city of swan

HANDBOOK OF STORMWATER DRAINAGE DESIGN

(Part of AUS-SPEC #1 Design Specification and should be read in conjunction with Document D5 Stormwater Drainage Design)

Amendment Record for this Specification Part

This Specification is Council's edition of the AUS-SPEC generic specification part and includes Council's primary amendments.

Details are provided below outlining the clauses amended from the Council edition of this AUS-SPEC Specification Part. The clause numbering and context of each clause are preserved. New clauses are added towards the rear of the specification part as special requirements clauses. Project specific additional script is shown in the specification as italic font.

The amendment code indicated below is 'A' for additional script 'M' for modification to script and 'O' for omission of script. An additional code 'P' is included when the amendment is project specific.

Amendment Sequence No.	Key Topic addressed in amendment	Clause No.	Amendment Code	Author Initials	Amendment Date
1	**Stormwater Drainage Design**	1	M,A	AW	Nov 2004
2	**PERCENTAGE IMPERVIOUS & RUNOFF COEFFICIENTS**	3	M,A	AW	Nov 2004
3	**Detention Requirements**	11	M,A	AW	Nov 2004
4	**Map Coefficients**	2.1	M	AW	June 2005
5	**Introduction**	1	M,A	YKW	Jan 2008
6	**Percentage Impervious & Runoff Coefficients**	3	M,A,O	YKW	Jan 2008
7	**Hydrological Calculations**	4.1	A	YKW	Jan 2008
8	**Gutter Flow**	6	A	YKW	Jan 2008
9	**Stormwater Detention Design**	11	M, A	YKW	Jan 2008
10	**Soakwells**	7	M, A	YKW	Jan 2011
11	**Stormwater Drainage Design**	1	M,A	AW	Jan 2012
12	**PERCENTAGE IMPERVIOUS & RUNOFF COEFFICIENTS**	3	M,A	AW	Jan 2012
13	**Soakwells**	7	M, A	YKW	Jan 2012
14	**Conduits and materials standard**	12	A	YKW	April 2022

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Stormwater Drainage Design

1. INTRODUCTION

This handbook provides design criteria to be used in submission of drainage designs presented to the City of Swan Council. All basic design principles shall be in accordance with the Australian Rainfall & Runoff (AR&R) current edition. This handbook forms part of Aus-Spec #1. Consultants should read this handbook in conjunction with the AR&R and Document D5 - Stormwater Drainage Design of Aus-Spec #1. The consultant should also consider water quality treatment when carrying out drainage design such as treatment at source where the soil conditions permit. Design principles in accordance with “Better Urban Water Management” produced by the Department of Planning and the guidelines in the “Stormwater Management Manual for Western Australia” produced by the Department of Water should be followed wherever possible.

Intensity Frequency Duration Relationship

2. DESIGN IFD CURVES FOR CITY OF SWAN (Section D5.04)

Following are design intensity-frequency-duration rainfall relationships for the City of Swan urban areas and Bullsbrook/Gidgegannup rural areas. For other areas calculations shall be in accordance with AR&R. These calculations are to be submitted with the design information.

2.1 Urban Areas – Ballajura / Beechboro / Midland

Map coefficients (based on Midland)

2i1	2i12	2i72	50i1	50i12	50i72	G	F2	F50
21.25	4.58	1.34	36.00	7.03	2.20	0.68	4.85	17.1

(Values obtained from maps in Volume 2 of AR&R 1987)

LPIII Rainfall Intensities

ARI Duration	1 Years	2 Years	5 Years	10 Years	20 Years	50 Years	100 Years
5 min	59.3	78.4	103	121	145	182	214
6 min	55.1	72.8	95.3	111	134	167	196
10 min	44.0	57.9	75.0	87.2	104	129	151
20 min	30.9	40.3	51.4	59.1	70.0	86.0	99.5
30 min	24.5	31.8	40.1	45.8	54.0	65.8	75.8
1 hr	15.9	20.5	25.4	28.6	33.4	40.1	45.8
2 hr	10.5	13.5	16.5	18.5	21.4	25.5	29.0
3 hr	8.22	10.5	12.7	14.2	16.4	19.5	22.0
6 hr	5.37	6.82	8.17	9.06	10.4	12.2	13.8
12 hr	3.52	4.44	5.26	5.79	6.59	7.71	8.62
24 hr	2.23	2.83	3.39	3.75	4.30	5.07	5.69
48 hr	1.37	1.75	2.13	2.37	2.74	3.25	3.67
72 hr	1.01	1.29	1.58	1.77	2.05	2.44	2.77

Overland flow travel time aid Table of t.I$^{0.4}$ values for use in conjunction with Technical Note 3, Urban Stormwater Management, AR&R

ARI Duration	1 Years	5 Years	10 Years	20 Years	100 Years
5 min	25.6	31.8	33.9	36.5	42.5
6 min	29.8	37.1	39.5	42.6	49.6
7 min	33.9	42.2	44.9	48.3	56.2
8 min	37.9	47.1	50.1	53.8	62.6
9 min	41.8	51.8	55.0	59.1	68.7
10 min	45.5	56.3	59.8	64.3	74.5
12 min	52.7	65.1	69.0	74.1	85.8
14 min	59.6	73.4	77.8	83.4	96.4
16 min	66.2	81.4	86.2	92.3	107
18 min	72.6	89.1	94.3	101	116
20 min	78.8	96.6	102	109	126

2.2 Rural Areas - Bullsbrook / Gidgegannup

Map coefficients (based on Bullsbrook)

2i1	2i12	2i72	50i1	50i12	50i72	G	F2	F50
20.00	4.00	1.20	36.00	6.75	1.90	0.68	4.80	16.5

LPIII Rainfall Intensities

ARI Duration	1 Year	2 Years	5 Years	10 Years	20 Years	50 Years	100 Years
5 min	55.2	73.3	97.5	115	139	176	208
6 min	51.3	68.1	90.3	106.	129	162	191
10 min	41.0	54.2	713	83.6	101	126	148
20 min	28.7	37.7	49.1	57.1	68.3	85.0	99.2
30 min	22.7	29.8	38.4	44.5	53.0	65.6	76.2
1 hr	14.8	19.2	24.4	28.0	33.1	40.6	46.8
2 hr	9.34	12.1	15.1	17.1	20.1	24.3	27.9
3 hr	7.10	9.14	11.3	12.8	14.9	17.9	20.4
6 hr	4.43	5.66	6.88	7.70	8.89	10.6	12.00
12 hr	2.77	3.52	4.20	4.66	5.33	6.28	7.05
24 hr	1.73	2.17	2.56	2.80	3.18	3.70	4.13
48 hr	1.05	1.31	1.51	1.64	1.84	2.12	2.35
72 hr	0.76	0.94	1.08	1.16	1.3	1.48	1.63

Overland flow travel time aid, Table of t.I$^{0.4}$ values for use in conjunction with Technical Note 3, Urban Stormwater Management, AR&R

ARI Duration	1 Years	5 Years	10 Years	20 Years	100 Years
5 min	24.8	31.8	33.3	36.0	42.2
6 min	29.0	37.1	38.8	41.9	49.1
7 min	33.0	42.2	44.1	47.6	55.7
8 min	36.8	47.1	49.2	53.0	62.0
9 min	40.6	51.8	54.1	58.3	68.1
10 min	44.2	56.3	58.8	63.4	74.0
12 min	51.2	65.1	67.9	73.1	85.3
14 min	57.9	73.4	76.6	82.4	96.0
16 min	64.3	81.4	84.9	91.4	106
18 min	70.5	89.1	93.0	100	116
20 min	76.5	96.6	101	108	126

3. PERCENTAGE IMPERVIOUS & RUNOFF COEFFICIENTS (Section D5.06)

Percentage Impervious & Runoff Coefficients

The following percentage impervious (of total development area) and C_{10} runoff coefficients can be adopted for the various zonings in **clay** areas.

	% impervious	C_{10} runoff coefficient
Rural Residential development	12.5	0.24
Residential development	55	0.56
Medium density development	65	0.64
Commercial development	80	0.75
Industrial development	90	0.82
Roads	64	0.63

Alternately percentage impervious can be calculated for the fully developed catchment and the following table utilized to calculate the C_{10} runoff coefficients.

Fraction impervious	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Runoff coefficient	0.15	0.22	0.3	0.37	0.45	0.52	0.6	0.67	0.75	0.82	0.9

For events of an ARI other than 10 years, the C_{10} Runoff Coefficient shall be multiplied by the frequency factor (F_y) from Table 14.6 on p307 of *Australian Rainfall & Runoff*, 1987 or Table 1.6 on p VIII-19 of *Australian Rainfall & Runoff*, 1998 as indicated below

ARI (years)	1	2	5	10	20	50	100
Frequency Factor Fy ($Fy = C_{ARI}/C_{10}$)	0.8	0.85	0.95	1.0	1.05	1.15	1.2

For residential (R20) development in **sandy** areas a runoff coefficient of 0.8 (constant) over the entire road reserve excluding lots is acceptable. Subsoil drains will be required where the maximum ground water level (MGL) is less than 1.2 metres below the proposed lot levels. The MGL must be determined from on site ground water monitoring and historic records, and must be agreed to by the Department of Water. Refer to Section D5.04 (4& 5) regarding major/minor storm design in Document D5 mentioned in the Introduction.

Hydrology

4. SUMMARY SHEET FOR HYDROLOGICAL CALCULATIONS AND REQUIREMENTS FOR USING COMPUTER ANALYSIS PROGRAMS (Section D5.07)

Information Required

4.1 Summary Sheet for Hydrological Calculations

Hydrological calculations shall be summarised on the sheet in Appendix A. The requested data shall be provided for each subcatchment, and also for the catchment as a whole. Variations to the format are allowed subject to Council approval.

4.2 Requirements for use of Computer Analysis Programs

Requirements for ILSAX are given. For other programs assumptions and criteria used for the computer program shall be submitted.

Standard ILSAX Parameters

4.2.1 ILSAX

Piped and open channel drainage systems in urban areas may be modelled using ILSAX for the minor and major. ILSAX is not suitable for modelling rural residential development. The drainage system shall be modelled for both the minor and major event, using design durations of 5, 10, 15, 20, 30, 45, 60, 90, 120 and 180 minutes. These events may be modelled as a "stacked storm". The appropriate rainfall intensities for the minor and major events shall be selected from section 2 of this handbook.

Parameters that shall be adopted for the rainfall input file are:

Global value of manning's "n"	0.015 for concrete pipes 0.011 for PVC pipes
New pipe manning's "n"	0.013 for concrete pipes 0.011 for PVC pipes
Paved depression storage	2mm
Grassed depression storage	5mm
Soil type	3.0 (clayey soil) 1.0 (sandy soil)
Antecedent moisture content	3.0 (for all events)
Rainfall distribution type	5 (AR&R)
Rainfall zone	8

Parameters that shall be adopted for the rainfall input file are:

Grassed and paved time of entry	To be determined for site using kinetic wave equation and noted in comment line of input: otherwise use 5 min. for paved and 10 min. for grassed surfaces.

Hydraulics

5. HYDRAULIC CALCULATIONS (Section D5.08)

Hydraulic calculations shall be summarised on the sheet in Appendix A. The requested information shall be provided for each inlet pit, junction and other hydraulic constraint. Variations to the format are allowed subject to Council approval.

Gutter Flow Widths

6. GUTTER FLOW (Section D5.09)

The maximum discharge in the kerb and gutter at any one point shall not exceed 15 litres ($0.015m^3$) per second and/or the width of flow shall not exceed the following:

Crown	2.0m
One way crossfall	1.5m

Council will accept design based on Main Roads WA Standards on Piped Systems (Document 67-08-75); accessible on-line at the Main Roads WA web site.

Inlet Characteristics

7. PIT CAPACITIES (Section D5.10)

For design purposes, pit capacities shall be reduced by the following factors to account for clogging:

On-grade grated kerb inlet pit	- multiply by 0.9 (10% reduction)
On-grade grated pit	- multiply by 0.5 (50% reduction)
Sag grated pit	- multiply by 0.5 (50% reduction)
Sag grated kerb inlet pit	- kerb inlet capacity only

Pit capacities shall be determined using the chart shown in Figure 1 at the end of section 15 for on-grade pits, and the equations shown below for grated sag pits and side entry sag pits.

For side entry sag pit

$$Q_i = 1.66\ L\ h^{1.5}$$

where

Q_i = Maximum inlet capacity m^3/s
L = Lintel length (assumes grate blocked as per section D5.10)
h = Depth at kerb for maximum allowable gutter flow width

For grated sag pit

$$Q_i = 0.6\ A\ C\ (19.6\ h)^{0.5}$$

where

Q_i = Maximum inlet capacity m^3/s
A = Area of clear opening m^2
h = Depth at centre of grate for allowable gutter flow width
C = Clogging factor (0.5 for grated sag pit)

For a combined side entry and grated pit

$$Q_i = 1.66Lh^{1.5} + 1.66Ph^{1.5}$$

Where Q_i = Maximum inlet capacity in m^3/s
L = Lintel Length
H = depth at kerb or ponding
P = effective permeter around grate

The formula is valid only if depth of ponding is not greater than 1.4 times the height of kerb inlet.

Soakwells

Soakwell capacity in sandy areas can be calculated using the following formulae:

$$\mathbf{V = 0.0122A_i} \text{ (for R25 or lower density coding)}$$

(5 year 10 minute storm)

$$\mathbf{V = 0.0159A_i} \text{ (for commercial/industrial/R30 or higher density coding)}$$

(1 year 60 minute storm which is roughly equivalent to the 10 year 12 minute storm)

Where V = Volume of soakwell required in cubic metres
A_i = Impervious area (roof, pavement,etc) in square metres

Missouri Charts

8. PRESSURE CHANGE COEFFICIENT CHARTS (Section D5.11)

Appropriate pressure change coefficients shall be adopted from the Missouri Charts summarised in Appendix B.

Gutter Flow

9. FLOW ADJUSTMENT FACTORS (Section D5.12)

The general equation recommended in AR&R, Urban Stormwater Management, Technical Note 4 shall be used to calculate gutter flows, using the following factors:

Flow adjustment factor F	= 0.8.
Manning's 'n' values	= 0.012 (Concrete)
	= 0.014 (Hot-mix)
	= 0.018 (Flush seal)

Entrance Loss Coefficients

10. CULVERT DESIGN CHARTS (Section D5.14)

Culverts shall be designed in accordance with the loss parameters contained in Chart 44 in Appendix B.

Detention Requirements

11. STORMWATER DETENTION DESIGN (Section D5.16)

Stormwater detention shall be designed so that the peak flow from the design 5, 10, 20 and 100 year ARI events, for durations between 5minutes and 3 hours, does not exceed the peak flow from the existing site. The peak flow for both the post-development and pre-development site shall be calculated using the same methodology. The computer hydrologic modelling programs ILSAX and XP-STORM are acceptable for the design of detention storages. PC SUMP and the Modified COPAS method are also acceptable for design of detention basins. Parameters to be used in ILSAX are outlined in Section 3 of this handbook. Where ILSAX is being used to model the detention storage, discharge from the outlet shall be determined using the orifice equation, with the head calculated as:

$$\text{Outlet head} = \text{Elevation} - \frac{\text{Pipe Diameter}}{2}$$

Hydraulic analysis of detention storages shall be undertaken, and the drainage plans submitted to Council shall include an elevation-storage-discharge table for the detention storage.

Although outlets can be standard diameter pipelines preference shall be given to orifice controlled outlets. A suitable galvanised steel screen to prevent access to children but not to trap trash shall cover the outlet from the basin.

Detention basins/devices should be capable of retaining first flush in accordance with the criteria outlined in the Water & Rivers Commission’s “A manual for managing urban stormwater quality in Western Australia”(1998) now superseded by the Department of Water's Stormwater Management Manual for Western Australia (2004-2007). The design should at all times treat runoff at source, such as infiltration, for the 1 in 6 months event minimum(1 year 1 hour storm desirable), before allowing the runoff into the piped system whenever and wherever possible. Gross pollutant devices should be incorporated in the drainage system prior to discharge into basins or creek.

Pipe Requirements

12. CONDUIT AND MATERIAL STANDARDS & CONDUIT JOINTING DETAILS (Section D5.18)

Conduits shall comply with the following:-

- Precast Concrete Pipes AS4058-1992
- uPVC Pipes AS1260 (sewer grade)
- uPVC Pipes AS1254

Where precast concrete pipes shall be subject to tidal influence, salt water concrete cover (Exposure Classification B2 or C) shall be specified.

The class of pipe shall be selected for the anticipated loading on the buried pipe and be in accordance with the manufacturers recommendations. The minimum cover to be provided over conduits shall generally be 450mm for areas not subject to vehicular loading ie: clear of road reserve, and 600mm for areas subject to vehicular loadings. For concrete pipes, compaction and pipe support in conduit trenches shall be to the requirements of AS3725-1989 and to AS3725 Supplement 1 -1989. For pipes under road pavement minimum class for rubber ring jointed reinforced concrete pipes required is 4.

Rubber Ringed spigot and socket pipes shall be used for all sizes of pipe.

Where multiple pipes are used, they shall be spaced sufficiently to permit adequate compaction of the fill between the pipes. The minimum spacing between the outer face of the walls of multiple pipes shall be 300mm for pipes of 600mm diameter or less and 600mm for pipes of diameter greater than 600mm. Generally uPVC pipes are not allowed within road reserve.

Scour Protection

13. GUIDELINES FOR SCOUR PROTECTION AT OUTLETS (Section D5.20)

Scour protection shall be provided at the outlet of all conduits which discharge into open channels. The design of scour protection shall include consideration of the following:

Scour protection shall generally be provided by rip-rap (minimum 75mm diameter), grouted riprap or gabions. The minimum depth of rip-rap shall be 150mm and the rip rap can be 50mm all in railway ballast.
The length of the scour protection shall extend a minimum of 3.0metres beyond the headwall.
Any subgrade fill shall be compacted to the density of the surrounding undisturbed material.
A toewall of minimum depth 500mm shall be provided at the downstream end of the scour protection. The exit velocity at end of structure for ARI of 1 in 5 year storm shall not exceed 1.5 metres per second.

