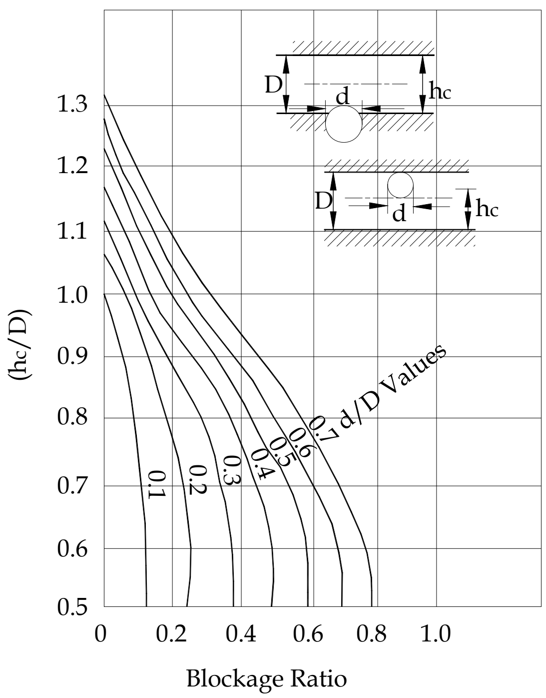


Chart 15.A8: Bend Loss Coefficients Source(D.O.T., 1992)



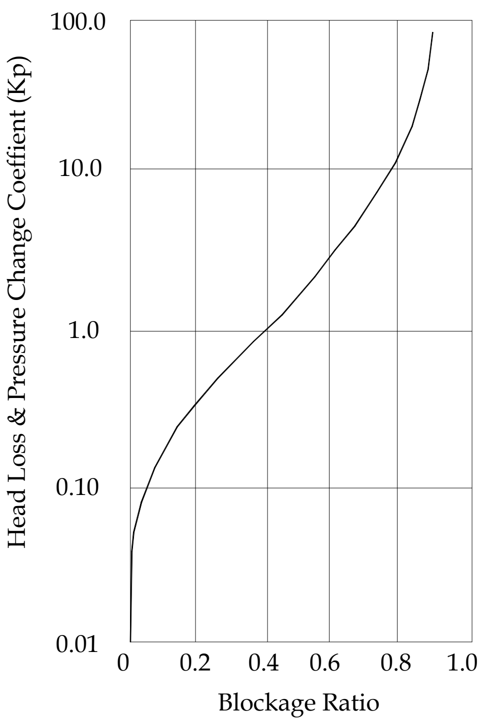
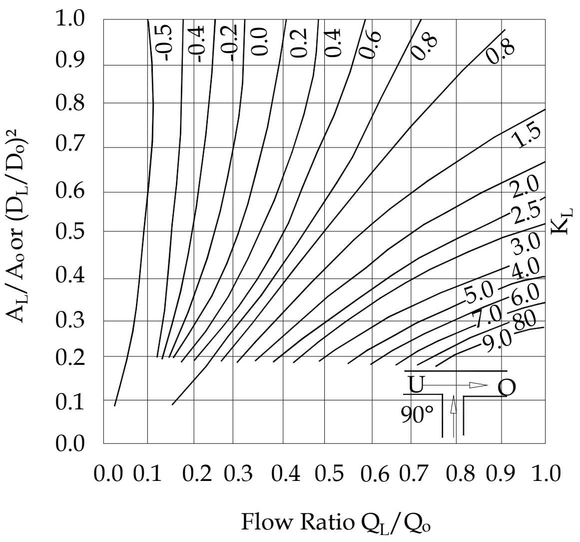


Chart 15.A9: Penetration Loss Coefficients (Black, 1987)