Easements

14. REQUIREMENTS FOR STORMWATER DRAINAGE EASEMENTS (Section D5.17 & D5.20)

A 3 m minimum width drainage easment is required for all piped stormwater lines less than 3m deep through private property. For deeper drains easement width needs to be assessed by Council. All easement shall be of sufficient width to contain all stormwater pits and pipes and to allow sufficient width for maintenance requirements. Easements for stormwater drains under 375mm diameter shall benefit the upstream properties while lines of 375mm diameter and over shall be in favour of the City of Swan Council.

Easements shall be provided over all non-piped drainage systems such as open drains through private property. The easements shall be a minimum of 5 metres for defined channels and 10 metres for dispersed flows.

The City of Swan Council shall be the authority empowered in all instances to vary or extinguish stormwater easements.

Acknowledegments

15. ACKNOWLEDGEMENTS

The City of Swan wishes to thank all organisations who have directly or indirectly given permission to reproduce parts of their publications in the Appendices.

LIST OF FIGURES

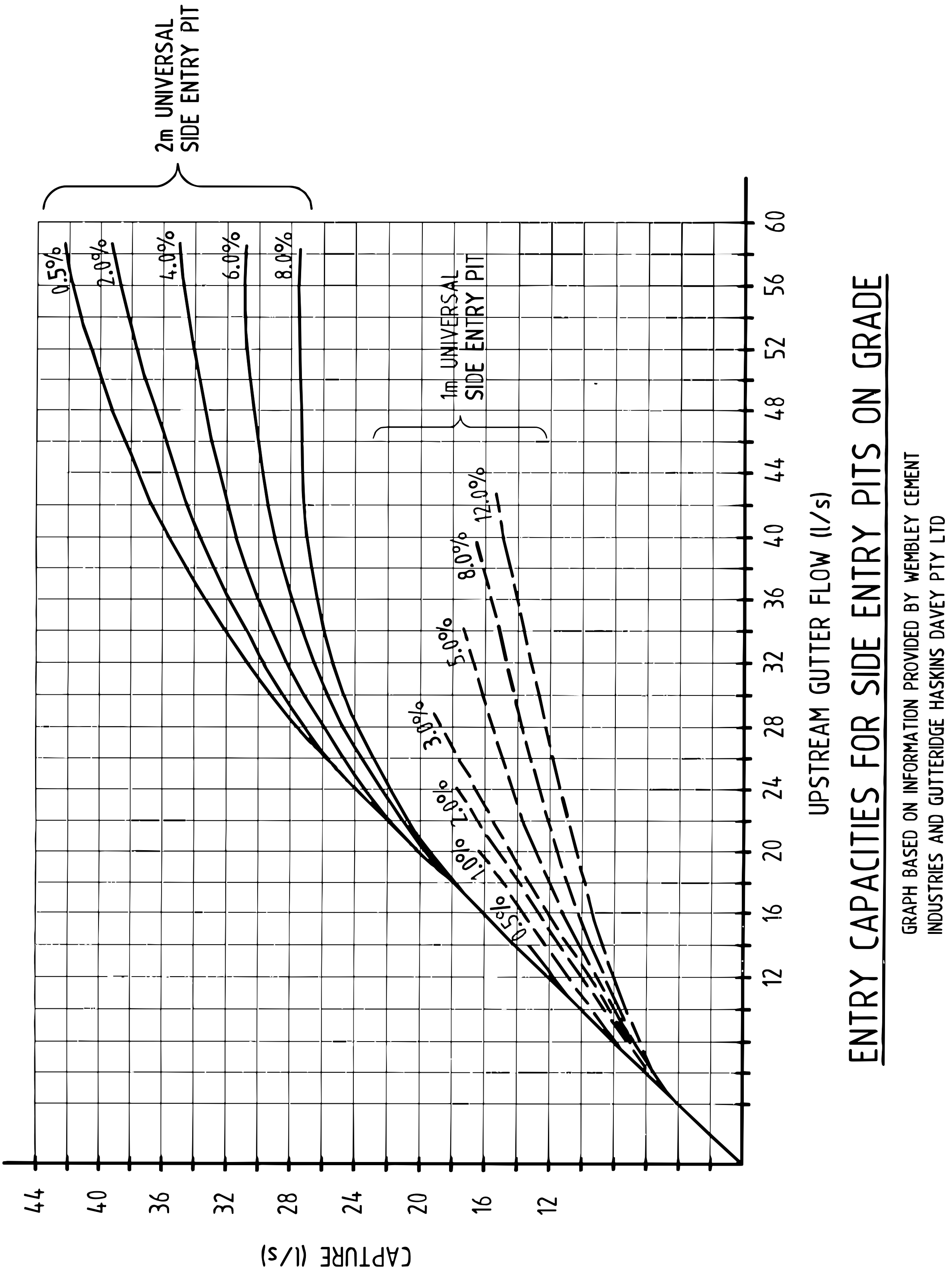


FIGURE 1 – ONGRADE KERB INLET CAPACITY CHART

APPENDIX A - SUMMARY SHEETS

a) Hydrologic Calculations

b) Hydraulic Calculations

a) Hydrologic Calculations Sheet___ of __

Analysis by: ______________________________

Date: ______________________________

Project: ______________________________

Catchment and Analysis Details:

Catchment: ______________________________
(attach map)
For: ______________________________

____

Hydrologic Method ______________________________

____

Catchment Landuse ______________________________
(minor event ARI & % impervious)

Sub Catchment	Area (ha)	Average Slope (m/m)	Channel/ Pipe Slope (m/m)	Channel/ Pipe Length (m)	Subcatchment Time of Concentration (min)	Minor Event peak flow (m^3/s)	100 year ARI peak flow (m^3/s)
Total Catchment							

b) Hydraulic Calculations Sheet___ of __

Analysis by: ____________________

Date: ____________________

Project: ____________________

Catchment and Analysis Details:

Catchment: ____________________
(attach map)
For: ____________________

____

Hydraulic Method ____________________

____

Pit No	Surface Level (AHD)	HGL at Pit (AHD)	Invert Level (AHD)	Design Flow (m^3/s)	Outlet pipe diameter (mm)	D/S Pit No

APPENDIX B - CHARTS

Pressure loss (or head loss) at junctions may be expressed as a function of the velocity head of the flow in the conduit downstream of the junction, $V_o^2/2g$:

thus $h_s = K.V_o^2/2g$ (5.21.7)

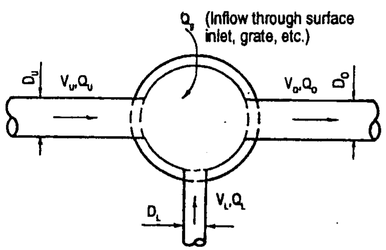
where h_s = pressure change at a structure

K = pressure change coefficient

The charts contained in Volume 2 of this Manual provide pressure change coefficients for the majority of junction types encountered in urban drainage design.

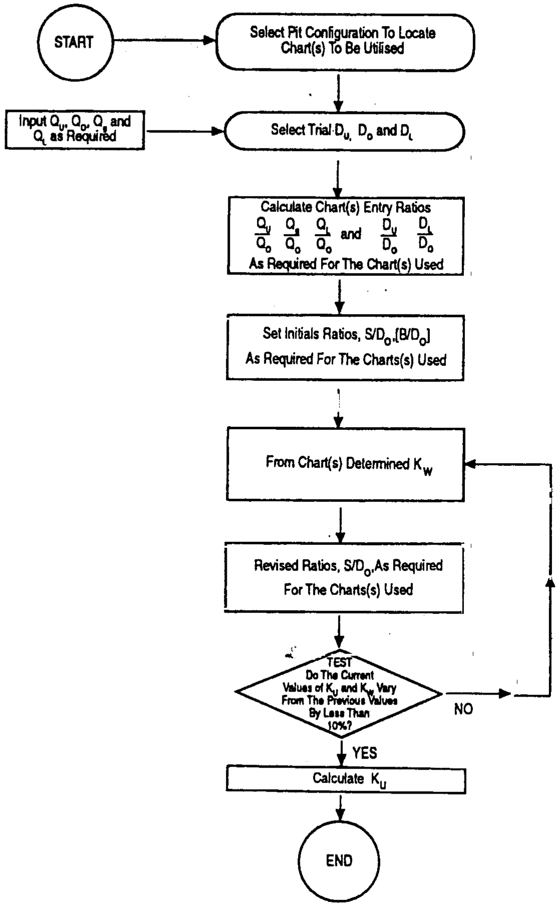
Note that where a structure has lateral as well as through flow the pressure change coefficient which applies to the through (main) line may be different to that for the lateral line i.e. K_u may not equal K_L.

The appropriate charts should be used to determine correct values of K_u and K_L. The pressure change coefficients K_u and K_L should be applied to the velocity head $V_o^2/2g$ in the outlet pipe from the structure.



Nomenclature at Structures

Figure 5.21.4



Coefficient K_u and K_W Calculation Procedure Flow Chart

Figure 5.21.5

STRUCTURE SCHEMATIC DIAGRAM	APPLICABLE CHARTS Ku	APPLICABLE CHARTS Kw
Kg=Kw	32	32
Kg=Kw	32	32
Ku=Kw	33	33
22 1/2° Ku=Kw	34	34
22 1/2° Ku=Kw	35	36
45° Ku=Kw	37	37
45° Ku=Kw	37	37
45° Ku≠Kw	38	39
45° Ku≠Kw	40	41
67 1/2° Ku≠Kw	42	43
67 1/2° Ku≠Kw	44	45
Ku≠Kw	46	47
Ku≠Kw	48	48
KL, Kt=Kw	49	49
Ku≠Kw	49	49
Ku≠Kw	50	50
H.G.L. Ku≠Kw	51	51
Ku=Kw	52	52
Ku=Kw For Qu/Qo x Do/Du>1 For DL/Do<0.6	53	53
15° & 30°	54	N/A
45° & 60°	55	N/A
	56	N/A
MITRED BEND	56	N/A
	57	N/A
	57	N/A
	58	N/A
BENCHING	58	58
	59	N/A

□ DENOTES STRUCTURE WITH GRATE (GUTTER) INFLOW

○ DENOTES STRUCTURE WITH NO GRATE (GUTTER) INFLOW

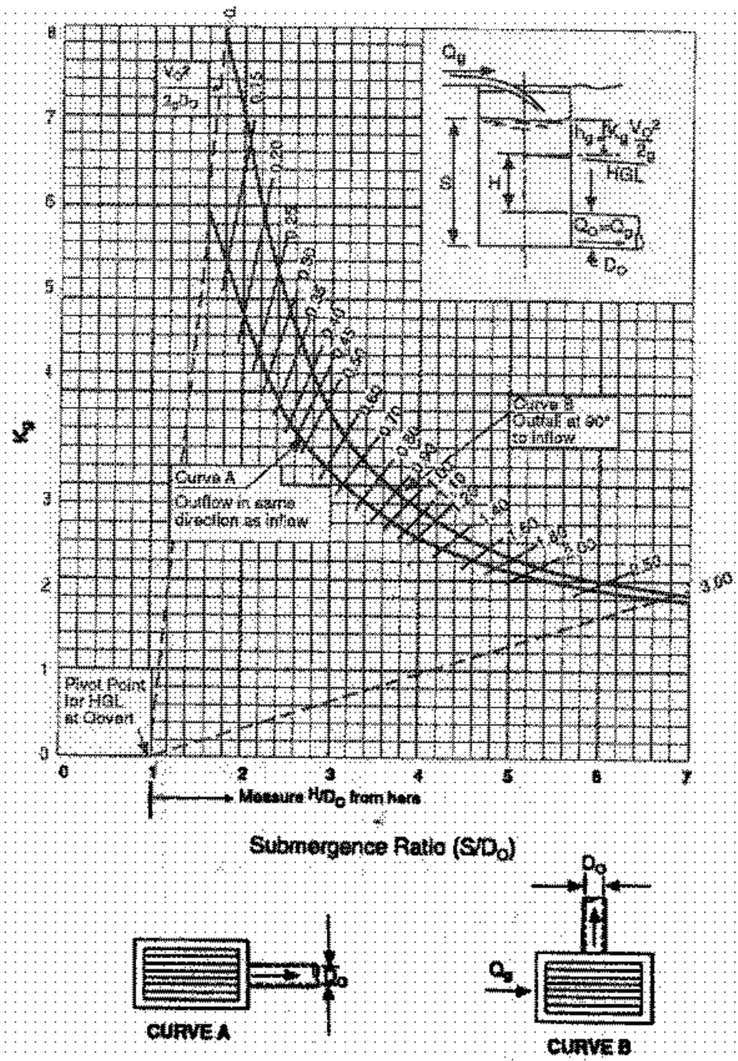
Notes:

1. Where a ***Side Inlet*** is used, the inflow direction should be taken as the direction of flow in the kerb and channel.

2. Where the outflow direction is within *15°* of the direction of inflow, use *Curve A*.

3. Where the outflow direction is greater than *15°* from the direction of inflow, use *Curve B*.

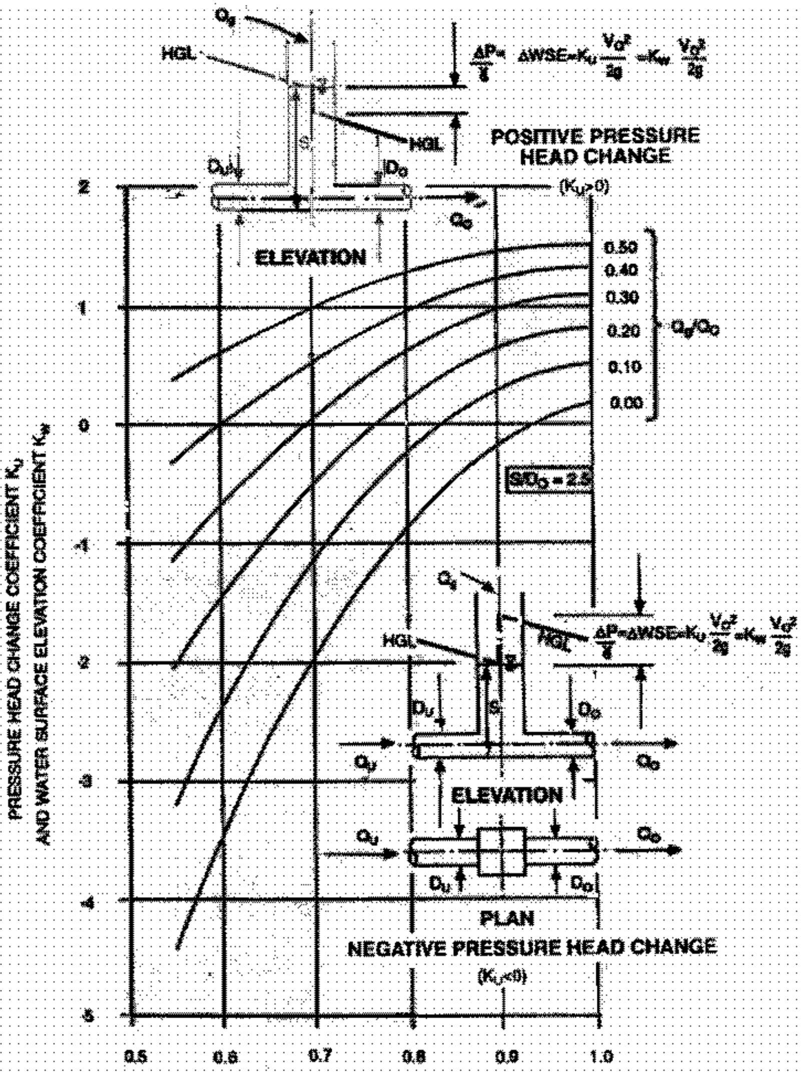
4. $K_w = K_g$.