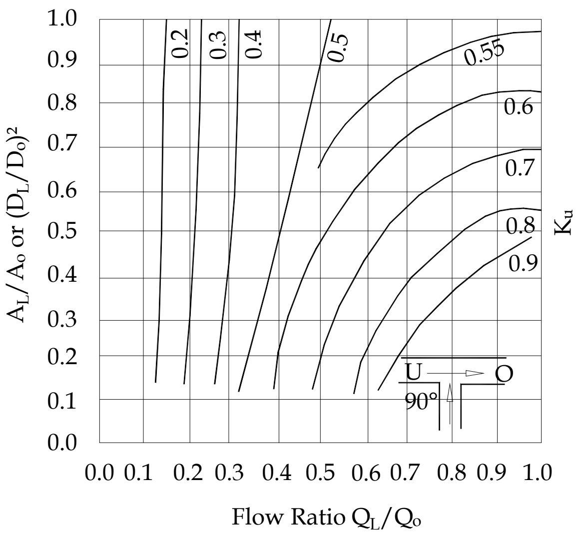


Chart 15.A10: Pressure Loss Coefficients at Branch Lines Source

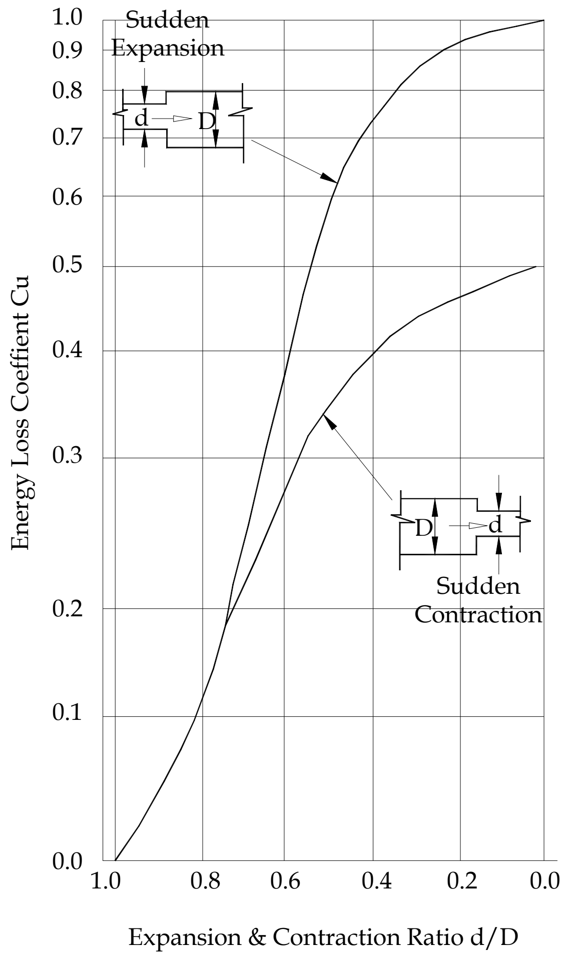


Chart 15.A11: Expansion and Contraction Loss Coefficients

Table 15.A1: Entrance Loss Coefficients

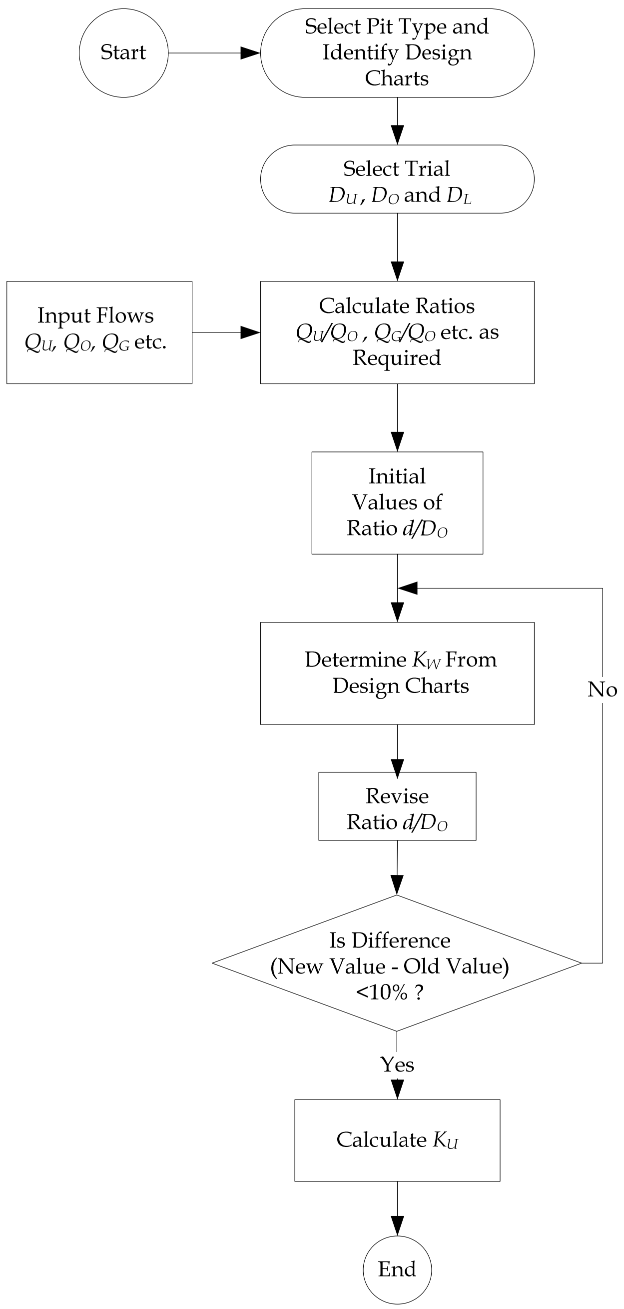
Type of Structure and Entrance Design	Coefficient K_e
Concrete Pipe	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls:	
Socket end of pipe (groove end)	0.2
Square edge	0.5
Rounded (radius = D/12)	0.2
Mitred to conforming to fill slope	0.7
End section conforming to fill slope	0.5
Hooded inlet projecting from headwall	See note
Corrugated Metal Pipe	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwall square edge	0.5
Mitred to comform to fill slope	0.7
End section conforming to fill slope	0.5
Reinforced Concrete Box	
Headwall parallel to embankment (no wingwalls)	
Square edges on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension	0.2
Wingwalls at 30° to 75° to barrel	
Square edged at crown	0.4
Crown edge rounded to radius 1/12 barrel dimension	0.2
Wingwalls at 10° to 25° to barrel	
Square edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square edged at crown	0.7