Structure Pressure Change Coefficient for Rectangular Inlet with Grate Flow Only Modified from DOT (1992)

Q_g/Q_o / S/D_o	0.00	0.10	0.20	0.30	0.40	0.50
1.5	0.00	0.11	0.22	0.33	0.44	0.55
2.0	0.00	0.04	0.08	0.12	0.16	0.20
2.5	0.00	0.00	0.00	0.00	0.00	0.00
3.0	0.00	−0.03	−0.06	−0.09	−0.12	−0.15
3.5	0.00	−0.04	−0.08	−0.12	−0.16	−0.20
4.0	0.00	−0.05	−0.10	−0.15	−0.20	−0.25

Correction to be added to K_U and K_W for $S/D_O \neq 2.5$
(Charts 33, 34, 37)
Source: Hare (1980)

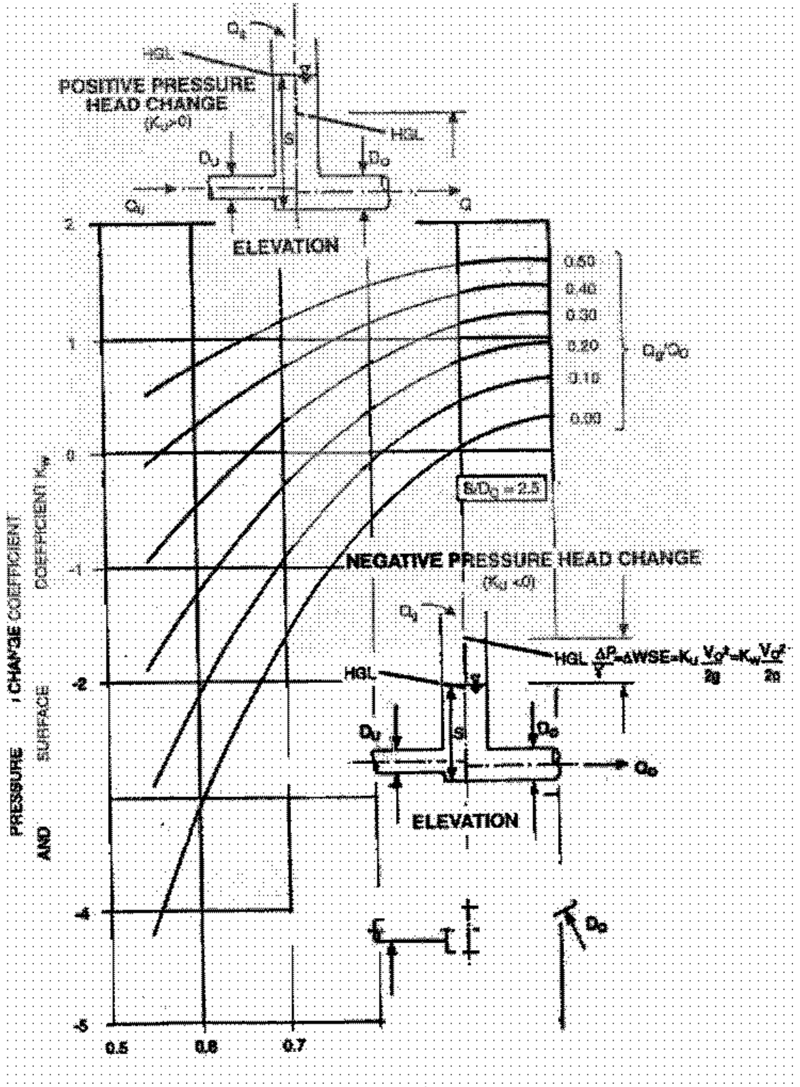


PRESSURE HEAD CHANGE AND WATER SURFACE ELEVATION COEFFICIENTS for STRAIGHT THROUGH FLOW FOR SUBMERGENCE RATIO S/D_O=2.5

Source: Hare (1980)

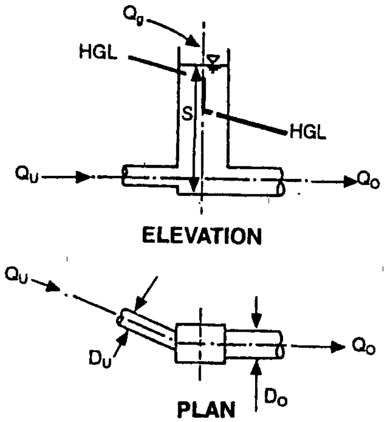
Q_g/Q_o \ S/D_o	0.00	0.10	0.20	0.30	0.40	0.50
1.5	0.00	0.11	0.22	0.33	0.44	0.55
2.0	0.00	0.04	0.08	0.12	0.16	0.20
2.5	0.00	0.00	0.00	0.00	0.00	0.00
3.0	0.00	−0.03	−0.06	−0.09	−0.12	−0.15
3.5	0.00	−0.04	−0.08	−0.12	−0.16	−0.20
4.0	0.00	−0.05	−0.10	−0.15	−0.20	−0.25

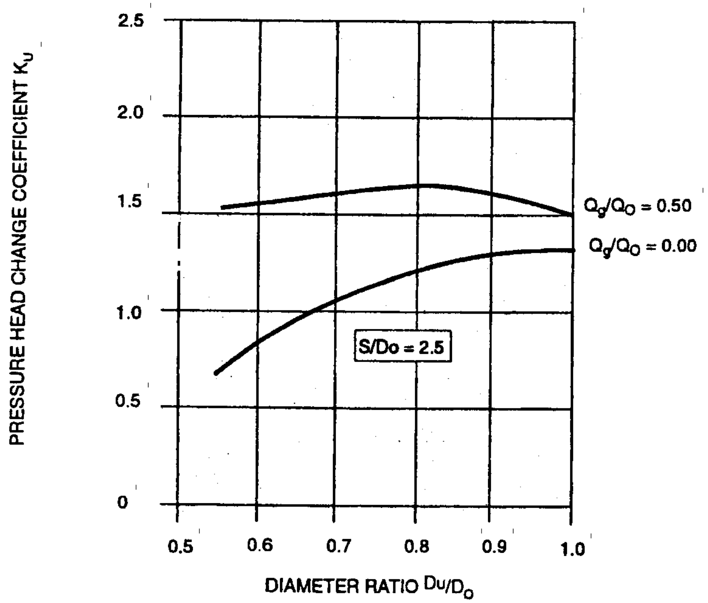
Correction to be added to K_U and K_W for $S/D_O \neq 2.5$
(Charts 33, 34, 37)
Source: Hare (1980)



PRESSURE HEAD CHANGE AND WATER SURFACE ELEVATION COEFFICIENTS for 22 ½° BENDS AT PIT JUNCTIONS - BRANCH POINT ON DOWNSTREAM FACE OF PIT. FOR SUBMERGENCE RATIO OF 2.5

Source: Hare (1980)





PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 22 $1/2$° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED ON UPSTREAM FACE OF PIT - FOR SUBMERGENCE RATIO OF 2.5.

Source: Hare (1980)

To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used:

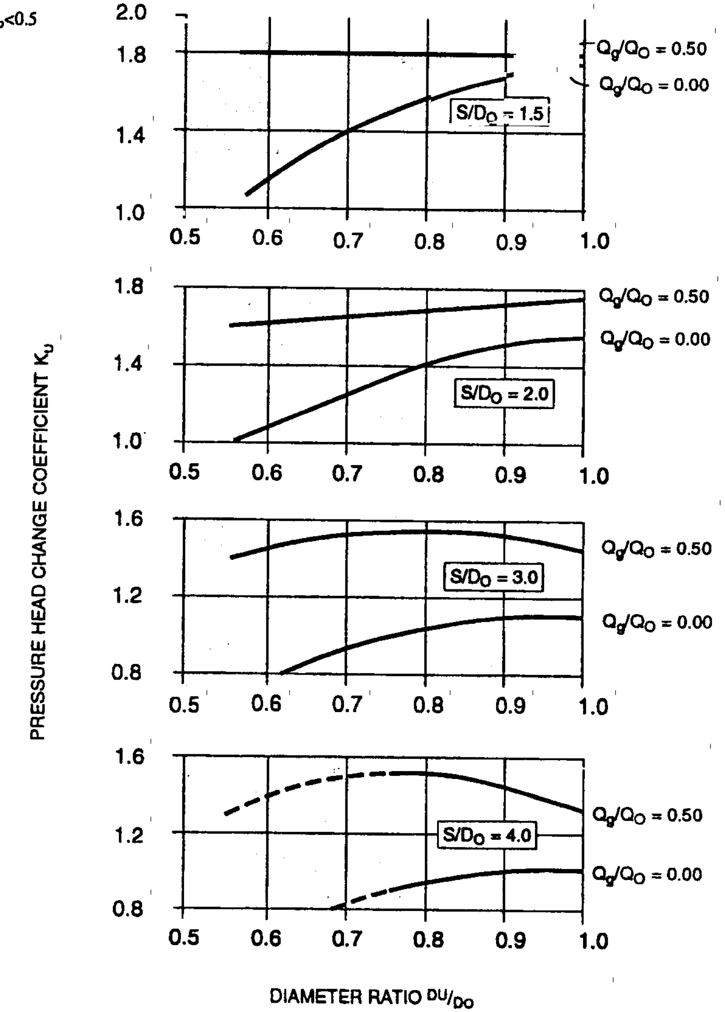
$$K_U = K_{0.0} + C_g\,(K_{0.5} - K_{0.0})$$

where $C_g = 1.33\,[1-(Q_U/Q_O)^2]$

$K_{0.0} = K_U$ for $Q_g/Q_O = 0.0$

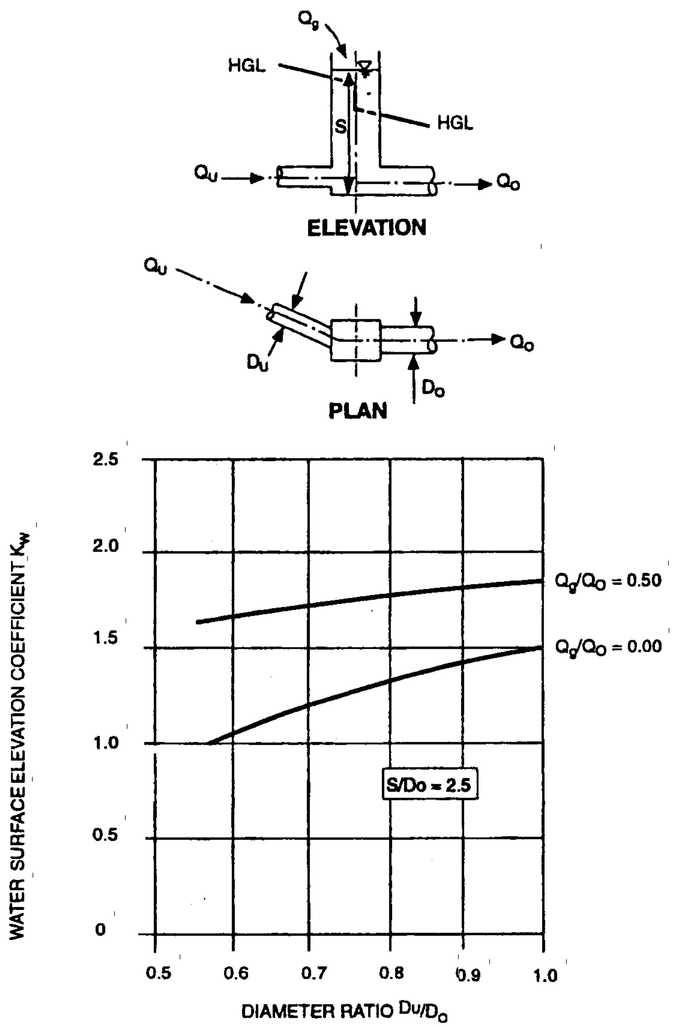
$K_{0.5} = K_U$ for $Q_g/Q_O = 0.5$

Q_g/Q_O	Q_U/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00



PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 22 $1/2$° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED ON UPSTREAM FACE OF PITS - FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

Source: Hare (1980)



WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 22 1/2° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED ON UPSTREAM FACE OF PIT - FOR SUBMERGENCE RATIO OF 2.5.

Source: Hare (1980)

To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used:

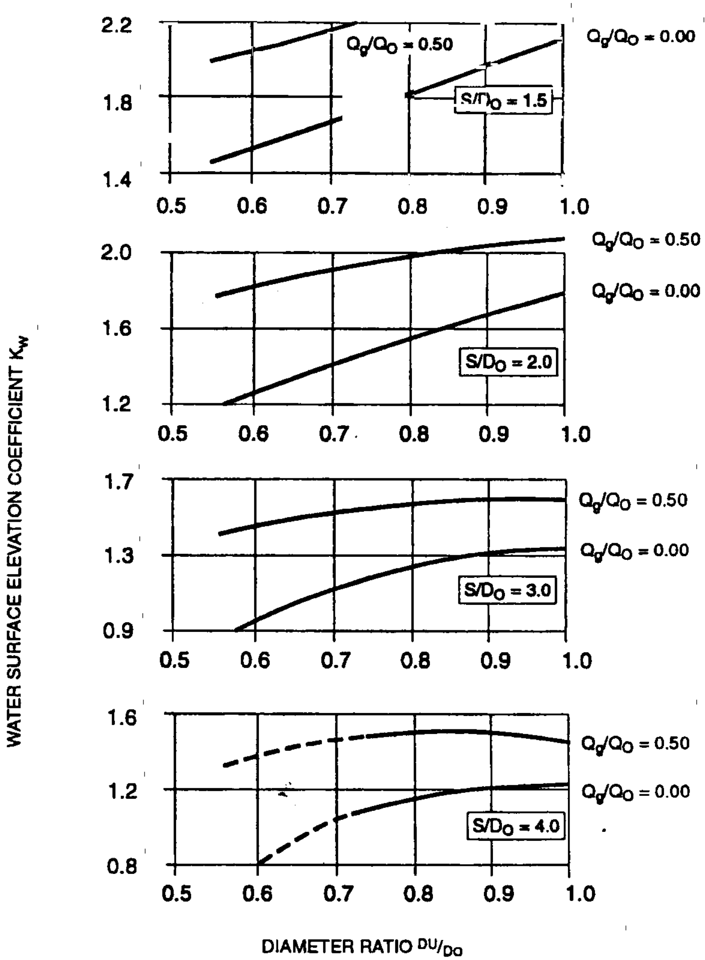
$$K_W = K_{0.0} + C_g\,(K_{0.5} - K_{0.0})$$

where $C_g = 1.33\,[1-(Q_U/Q_O)^2]$

$K_{0.0} = K_W$ for $Q_g/Q_O = 0.0$

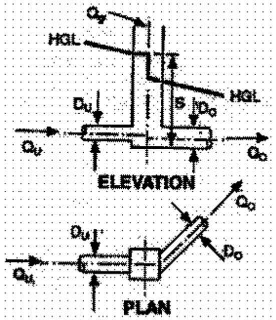
$K_{0.5} = K_W$ for $Q_g/Q_O = 0.5$

Q_g/Q_O	Q_u/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00



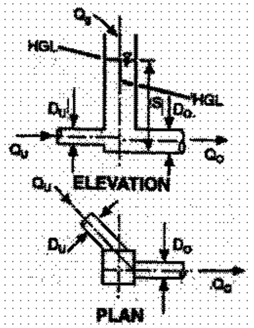
WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 22 1/2° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED ON UPSTREAM FACE OF PIT - FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

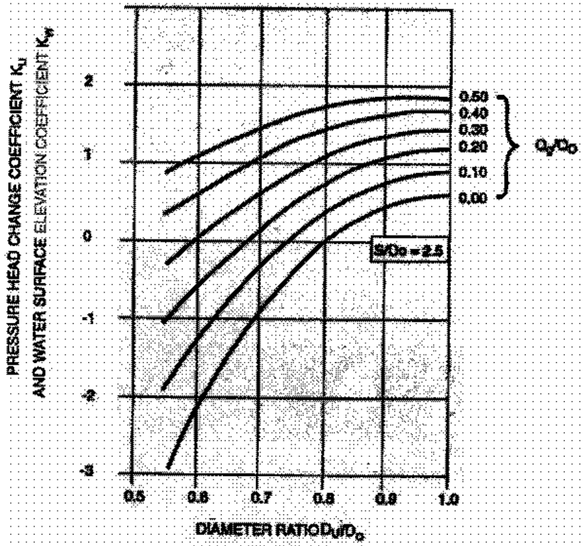
Source: Hare (1980)



S/D_o \ Q_g/Q_o	0.00	0.10	0.20	0.30	0.40	0.50
1.5	0.00	0.11	0.22	0.33	0.44	0.55
2.0	0.00	0.04	0.08	0.12	0.16	0.20
2.5	0.00	0.00	0.00	0.00	0.00	0.00
3.0	0.00	-0.03	-0.06	-0.09	-0.12	-0.15
3.5	0.00	-0.04	-0.08	-0.12	-0.16	-0.20
4.0	0.00	-0.05	-0.10	-0.15	-0.20	-0.25

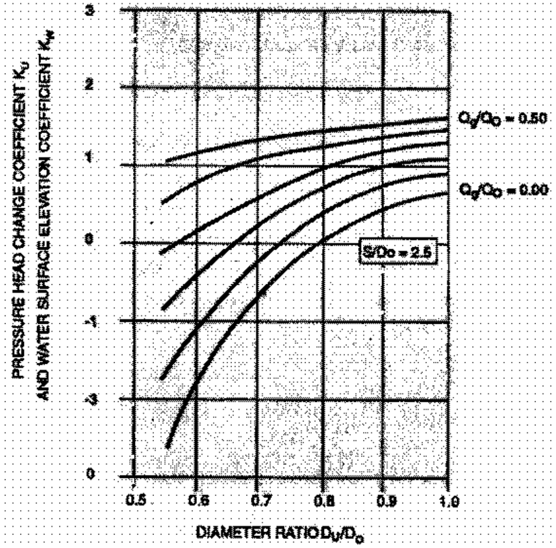
Correction to be added to K_U and K_W for $S/D_O \neq 2.5$
(Charts 33, 34, 37)
Source: Hare (1980)





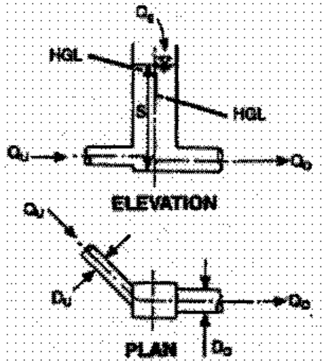
PRESSURE HEAD CHANGE AND WATER SURFACE ELEVATION COEFFICIENTS for 45 BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED ON DOWNSTREAM FACE OF PIT - FOR SUBMERGENCE RATIO OF 2.5

Source: Hare (1980)



PRESSURE HEAD CHANGE AND WATER SURFACE ELEVATION COEFFICIENTS for 45° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED ON DOWNSTREAM FACE OF PIT - FOR SUBMERGENCE RATIO OF 2.5.