Note : Refer Argue (1960) and O'Loughlin (1960)

Table 15.A2: Pressure Loss Coefficient at Mitred Fittings (ARR, 1987)

Type	K_b
90° double mitred bend	0.47
60° double mitred bend	0.25
45° single mitred bend	0.34
22.5° single mitred bend	0.12

APPENDIX 15.B DESIGN FLOW CHARTS



Flowchart 15.B1: Procedure for K_u and K_w Calculation

APPENDIX 15.C EXAMPLE – PIPE DRAINS DESIGN

Problem:

It is required to estimate the size and slope of the proposed pipe system to discharge the surface runoff from an urban area as shown in Figure 15.C1. The landuses of the area comprise of high density residentials, roads, open space and a service station. Through hydrologic data analysis performed from each pits the flow rates in the proposed pipelines are estimated for design use. Pits level and pipe segment flow rates are given in Table 15.C1 and 15.C2, respectively.

Table 15.C1: The Proposed Levels at Pits.

Pit No.	Surface Level (m)
1	28.02
2	26.23
3	24.51
4	24.48
5	24.38
6	24.38

Table 15.C2: Estimated Flow Rates

Pipe Segment	Flow Rates (L/s)
1 – 2	167
2 – 4	243
3 – 4	56
4 – 5	375
5 - 6	451

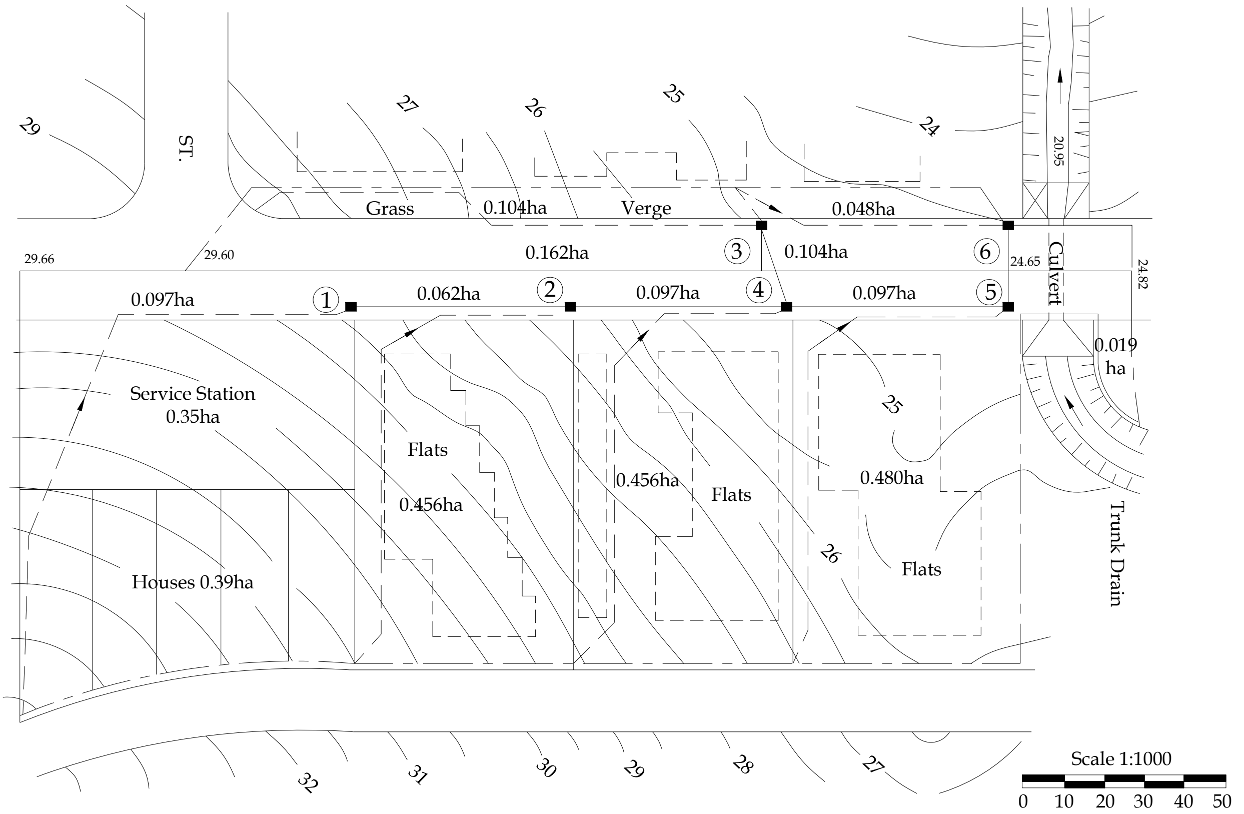


Figure 15.C1: Proposed Pipe Drains Layout

Solution:

The calculation starts at the upstream pit and moves downstream. Figure 15.C2 shows a pipe reach on which features are identified and linked to various column of the calculation sheet in Table 15.C3. Column 1 to 6 present the basic design information, while Column 7 to 16 are for calculation of the hydraulic grade line (HGL) position. The remaining columns are used to determine pipe invert levels, allowing for hydraulic considerations, cover and positions of upstream pipes. Pipes slopes are calculated to check for sedimentation problems. The calculation steps are given below.

The calculation start at the upstream pit with a trial pipe diameter of 381 mm being selected. This diameter is normally specified as minimum diameter to allow for passage of debris. The freeboard of 0.15m below the surface level is adopted for water levels in pits.