Source: Hare (1980)



To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used:

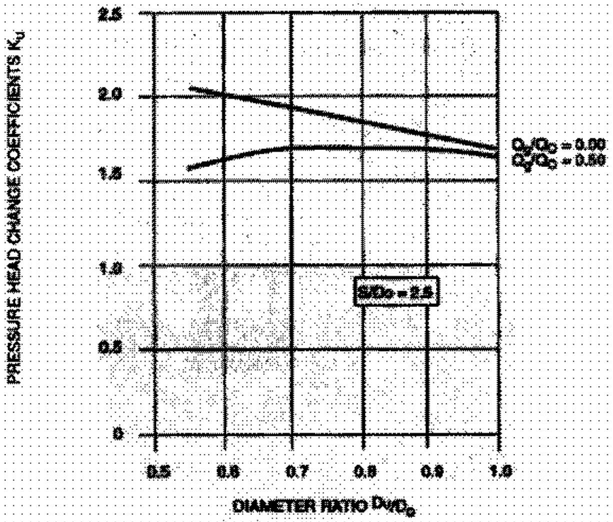
$$K_U = K_{0.0} + C_g\,(K_{0.5} - K_{0.0})$$

where $C_g = 1.33\,[1-(Q_U/Q_O)^2]$

$K_{0.0} = K_U$ for $Q_g/Q_O = 0.0$

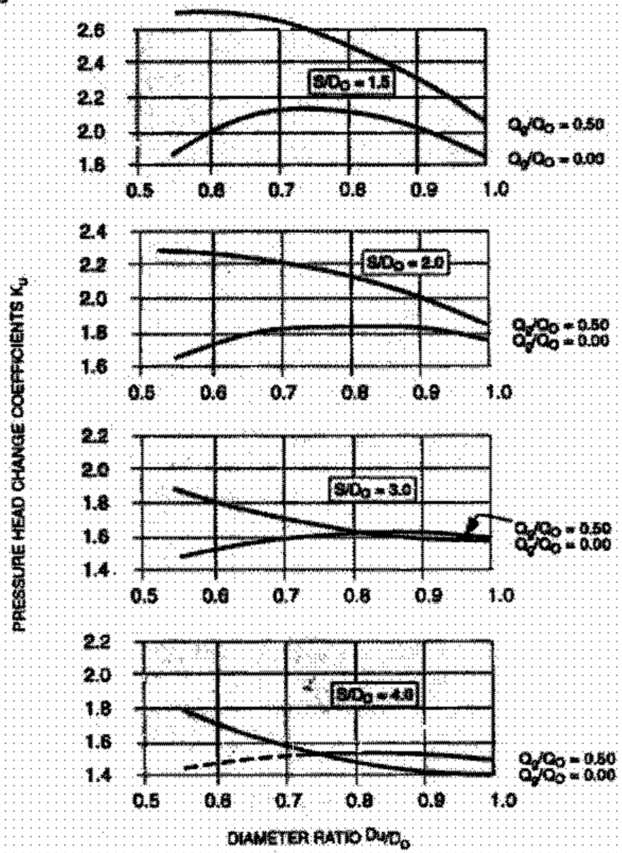
$K_{0.5} = K_U$ for $Q_g/Q_O = 0.5$

Q_g/Q_O	Q_u/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00



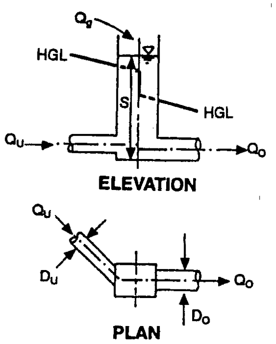
PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 45° BENDS AT PIT JUNCTIONS, BRANCH POINT LOCATED ON UPSTREAM FACE OF PIT - FOR SUBMERGENCE RATIO OF 2.5.

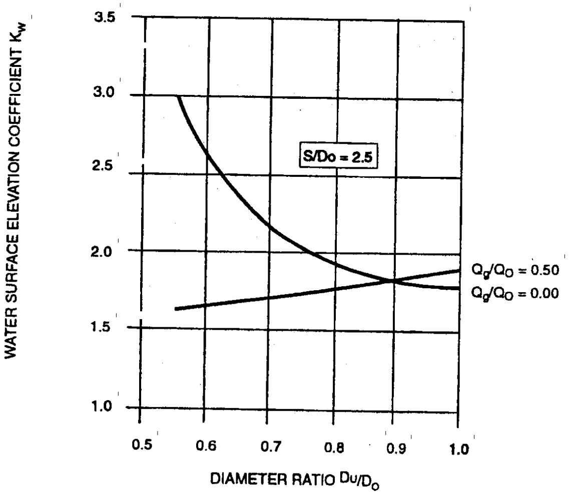
Source: Hare (1980)



PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 45° BENDS AT PIT JUNCTIONS, BRANCH POINT LOCATED ON UPSTREAM FACE OF PIT - FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

Source: Hare (1980)





WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 45° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED ON UPSTREAM FACE OF PIT - FOR SUBMERGENCE RATIO OF 2.5.

Source: Hare (1980)

To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used:

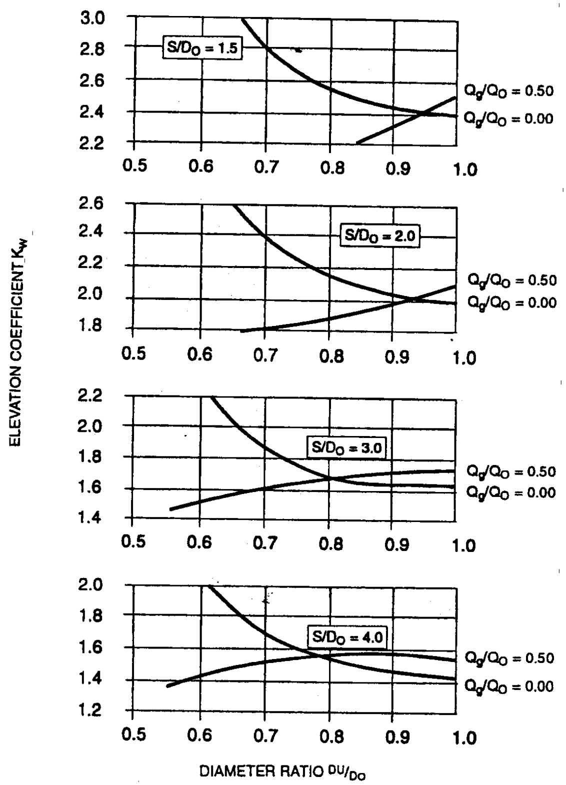
$$K_W = K_{0.0} + C_g\,(K_{0.5} - K_{0.0})$$

where $C_g = 1.33\,[1-(Q_U/Q_O)^2]$

$K_{0.0} = K_W$ for $Q_g/Q_O = 0.0$

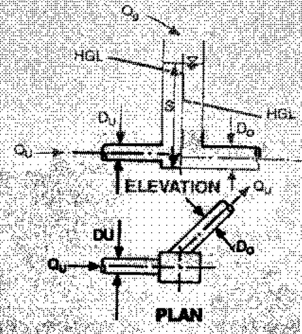
$K_{0.5} = K_W$ for $Q_g/Q_O = 0.5$

Q_g/Q_O	Q_u/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00



WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 45° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED ON UPSTREAM FACE OF PIT - FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

Source: Hare (1980)



To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used.

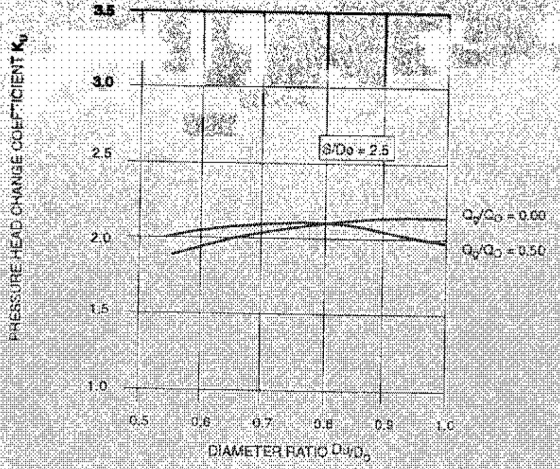
$$K_U = K_{0.0} + C_g (K_{0.5} - K_{0.0})$$

where $C_g = 1.33 [1-(Q_U/Q_O)^2]$

$K_{0.0} = K_U$ for $Q_g/Q_O = 0.0$

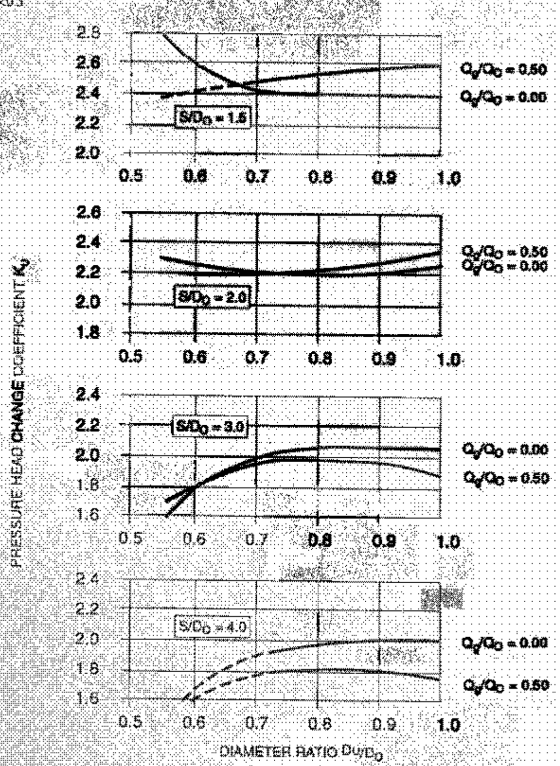
$K_{0.5} = K_U$ for $Q_g/Q_O = 0.5$

Q_g/Q_O	Q_U/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00



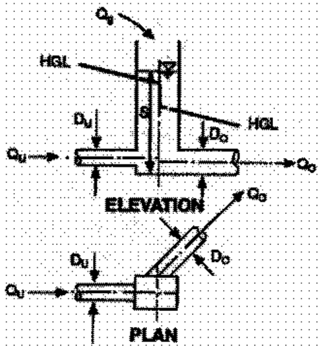
PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 45° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED ON UPSTREAM FACE OF PIT - FOR SUBMERGENCE RATIO OF 2.5

Source: Hare (1980)



PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 45° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED ON UPSTREAM FACE OF PIT FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

Source: Hare (1980)



To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used:

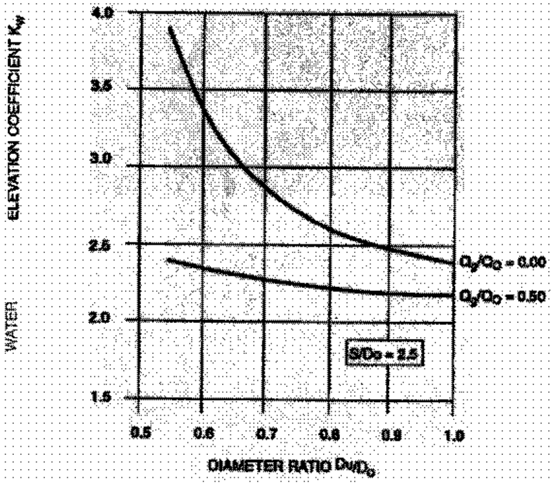
$$K_W = K_{0.0} + C_g\,(K_{0.5} - K_{0.0})$$

where $C_g = 1.33\,[1-(Q_U/Q_O)^2]$

$K_{0.0} = K_W$ for $Q_g/Q_O = 0.0$

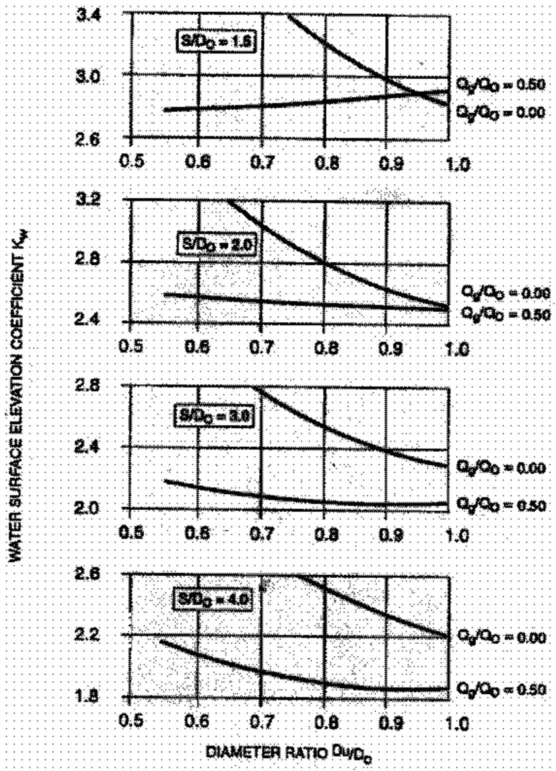
$K_{0.5} = K_W$ for $Q_g/Q_O = 0.5$

Q_g/Q_O	Q_U/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00



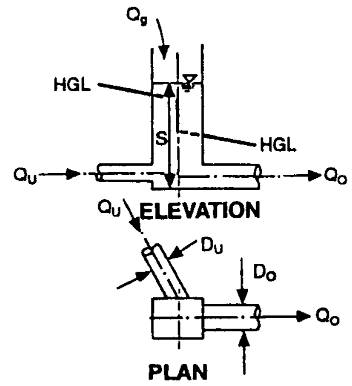
WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 45° BENDS AT PIT JUNCTIONS, BRANCH POINT LOCATED ON UPSTREAM FACE OF PIT - FOR SUBMERGENCE RATIO OF 2.5.

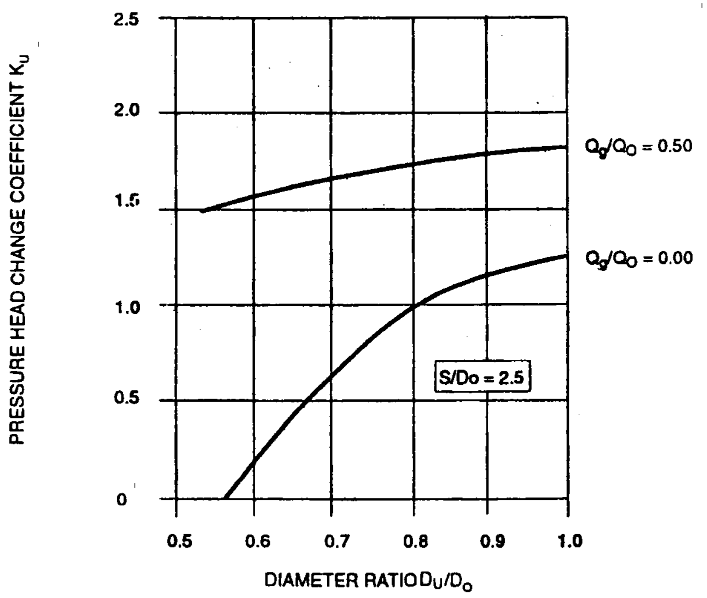
Source: Hare (1980)



WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 45° BENDS AT PIT JUNCTIONS, BRANCH POINT LOCATED ON UPSTREAM FACE OF PIT - FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

Source: Hare (1980)





PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 67 1/2° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED NEAR DOWNSTREAM FACE OF PIT - FOR SUBMERGENCE RATIO OF 2.5.

Source: Hare (1980)

To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used:

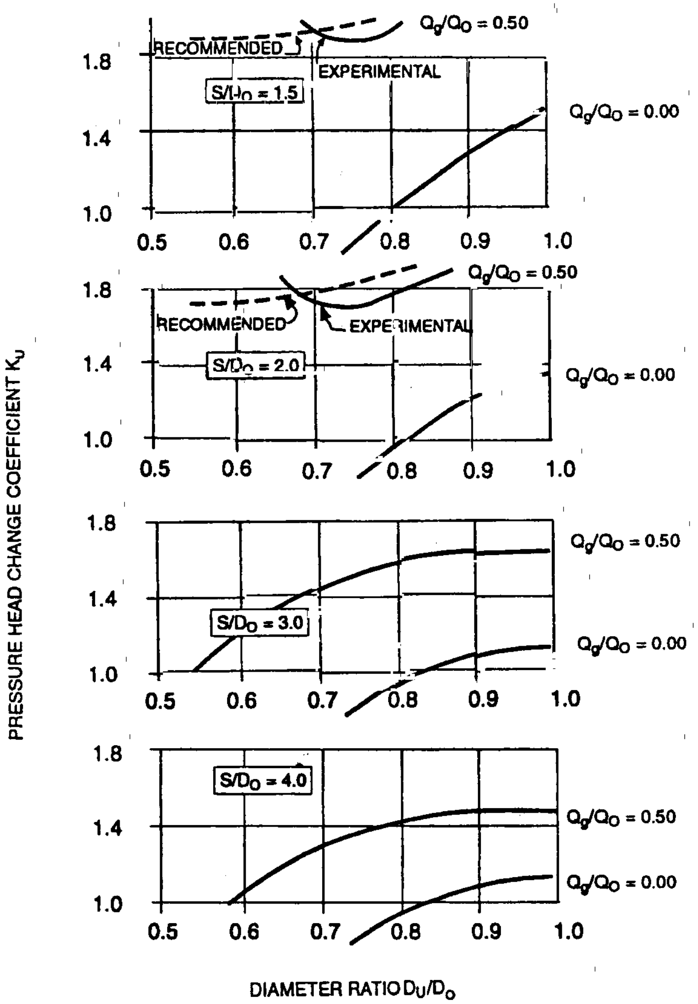
$$K_U = K_{0.0} + C_g (K_{0.5} - K_{0.0})$$

where $C_g = 1.33\,[1-(Q_U/Q_O)^2]$

$$K_{0.0} = K_U \text{ for } Q_g/Q_O = 0.0$$

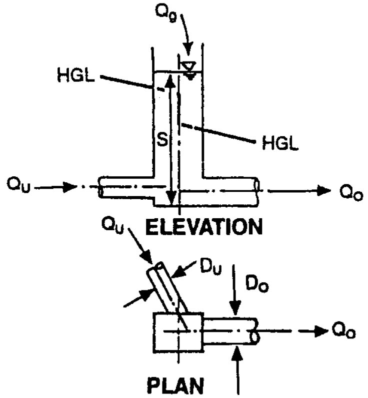
$$K_{0.5} = K_U \text{ for } Q_g/Q_O = 0.5$$

Q_g/Q_O	Q_U/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00



PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 67 1/2° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED NEAR DOWNSTREAM FACE OF PIT - FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

Source: Hare (1980)



To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used:

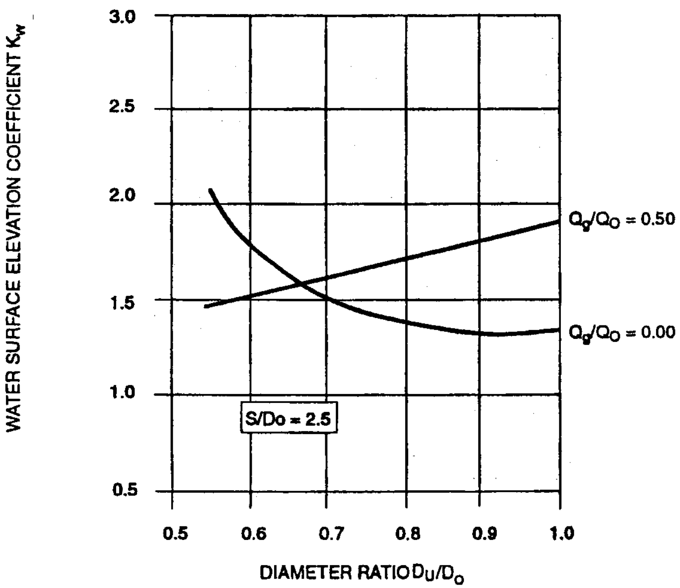
$$K_W = K_{0.0} + C_g\,(K_{0.5} - K_{0.0})$$

where $C_g = 1.33\,[1-(Q_U/Q_O)^2]$

$K_{0.0} = K_W$ for $Q_g/Q_O = 0.0$

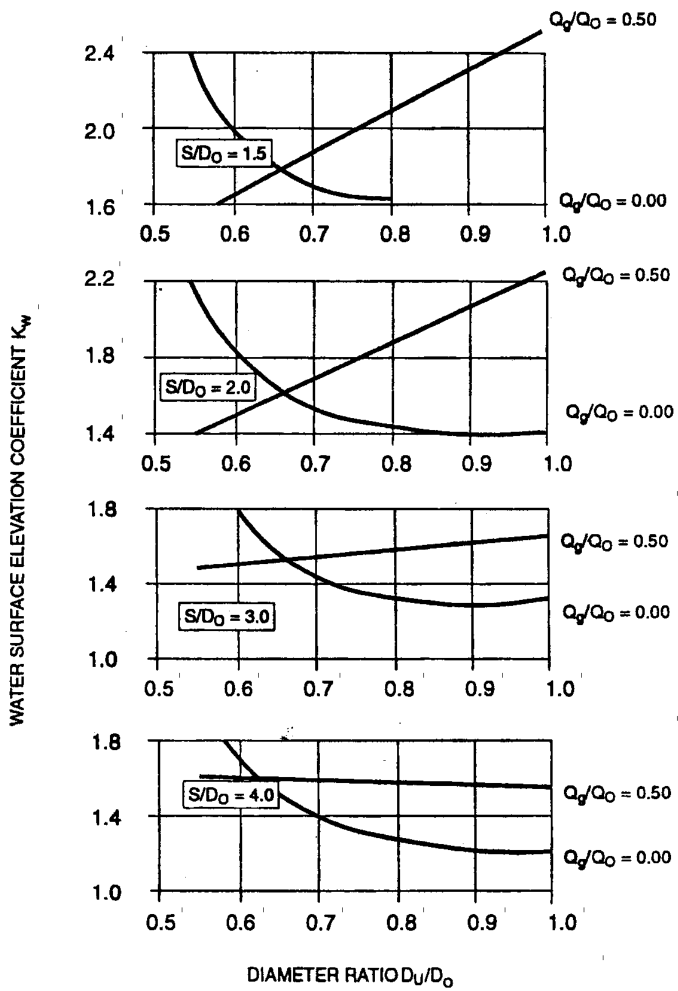
$K_{0.5} = K_W$ for $Q_g/Q_O = 0.5$

Q_g/Q_O	Q_u/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00



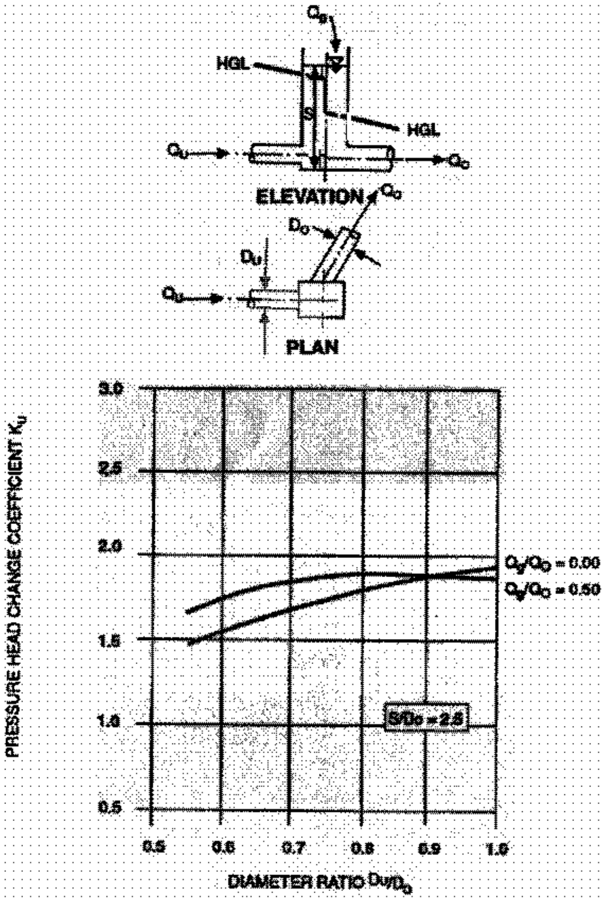
WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 67 1/2° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED NEAR DOWNSTREAM FACE OF PIT - FOR SUBMERGENCE RATIO OF 2.5.

Source: Hare (1980)



WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 67 1/2° BRANCH POINT LOCATED NEAR DOWNSTREAM FACE OF PIT - FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

Source: Hare (1980)



PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 67 1/2° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED NEAR THE UPSTREAM FACE OF THE PIT - FOR SUBMERGENCE RATIO OF 2.5.

Source: Hare (1980)

To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used:

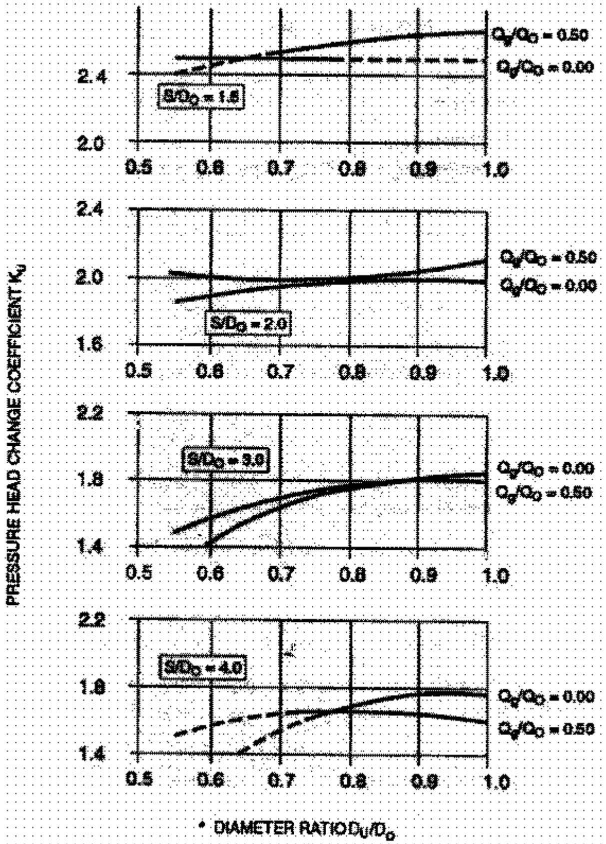
$$K_U = K_{0.0} + C_g\,(K_{0.5} - K_{0.0})$$

where $C_g = 1.33\,[1-(Q_U/Q_O)^2]$

$K_{0.0} = K_U$ for $Q_g/Q_O = 0.0$

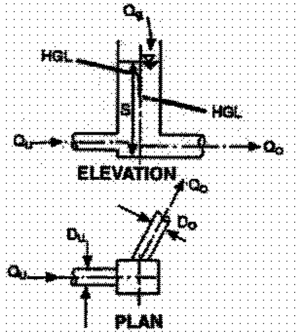
$K_{0.5} = K_U$ for $Q_g/Q_O = 0.5$

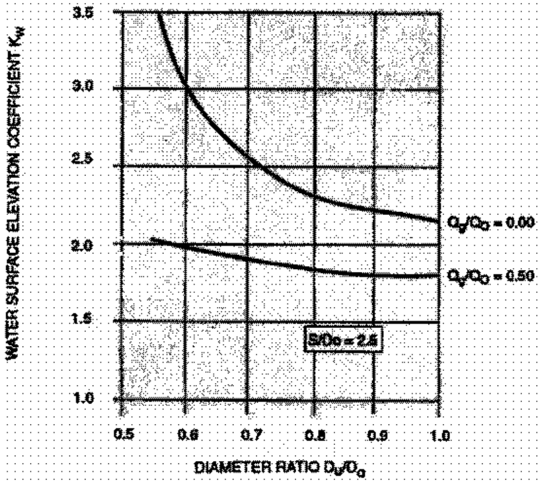
Q_g/Q_O	Q_u/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00



PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 67 1/2° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED NEAR THE UPSTREAM FACE OF THE PIT - FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

Source: Hare (1980)





WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 67 1/2° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED NEAR UPSTREAM FACE OF THE PIT - FOR SUBMERGENCE RATIO OF 2.5.

Source: Hare (1980)

To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used:

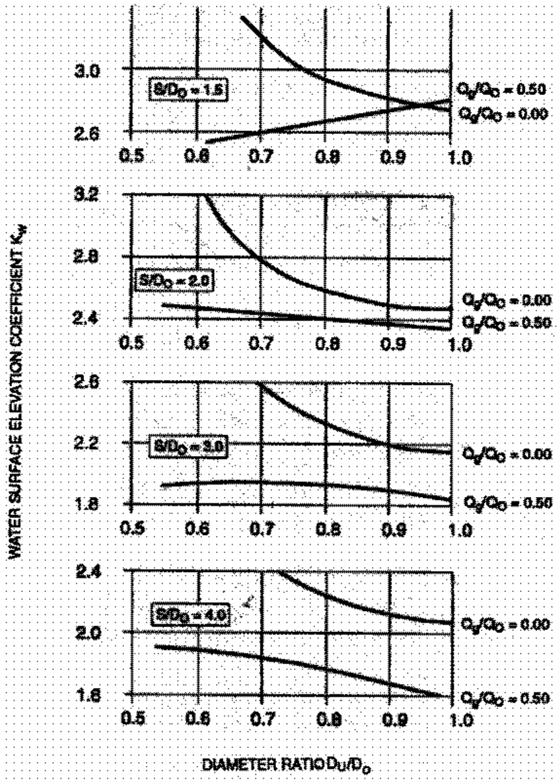
$$K_W = K_{0.0} + C_g (K_{0.5} - K_{0.0})$$

where $C_g = 1.33 [1-(Q_U/Q_O)^2]$

$K_{0.0} = K_W$ for $Q_g/Q_O = 0.0$

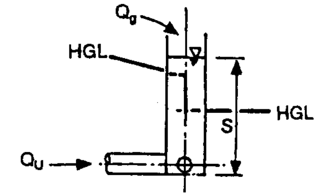
$K_{0.5} = K_W$ for $Q_g/Q_O = 0.5$

Q_g/Q_O	Q_u/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00

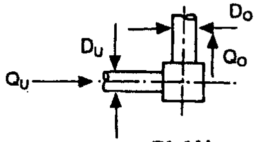


WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 67 1/2° BENDS AT PIT JUNCTIONS. BRANCH POINT LOCATED NEAR UPSTREAM FACE OF THE PIT - FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

Source: Hare (1980)



ELEVATION



PLAN

To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used:

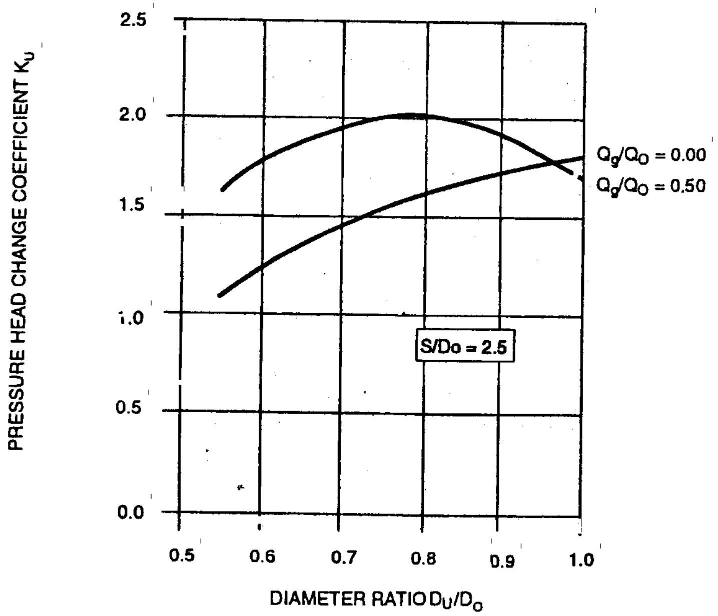
$$K_U = K_{0.0} + C_g\,(K_{0.5} - K_{0.0})$$

where $C_g = 1.33\,[1-(Q_U/Q_O)^2]$

$K_{0.0} = K_U$ for $Q_g/Q_O = 0.0$

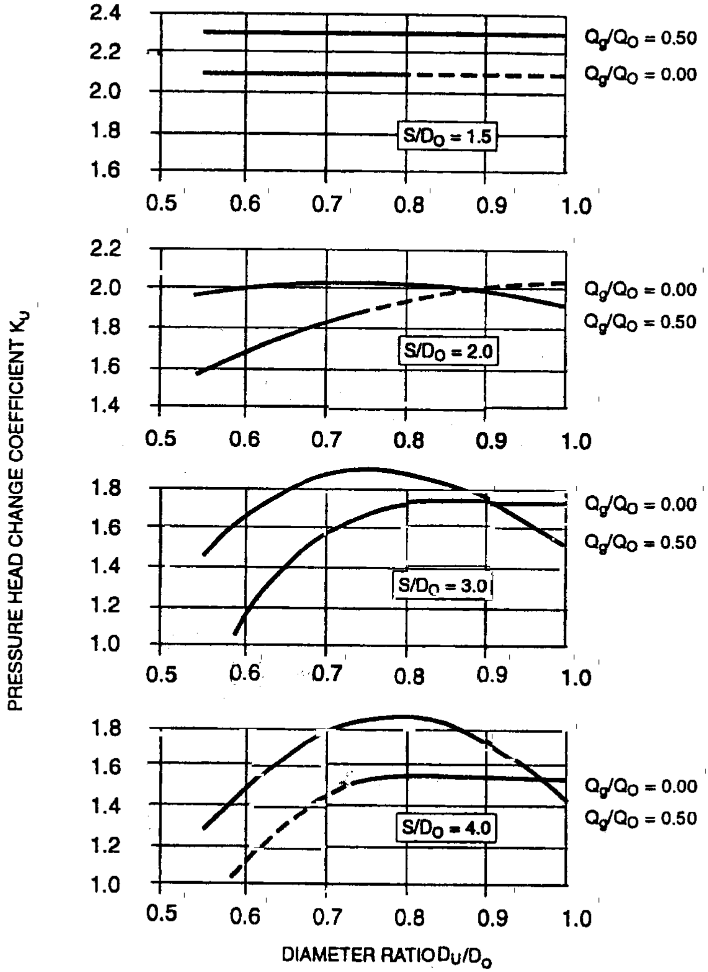
$K_{0.5} = K_U$ for $Q_g/Q_O = 0.5$

Q_g/Q_O	Q_u/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00



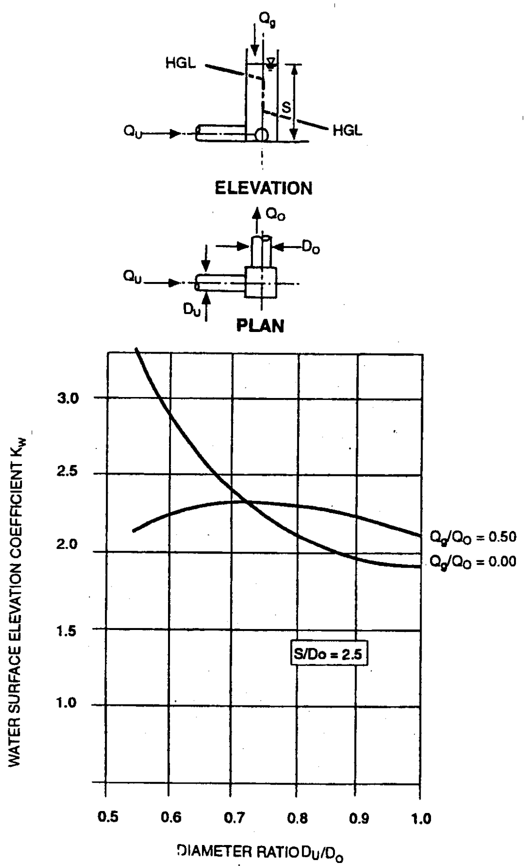
PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 90° BENDS AT PIT JUNCTIONS. FOR SUBMERGENCE RATIO OF 2.5.

Source: Hare (1980)



PRESSURE HEAD CHANGE COEFFICIENTS (K_U) for 90° BENDS AT PIT JUNCTIONS. FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

Source: Hare (1980)



WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 90° BENDS AT PIT JUNCTIONS. FOR SUBMERGENCE RATIO OF 2.5.