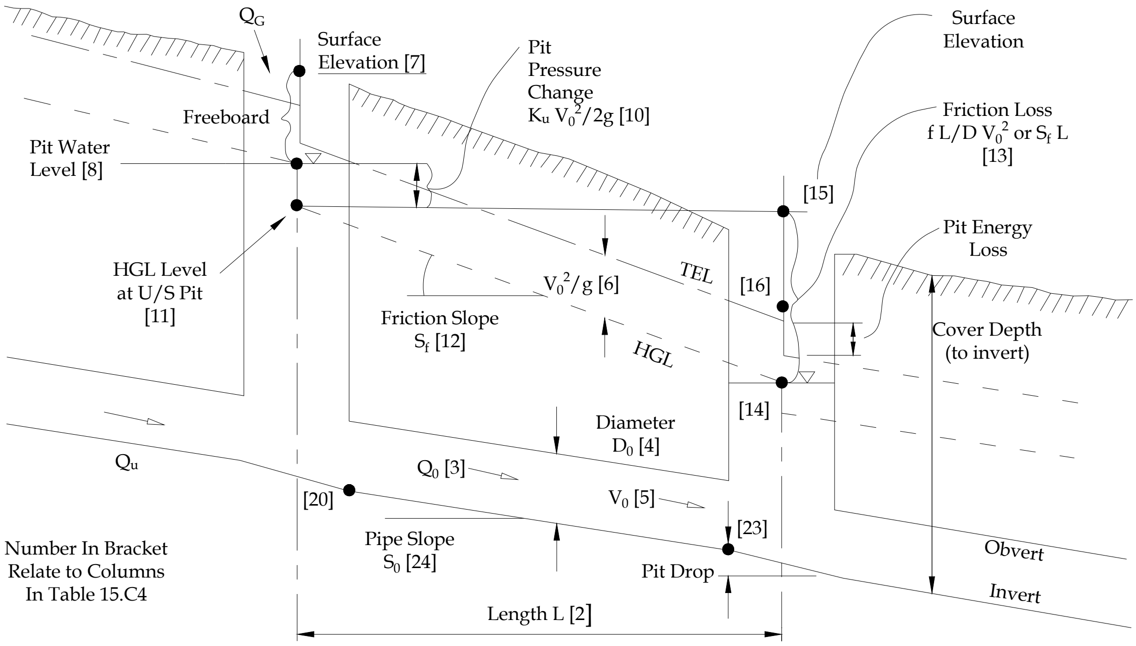


Figure 15.C2: Pipe Reach Showing Features Identified in Calculations

References	Calculation	Output
	Row 1 Column 1 : Pipeline 1-2 Column 2 : Pipe length L = 54.8 m Column 3 : Design Flow rate Q_{1-2} = 0.167 m³/s Column 4 : Trial diameter D_{1-2} = 0.381 m Column 5 : Full pipe velocity $V = Q/A$ = 0.167/ 0.114	$A=\frac{\pi D^2}{4}=\frac{\pi x(0.381)^2}{4}=0.114m^2$ V = 1.465m/s

	Column 6 : $V^2/2g = 1.465^2/2g$	$V^2/2g$ = 0.109m
	Column 7 : u/s Pit Surface = 28.02 m	u/s Pit Surface = 28.02 (Pit 1)
	Column 8 : Allowable water level in u/s Pit (Column 7 – freeboard) = u/s Pit Surface – Free board (0.15 m) = 28.02 – 0.15	WL in Pit 1 = 27.87m
	Column 9 : To find Pressure change coefficient K_u Assume depth in pit, d = 1.0	
Chart 15.A7(a)	$d/D_O = 1/0.381 = 2.63$	$K = K_g = K_u = K_w = 4.0$
Equation 15.4	Column 10: $H = \Delta P/\gamma = K_u\ V^2/2g = 4.0 \times 0.109$ Headloss in pit, H = 0.438m	H = 0.438m
	Column 11: HGL = Column 8 – Column 10 = 27.87 – 0.438	HGL = 27.432m at Pit 1
	Column 12: HGL Slope (S_f) for uniform subcritical flow from Manning's or Colebrook-White equation or charts.	
Equation 15.2	Using Equation 15.2, k = 0.3mm, Q = 167 L/s V = 1.465 m/s D = 381 mm	S_f = 0.553 % = 0.00553m/m
	Column 13: Friction energy loss $h_{f1-2} = S_f \times L = 0.00553 \times 54.8$	h_{f1-2} = 0.303m
	Column 14: HGL at the d/s pit = HGL at Pit 1 - h_f = 27.432 – 0.303	HGL at Pit 2 = 27.129m
	Column 15 : Surface level at the d/s pit	d/s Pit Surface = 26.23m
	Column 16 : Allowable water surface in d/s pit (Pit 2) is the lower of (i) Surface level – freeboard = 26.23 – 0.15 = 26.08m or (ii) HGL at pit = 27.129m	WL at pit 2 = 26.08m