Source: Hare (1980)

To interpolate the Hare Charts for $0.0<Q_g/Q_o<0.5$ the following equation is used:

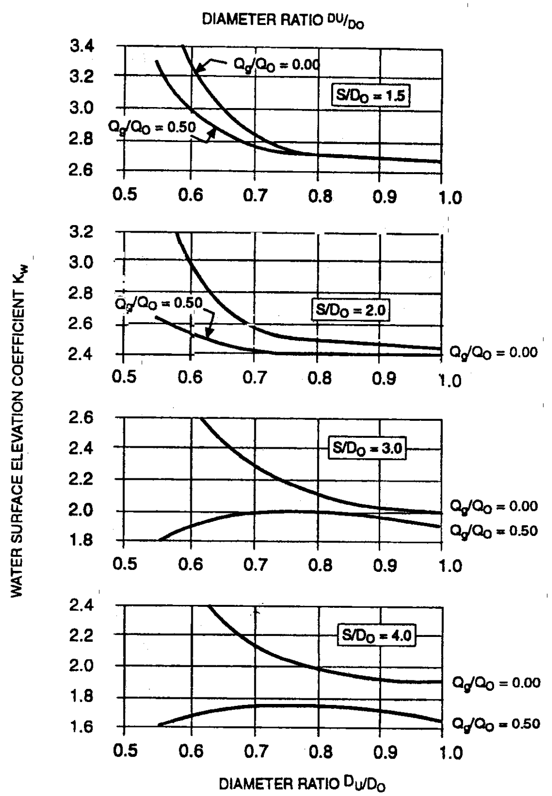
$$K_W = K_{0.0} + C_g\,(K_{0.5} - K_{0.0})$$

where $C_g = 1.33\,[1-(Q_U/Q_O)^2]$

$K_{0.0} = K_W$ for $Q_g/Q_O = 0.0$

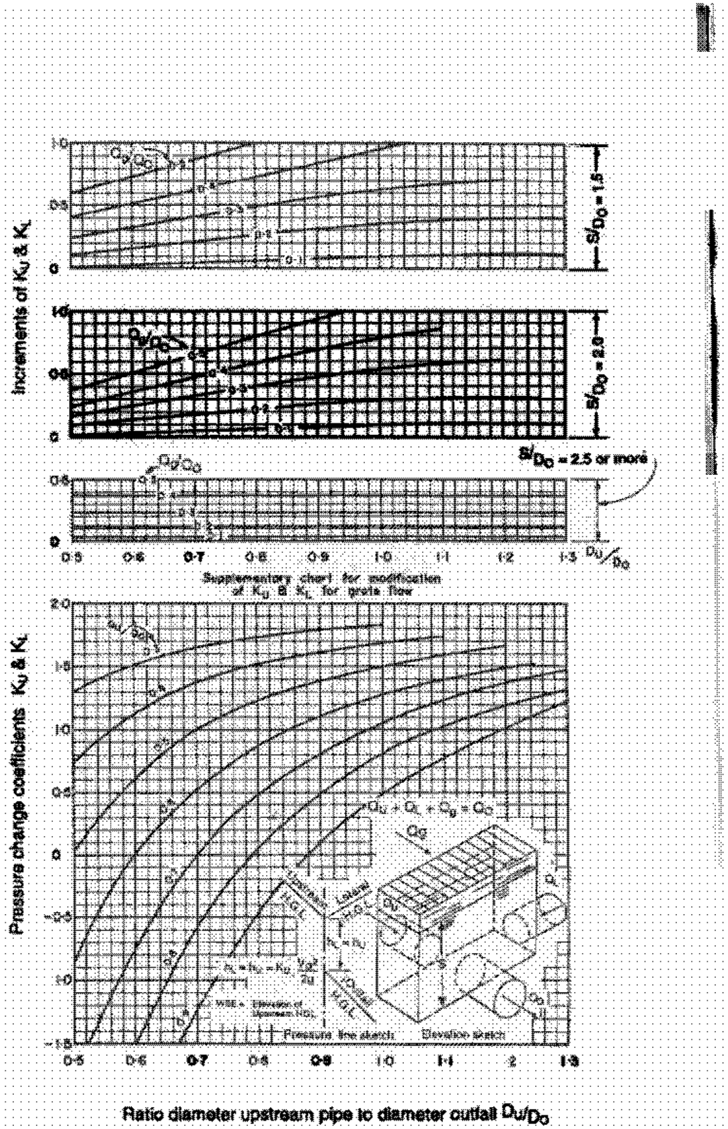
$K_{0.5} = K_W$ for $Q_g/Q_O = 0.5$

Q_g/Q_O	Q_u/Q_O	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0	1.00	0.00



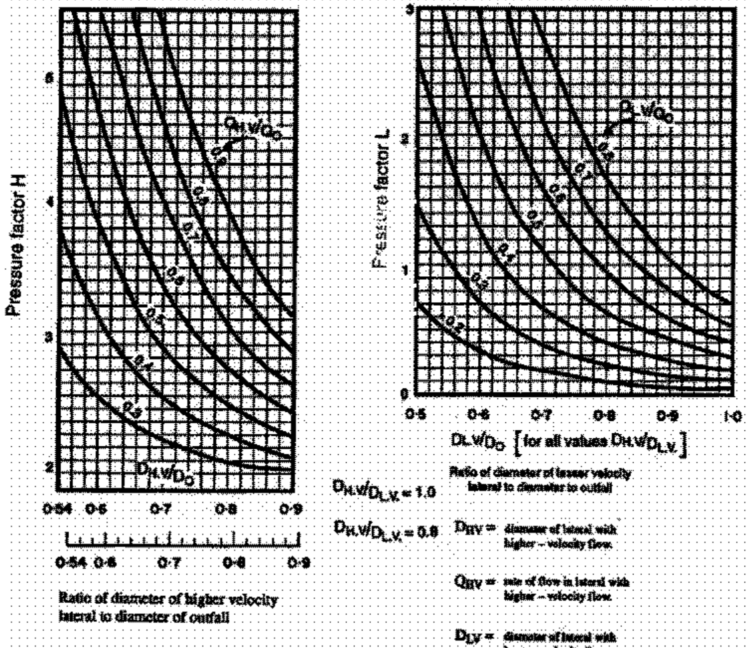
WATER SURFACE ELEVATION COEFFICIENTS (K_W) for 90° BENDS AT PIT JUNCTIONS. FOR SUBMERGENCE RATIOS OF 1.5, 2.0, 3.0 AND 4.0.

Source: Hare (1980)



Rectangular inlet with in-line upstream main and 90° lateral pipe (with or without grate flow)

Source: DOT (1992)



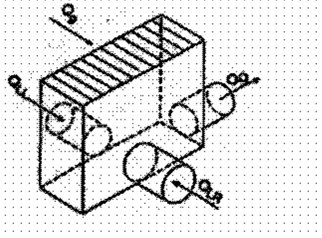
D_{HV} = diameter of lateral with higher – velocity flow.

Q_{HV} = rate of flow in lateral with higher – velocity flow.

D_{LV} = diameter of lateral with lower – velocity flow.

Q_{LV} = rate of flow in lateral with lower – velocity flow.

$Q_g + Q_{LR} + Q_{LL} = Q_O$



Elevation sketch

Note: When $Q_g = 0$, reduce K_{LL} and K_{LR} by 0.2

To find K_{LR} or K_{LL} for the right or left lateral pipe with flow at a lesser velocity than the other lateral, read H for higher velocity lateral D and Q, then read L for the lower velocity lateral D and Q then:

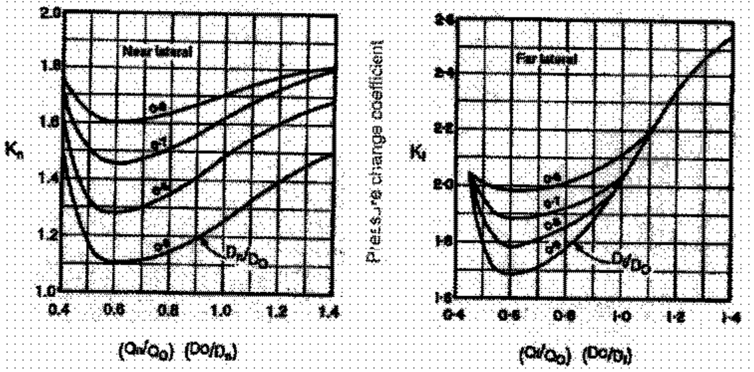
K_{LR} (or K_{LL}) = H-L

K_{LR} or K_{LL} for the lateral pipe with higher velocity flow is always 1.8.

$$h_{LL} = K_{LL} \frac{V_O^2}{2g} \quad h_{LR} = K_{LR} \frac{V_O^2}{2g}$$

WSE = Elevation of HGL for high velocity lateral

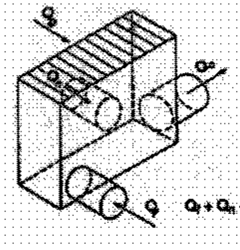
Rectangular inlet with offset opposed laterial pipes each at 90° to outfall (with or without grate flow)
Source: DOT (1992)



$$h_n = K_n \frac{V_O^2}{2g}$$

WSE = Elevation of HGL for far lateral

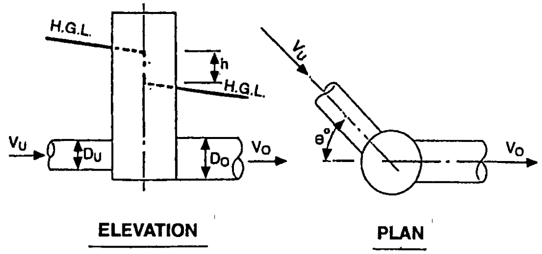
$$h_f = K_f \frac{V_O^2}{2g}$$



$Q_f + Q_n + Q_g = Q_O$

Elevation sketch

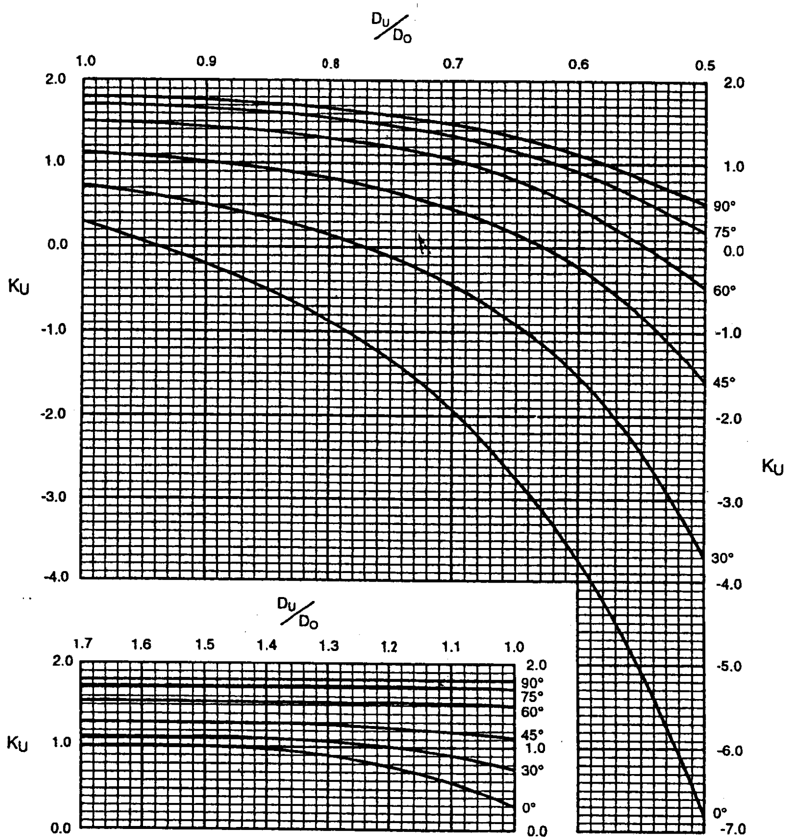
Rectangular inlet with offset opposed laterial pipes each at 90° to outfall (with or without grate flow)
Source: DOT (1992)



WSE = $K_U (V_o^2/2g) + K_w' (V_u^2/2g)$
above Elevation of Outfall HGL

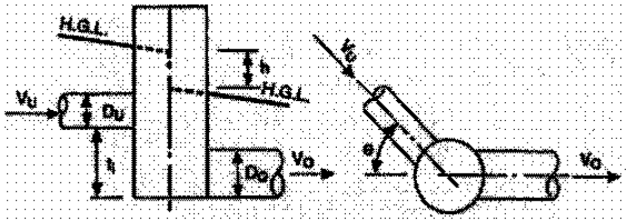
θ°	K_w'
0	0.05
1	0.11
2	0.13
3	0.14
4	0.15
5	0.16
10	0.18
15	0.20
30	0.23
45	0.25
60	0.27
75	0.29
90	0.30

Source: Hare (1980)



Circular Manhole Loss Coefficient
- Upstream Pipe Entering at Angles from 0° to 90°

Source: Cade and Thompson (1982)



ELEVATION PLAN

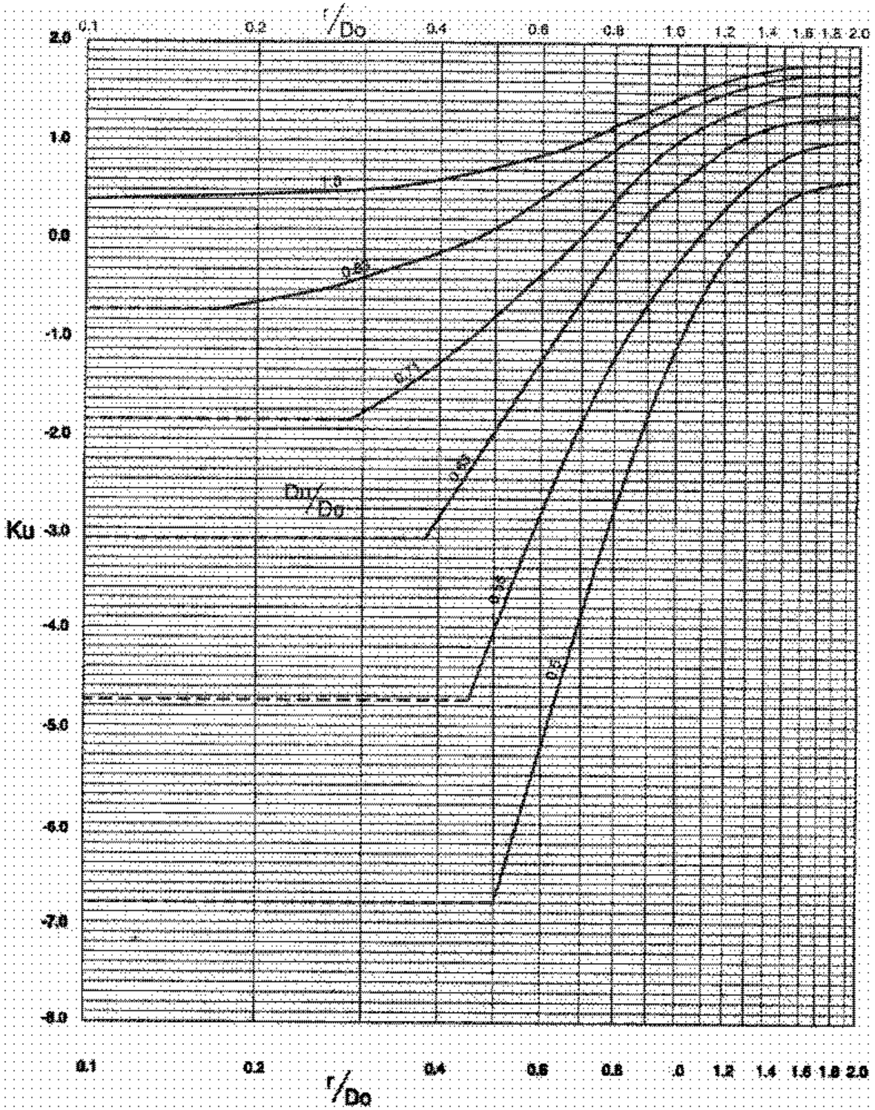
$$r/D_O = \sqrt{(t_i/D_O)^2 + (\theta/57.3)^2}$$

$h = K_U \dfrac{V_O^2}{2g}$
θ = Degrees
$t_i \neq D_O - D_U$

$WSE = K_U (V_O^2/2g) + K_W'(V_U^2/2g)$
above Elevation of Outfall HGL

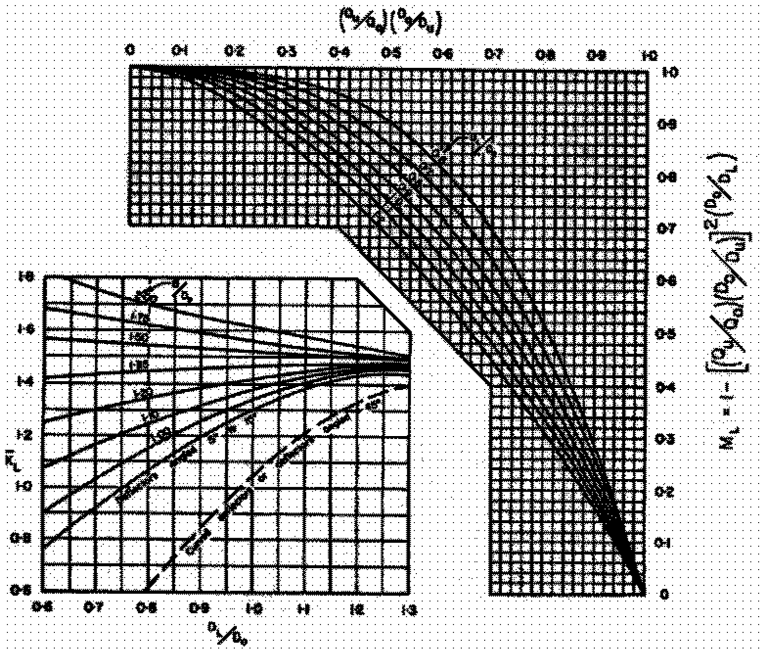
$\theta°$	K_W'
0	0.05
1	0.11
2	0.13
3	0.14
4	0.15
5	0.16
10	0.18
15	0.20
30	0.23
45	0.25
60	0.27
75	0.29
90	0.30

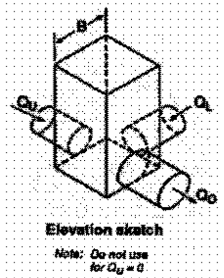
Source: Hare (1980)



Drop Manhole Loss Coefficient – Upstream Pipe Entering above Outfall Pipe and at Angles from 0° to 90°

Source: Cade and Thompson (1982)





To find K_L for the lateral pipe, first read $\bar{K}_L$ from the lower graph. Next determine M_L

Then:–

$$K_L = \bar{K}_L \times M_L$$

Dashed curve for curved or 45° angle deflectors applies only to manholes without upstream in-line pipes.

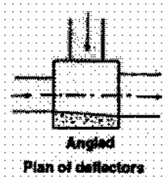
Use this chart for round manholes also.