	Column 17 : Calculation of pipe invert level for the u/s Pit (Pit 1) by hydraulic requirement; column 11 – column 4 = 27.432 – 0.381 = 27.051m	u/s invert level (Pit 1) Hydraulic requirement = 27.051m
	Column 18: Calculation of invert level for the u/s Pit (Pit 1) by cover requirement; Column 7– cover depth	u/s invert level (Pit 1) cover requirement = 27.007m

	(Note : cover depth = depth from surface to pipe crown + pipe thickness + internal diameter) = 28.02 – 0.6 – 0.032 – 0.381 = 27.007m	
	Column 19 : u/s Pit invert level minus any allowance for slope across the Pit, (drop) [Note : Recommended drop across pit = 0.03m]	no drop on slope of Pit 1
	Column 20 : Adopted u/s Pit invert, ie the lowest of [17], [18] and [19]	Adopted u/s Pit invert level (Pit 1) = 27.007m
	Column 21 : d/s pit invert level based on hydraulic requirement. Column 16 – Column 4 = 26.08 – 0.381 = 25.699 m	d/s Pit invert level by hydraulic requirement = 25.699m
	Column 22 : d/s pit invert level based on cover requirement. Column 15 – cover = 26.23 – 0.6 – 0.032 – 0.381 = 25.217 m	d/s pit invert level by cover requirement = 25.217m
	Column 23 : Adopted d/s pit invert level, i.e the lower of the values in Columns 21 & 22	Adopted d/s invert level (Pit 2) = 25.217m
	Column 24 : Calculate the $pipe_{1\text{-}2}$ Slope = (Column 20-Column 23) /Length = (27.007 – 25.217)/54.8 = 0.0327	$Pipe_{1\text{-}2}$ Slope = 0.327m/m
	Column 25 : Remarks	Pipe levels set by cover considerations
	Note : (1) The process are repeated for Pipe 2-4, 3-4,4-5 and 5-6 and the results are shown in Table 15.C3. (2) The assumptions made during calculation should now be re-checked. The assumed values of S and S/D_o in this example were reasonable. If not, they should be amended and the calculations repeated.	

Table 15.C3: Hydraulic Design Sheet

[1] Pipe	[2] Length L (m)	[3] Design Flow-rate Q (L/s)	[4] Trial Pipe Diameter D (m)	[5] Full Pipe Velocity V (m/s)	[6] $V^2/2g$ (m)	[7] U/S Surface Level (m)	[8] U/S Pit Water Level Limit* (m)	[9] Pit Pressure Change Coeff. k_u or k_w	[10] $k.V^2/2g$ (m) [9] x [6]	[11] HGL At U/S Pit (m) [8] – [10]
1-2	54.8	167	0.381	1.465	0.109	28.02	27.870	4.0	0.438	27.432
2-4	55.0	243	0.381	2.131	0.232	26.23	26.080	0.2	0.046	26.034
3-4	18.4	56	0.381	0.491	0.012	24.51	24.360	4.5	0.055	24.305
4-5	56.2	375	0.381	3.289	0.552	24.48	24.292	0.5	0.276	24.016
			0.457	2.286	0.267	24.48	24.292	0.3	0.080	24.212
5-6	17.0	451	0.457	2.750	0.386	24.38	23.615	1.7	0.656	22.959
			0.533	2.021	0.208	24.38	23.615	1.5	0.313	23.302

Table 15.C3: Hydraulic Design Sheet (Cont.)

[1]	[12]	[13]	[14]	[15]	[16]	[17]	[18]	[19]	[20]
						U/S Invert Levels (m)			
Pipe	HGL Slope S_f (m/m)	Pipe Friction Loss S_f . L [12] x [2]	HGL At D/S Pit (m) [11]-[13]	D/S Pit Surface Level (m)	D/S Pit Water Level Limit** (m)	Hydraulic [11]-[4]	Cover [7] - cover	U/S Pipe [23] – drop (0.03m)	Adopted Lowest of [17][18] & [19]
1-2	0.00553	0.303	27.129	26.23	26.080	27.051	27.007	-	27.007
2-4	0.01158	0.637	25.397	24.48	24.330	25.653	25.217	25.187	25.187
3-4	0.00066	0.012	24.292	24.48	24.292	23.924	23.497	-	23.497
4-5	0.02730	1.536	22.480	24.38	22.480	23.635	23.467	23.283	23.283
	0.01063	0.597	23.615	24.38	23.615	23.755	23.385	23.283	23.283
5-6	0.01531	0.260	22.699	24.38	22.699	22.502	23.285	22.691	22.502
	0.00690	0.117	23.185	24.38	23.185	23.769	23.205	22.691	22.691

** Lower of [14] or ([15] – Freeboard)

Table 15.C3: Hydraulic Design Sheet (Cont.)

[1]	[21]	[22]	[23]	[24]	[25]
	D/S Invert Levels (m)				
Pipe	Hydraulic [16]-[4]	Cover [15]- cover	Adopted Lower of [21] & [22]	Pipe Slope S_o $\frac{[20]-[23]}{[2]}$	Remarks
1-2	25.699	25.217	25.217	0.0327	Pipe levels set by cover consideration
2-4	23.949	23.467	23.467	0.0313	Cover still controls
3-4	23.911	23.467	23.467	0.0016	Too flat - adjust to 1% slope
			23.313	0.0100	
4-5	22.099	23.367	22.099	0.0211	Use 457mm, but adjust to 1% slope
	23.158	23.285	23.158	0.0022	
			22.721	0.0100	
5-6	22.242	23.285	22.242	0.0153	Use 533mm and adjust to 1% slope
	22.652	23.205	22.652	0.0023	
			22.521	0.0100	

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