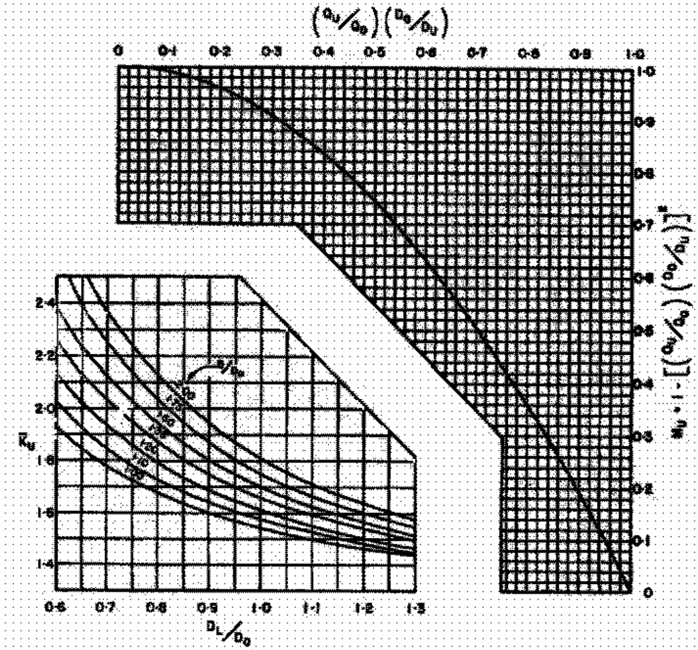
For rounded entrance to outfall pipe, reduce chart values of $\bar{K}_L$ by 0.2 for combining flow.

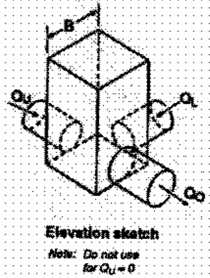
For $Q_U/Q_O \times D_O/D_U > 1$ use Chart 53

For $D_L/D_O < 0.6$ use Chart 53

$h_L = K_L \dfrac{V_O^2}{2g}$ WSE = Elevation of Upstream HGL







To find K_U for the upstream main, first read $\bar{K}_U$ from the lower graph. Next determine M_U

Then:–

$$K_U = \bar{K}_U \times M_U$$

For manholes with deflectors at 0° to 15°, read K_U on curve for $B/D_O = 1.0$

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\bar{K}_U$ by 0.2 for combining flow.

For deflectors refer to sketches on

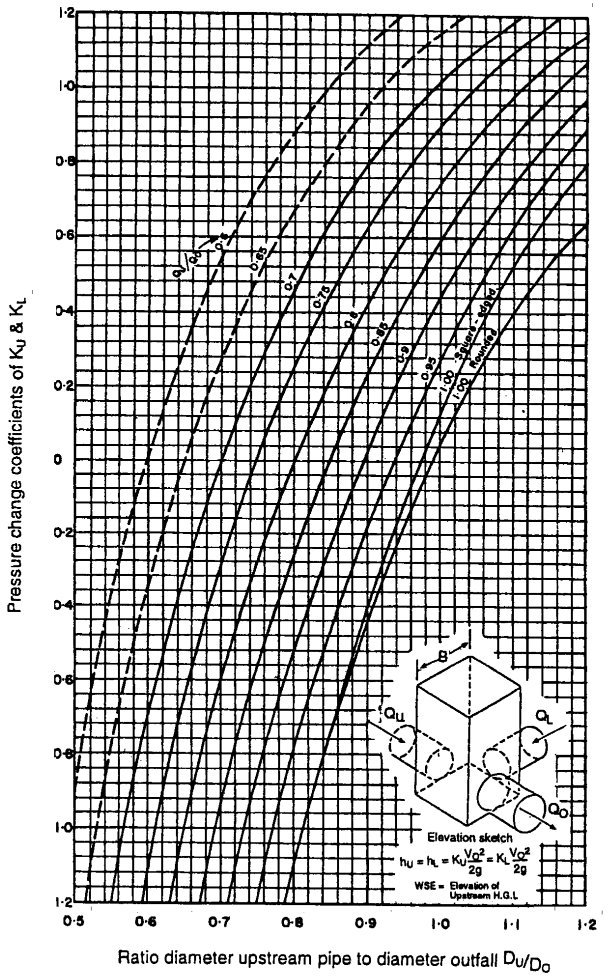
For $Q_U/Q_O \times D_O/D_U > 1$ use Chart 53

For $D_L/D_O < 0.6$ use Chart 53

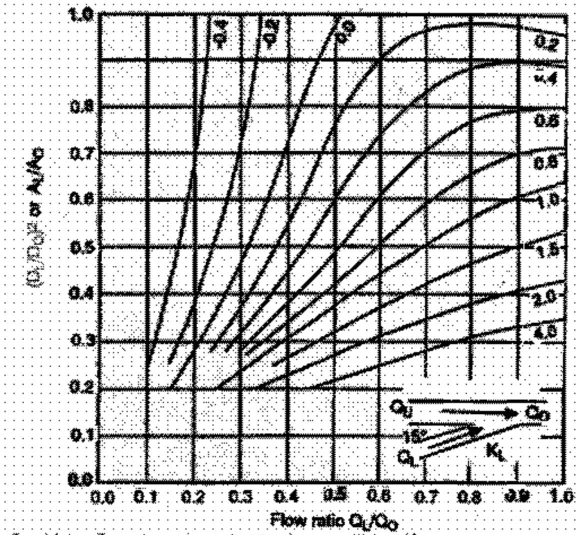
$h_U = K_U \dfrac{V_O^2}{2g}$ WSE = Elevation of Upstream HGL

Notes:

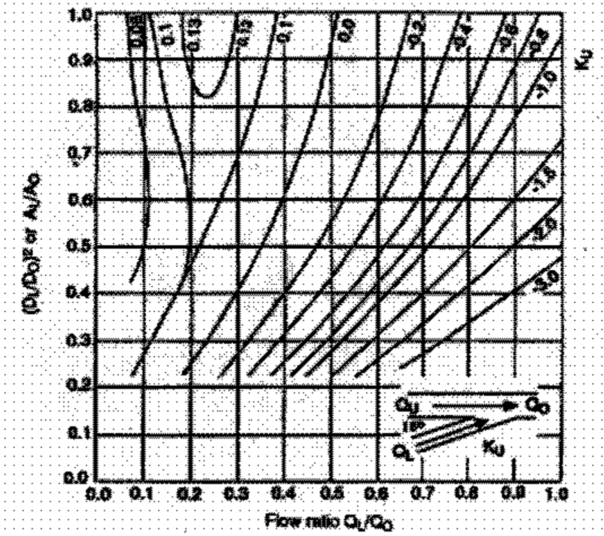
1. This chart is used instead of Chart 52 where *(Q_U/Q_O). $(D_O/D_U) > 1$* or *$(D_L/D_O) < 0.6$*
2. This chart should not be used for $Q_U/Q_O < 0.7$ if other solutions are possible.



Manhole on through pipeline at junction of a 90° lateral pipe (for conditions outside range of Chart 52)
Source: DOT (1992)

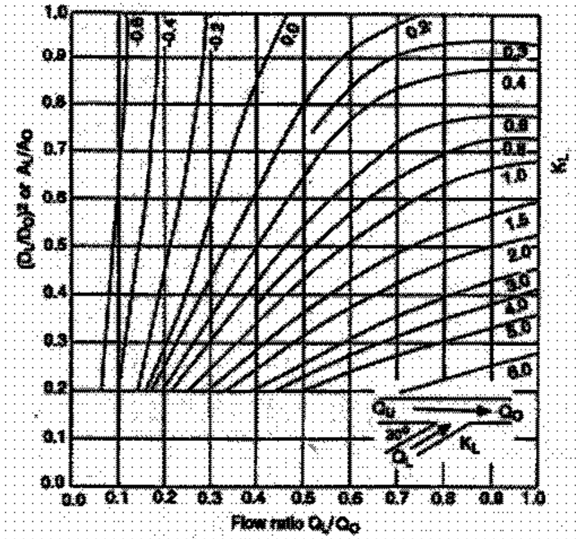


Combining flow - branch angle 15° - loss coefficient K_L

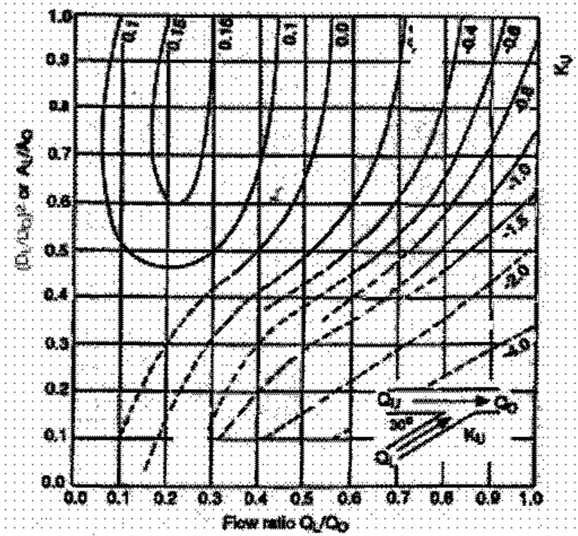


Combining flow - branch angle 15° - loss coefficient K_U

Source: Miller (1978)

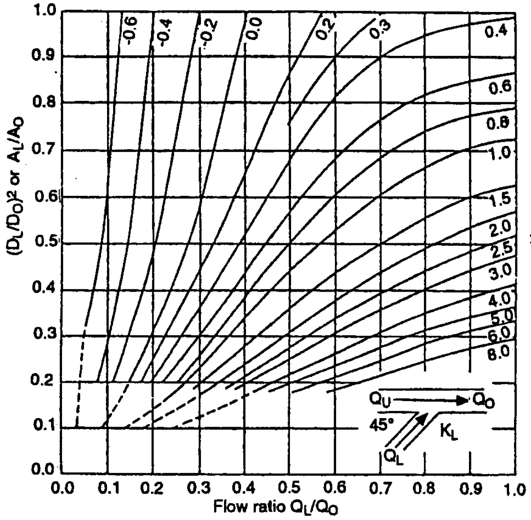


Combining flow - branch angle 30° - loss coefficient K_L

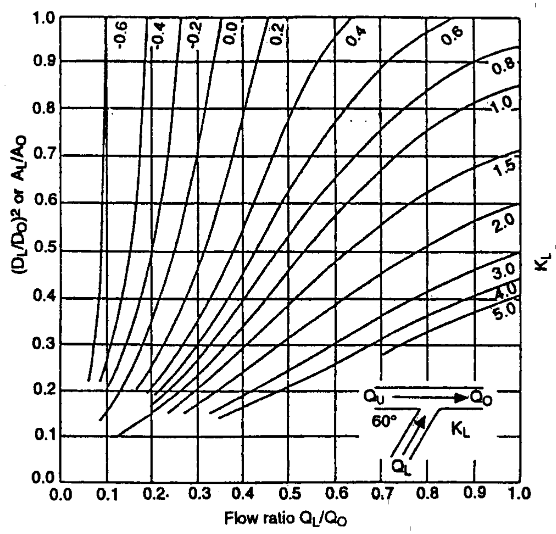


Combining flow - branch angle 30° - loss coefficient K_U

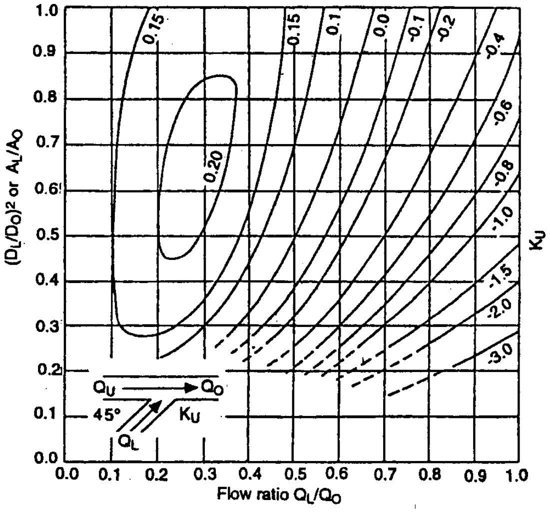
Source: Miller (1978)



Combining flow - branch angle 45° - loss coefficient K_L

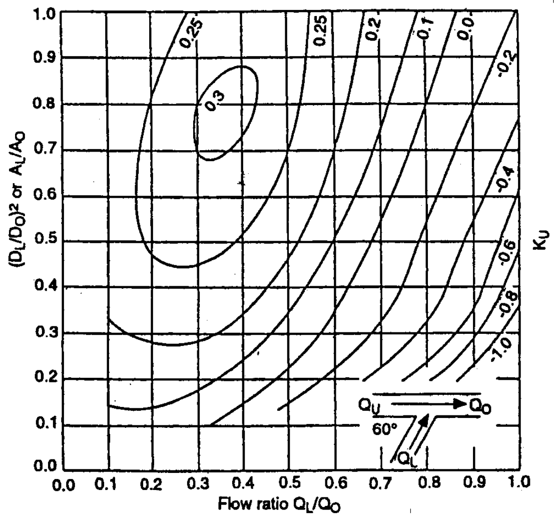


Combining flow - branch angle 60° - loss coefficient K_L



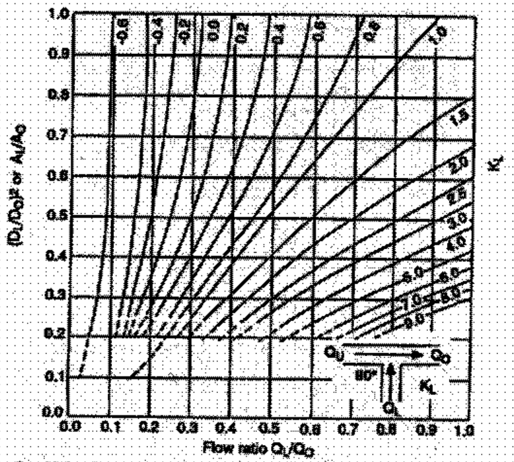
Combining flow - branch angle 45° - loss coefficient K_U

Source: Miller (1978)

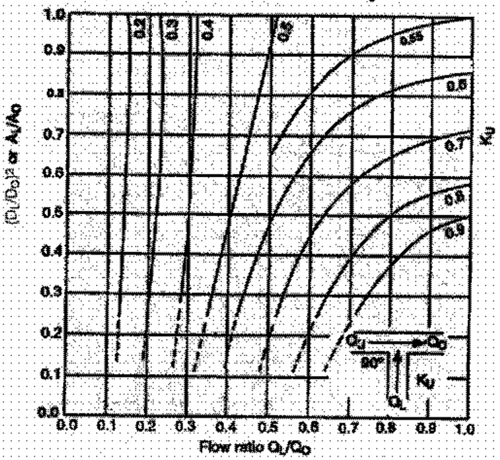


Combining flow - branch angle 60° - loss coefficient K_U

Miller (1978)



Combining flow - branch angle 90° - loss coefficient K_L



Combining flow - branch angle 90° - loss coefficient K_U

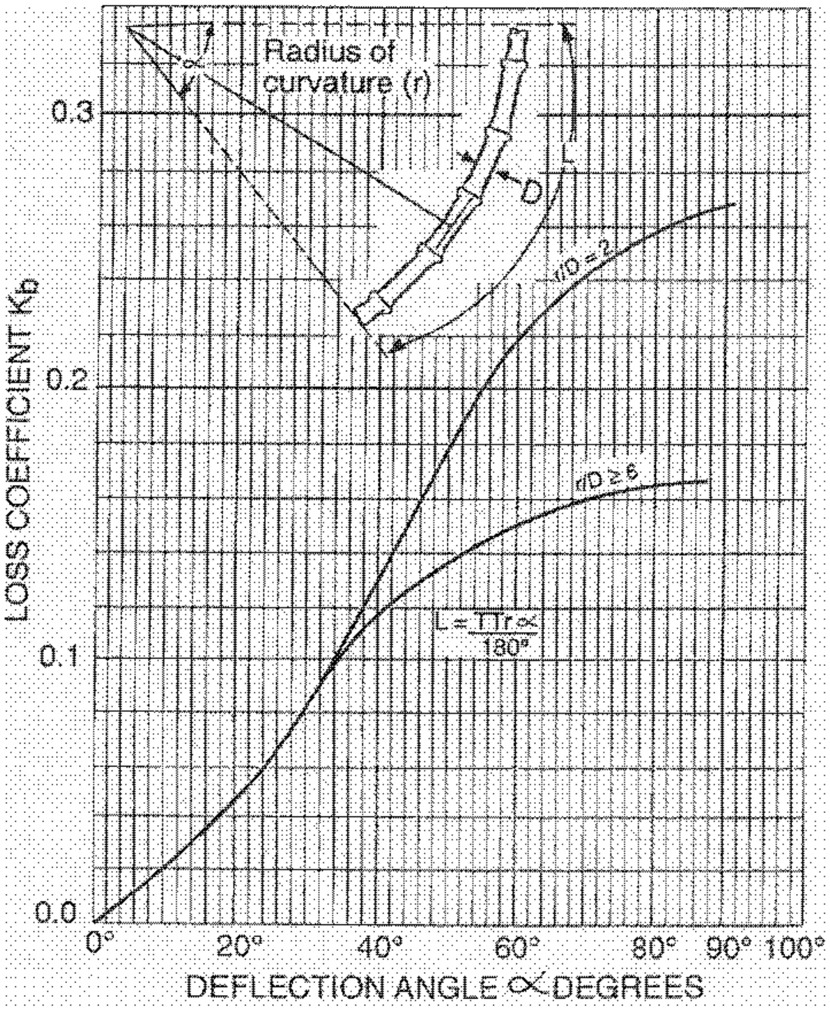
Source: Miller (1978)

Table 5.21.6

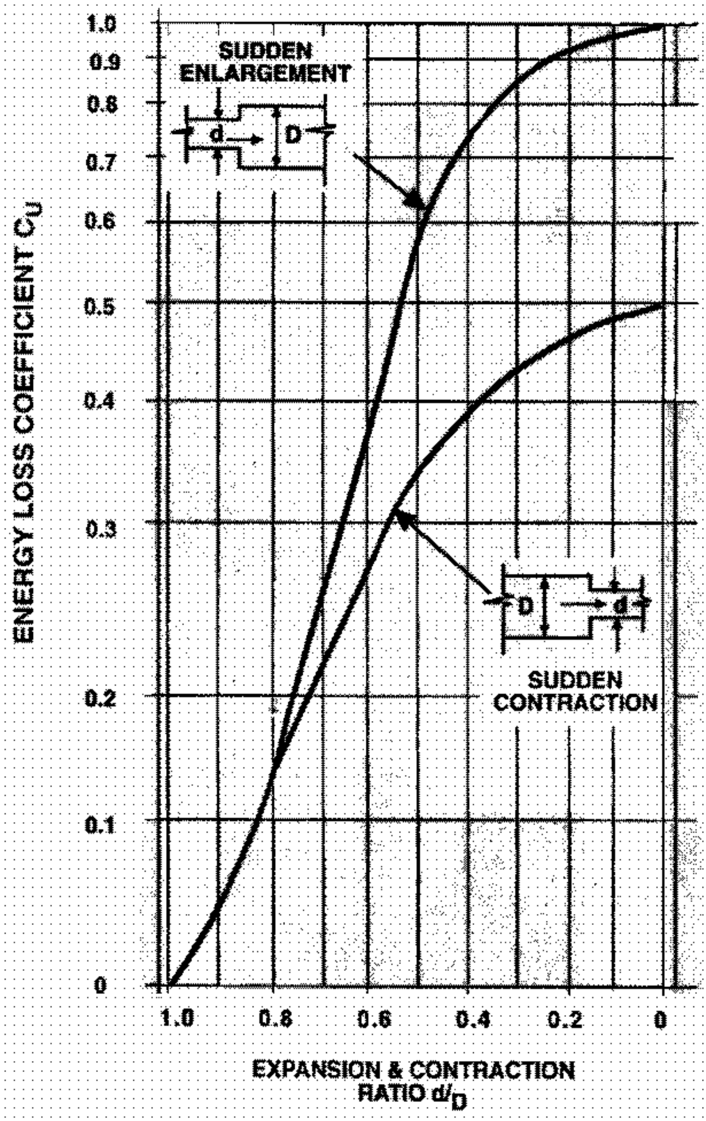
Pressure Loss Coefficients at Mitred Fittings

Type	K_L
90° double mitred bend	0.47
60° double mitred bend	0.25
45° single mitred bend	0.34
22½° single mitred bend	0.12

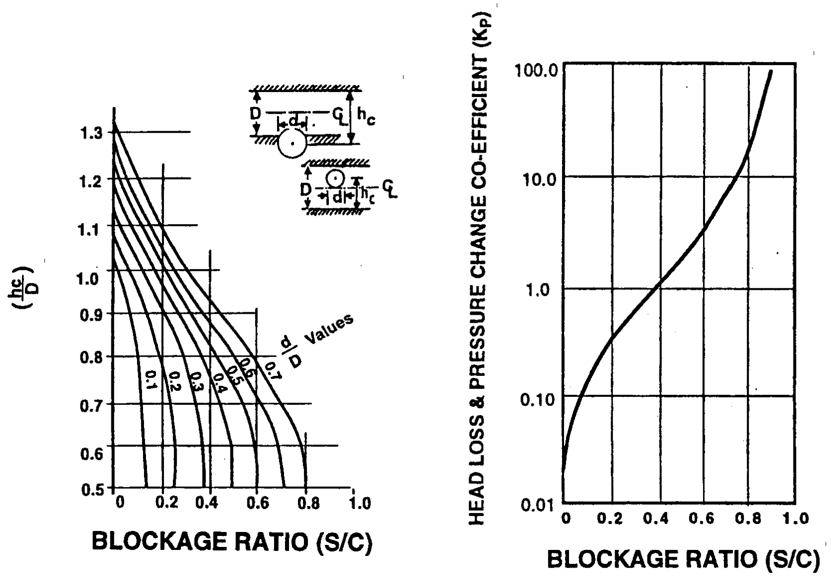
Source: ARR–1987 (p.327)



Bend Loss Coefficient
Source: D.O.T. (1992)
Figure 5.21.7



Source: A.S. 2200 - 1978



Pressure Changes Caused by Obstructions
Source: Black (1987b)
Figure 5.21.8

Table 5.21.4

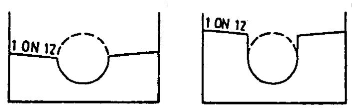
Potential Decrease in Pressure Change Coefficient as a Result of Benching

Manhole Type (3)	Potential Decrease in Pressure Change Coefficient (%)	
	Half-height Benching (1)	**Full-height Benching (2)**
Straight through	30	40
90° bend	20	40
Tee manhole with lateral inflow less than 50%	Nil	Nil
Tee manhole with lateral inflow approximately 50%	Nil	10
Tee manhole with lateral inflow approaching 100%	20	40

Notes:

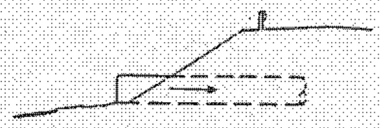
2.

3. Based upon testing of square manholes

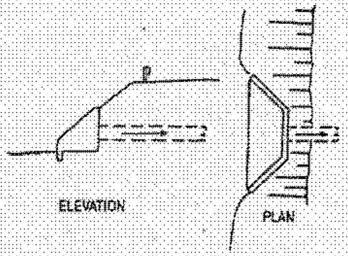


Half-height Full-height

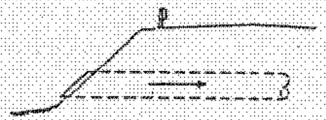
ENTRANCE LOSS COEFFICIENTS



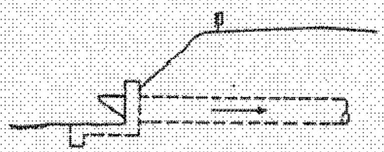
Projecting from Fill
Figure 5.21.6 (a)



Headwall with Wingwalls
Figure 5.21.6 (b)



Mitred to Conform to Fill Slope
Types of Entrances
Figure 5.21.6 (c)



Hooded Entrance
Figure 5.21.6 (d)

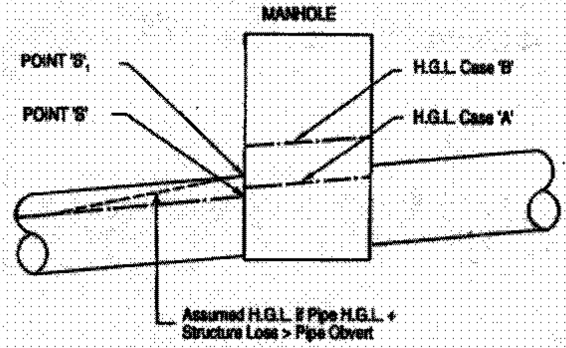
Table 5.21.5

Entrance Loss Coefficients

Type of Structure and Design of Entrance	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 conform 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 wingwalls square edge	0.5
Mitred to conform to fill slope	0.7
End section conforming to fill slope	0.5
Reinforced Concrete Box	
Headwall parallel to embankment (no wingwalls)	
Square edged 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).

Adapted from Hee (1969)



H.G.L. Determination for Pipes Partially Full

Figure 5.21.12

Configuration 1: Straight through line

(a) Determine H.G.L. at point 'S' for pipe running partially full.

(b) Add structure loss ($K_e . V_e^2/2g$) where V_e is the velocity in the downstream pipe running partially full and $K_e = 0.5$.

(c) (i) Case 'A': If the calculated H.G.L. at the structure is less than the obvert level (point S_1) adopt the calculated H.G.L. as the H.G.L.

(ii) Case 'B': If the calculated H.G.L. at the structure is greater than the obvert level (point S_1) then assume that the downstream pipe is running full at the outlet from the structure. A revised H.G.L. at the structure should then be determined using the appropriate head loss chart based upon the velocity in the downstream pipe running full, and with the structure loss added to the level of the obvert (point S_1).

Other Configurations:

A similar procedure should be used for the determination of H.G.L. except that in assessing the trial H.G.L. in step (b) the following values of K_e are recommended.

Table 5.21.7

Trial Values of K_e for use in

Determining H.G.L. under Partially Full Flow Conditions

Configuration	K_e
Straight through line	0.5
Change of direction 0° - 45°	0.75
Change of direction 46° - 90°	1.0
Multiple pipes	1.0

CULVERT DESIGN CRITERIA

3. HYDRAULIC DESIGN OF CULVERTS

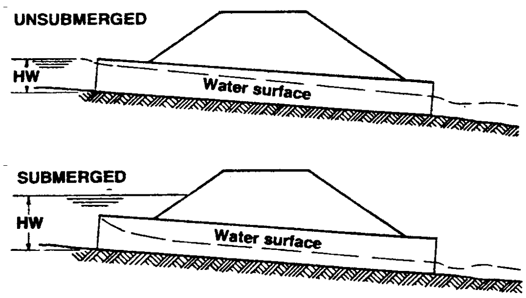
3.1 INTRODUCTION

Road Culverts, despite their apparent simplicity, are complex engineering structures from a hydraulic as well as a structural view point [3.5]. Their functional adequacy is no better than the estimate of the design flood, and the hydraulic design described below must be preceded by a careful flood evaluation together with an assessment of the cost resulting from damage caused by design flood being exceeded.
The hydraulic complexity of culverts is a result of the many parameters influencing their flow pattern. This influence can be summarized by referring to two major types of culvert flow.

3.1.1 TYPES OF CULVERT FLOW CONTROL

3.1.1.1 FLOW WITH INLET CONTROL

The culvert flow is restricted to the discharge which can pass the inlet with a given headwater level.
The discharge is controlled by the depth of headwater, the cross section area at the inlet and the geometry of the inlet edge. It is not appreciably affected by the length, roughness, slope or outlet conditions and the culvert is not flowing full at any point except perhaps at the inlet. This culvert type is mostly short or steep.



INLET CONTROL
FIGURE 3.1

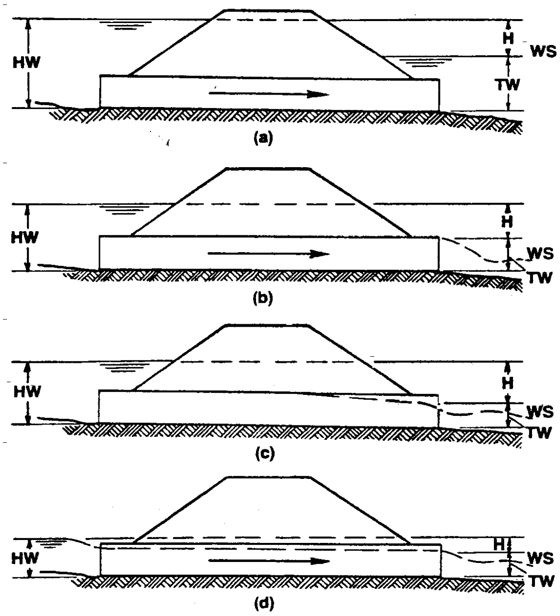
3.1.1.2 FLOW WITH OUTLET CONTROL

The culvert flow is restricted to the discharge which can pass through the pipe and get away from the outlet with a given tailwater level.
The culvert can run full over at least some of its length. The discharge is affected by the length, slope, roughness and outlet conditions in addition to the depth of headwater, the cross section area and inlet geometry.

3.1.1.3 DETERMINATION OF OPERATING CONDITION

It is rarely immediately obvious which pattern of flow a culvert is going to adopt, it is therefore necessary to investigate the consequences of both inlet and outlet control.

The most restrictive of the two flow types applies, i.e. the one giving least discharge for given headwater level, or requiring higher headwater level for given discharge.



OUTLET CONTROL
FIGURE 3.2

3.1.2 HEADWATER

Headwater (HW) is the depth of water at the inlet above the invert of the culvert. It is influenced by factors such as:

* Acceptable upstream flooding
* Pipe flow velocity
* Overtopping of the roadway
* Possibility of water penetration into the road or rail pavement.

Reference should be made to the appropriate Government authorities who have policies on headwater levels and the permissible frequencies and depths of road overtopping.

3.1.3 TAILWATER

Tailwater (TW) is the depth from the natural water surface at the outlet to the invert of the culvert. The tailwater level may be governed by downstream obstructions or the discharge from other streams.

3.1.4 FREEBOARD

Freeboard is the distance between the headwater level and the crown of the culvert. A minimum is sometimes included in Government authority's policies. [3.1]

3.2 CULVERTS WITH INLET CONTROL

Headwater-discharge relationships are for both pipes and box culverts strongly influenced by inlet geometry. [3.4].

Figure 3.3 shows the relationship between diameter, discharge and headwater depth for pipe culverts with square edged inlet and headwall, socketed inlet and headwall and socketed inlet projecting.

Similarly Figure 3.4 shows the relationship for box culverts with various wing wall angles.

3.3 CULVERTS WITH OUTLET CONTROL

A culvert flowing under outlet control may flow full, full for part of its length or even part full for its entire length as illustrated in Figure 3.2 (a), (b), (c) and (d).

3.3.1 CULVERTS FLOWING FULL

The simplest case of outlet control is illustrated in Figures 3.2a and 3.2b. Here the culvert is flowing full for its entire length. The energy head, H, required to maintain this flow can be expressed:

$$H = H_v + H_e + H_f$$

Where: H_v (velocity head) equals $v^2/2g$

H_e (entry loss) equals $k_e v^2/2g$

The entrance loss coefficient k_e is given in Table 3.1 for various pipe and box culvert entry conditions and culverts flowing with outlet control.

The energy loss, H_f is ideally calculated from the Colebrook-White equation (see 1.2.3.2) but in this particular context the Manning formula has been used because it has been used in [3.4] which forms the basis for most of this chapter.

Figure 3.5 shows the relationship between diameter, discharge and energy head for two different entrance loss coefficients and the selected value n = 0.011. Similarly Figure 3.6 shows the same relationship for box culverts.

Knowing the energy head H, the headwater, HW, can be calculated from the equation:

$$H = HW + Ls_o - TW$$

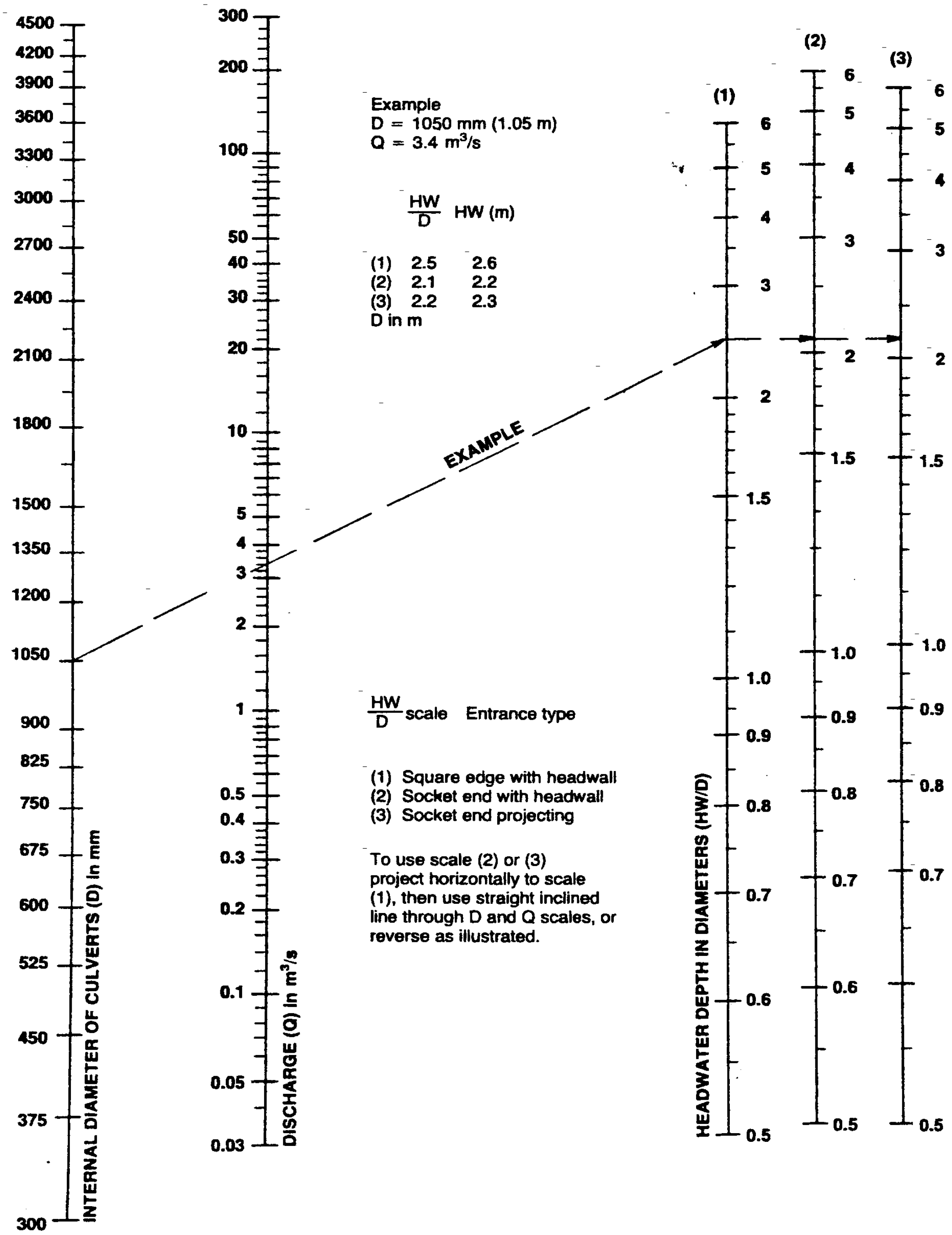
when TW is known.

This equation derives from Figure 3.7 and highlights the importance of the tailwater under outlet control.

DESIGN OF ENTRANCE	k_e
PIPE CULVERTS	
Pipe projecting from fill, square cut end	.5
socket end	.2
Headwall with or without wingwalls,	
square end	.5
socket end	.2
Pipe mitred to conform to fill slope, precast end	.5
field cut end	.7
BOX CULVERTS	
No wing walls, headwall parallel to embankment	
square edge on three edges	.5
three edges rounded to 1/12 barrel dimensions	.2
Wing walls at 30° to 75° to barrel	
square edge at crown	.4
crown rounded to 1/12 culvert height	.2
Wing walls at 10° to 30 ° to barrel	
square edge at crown	.5
Wing walls parallel (extension of sides)	
square edge at crown	.7

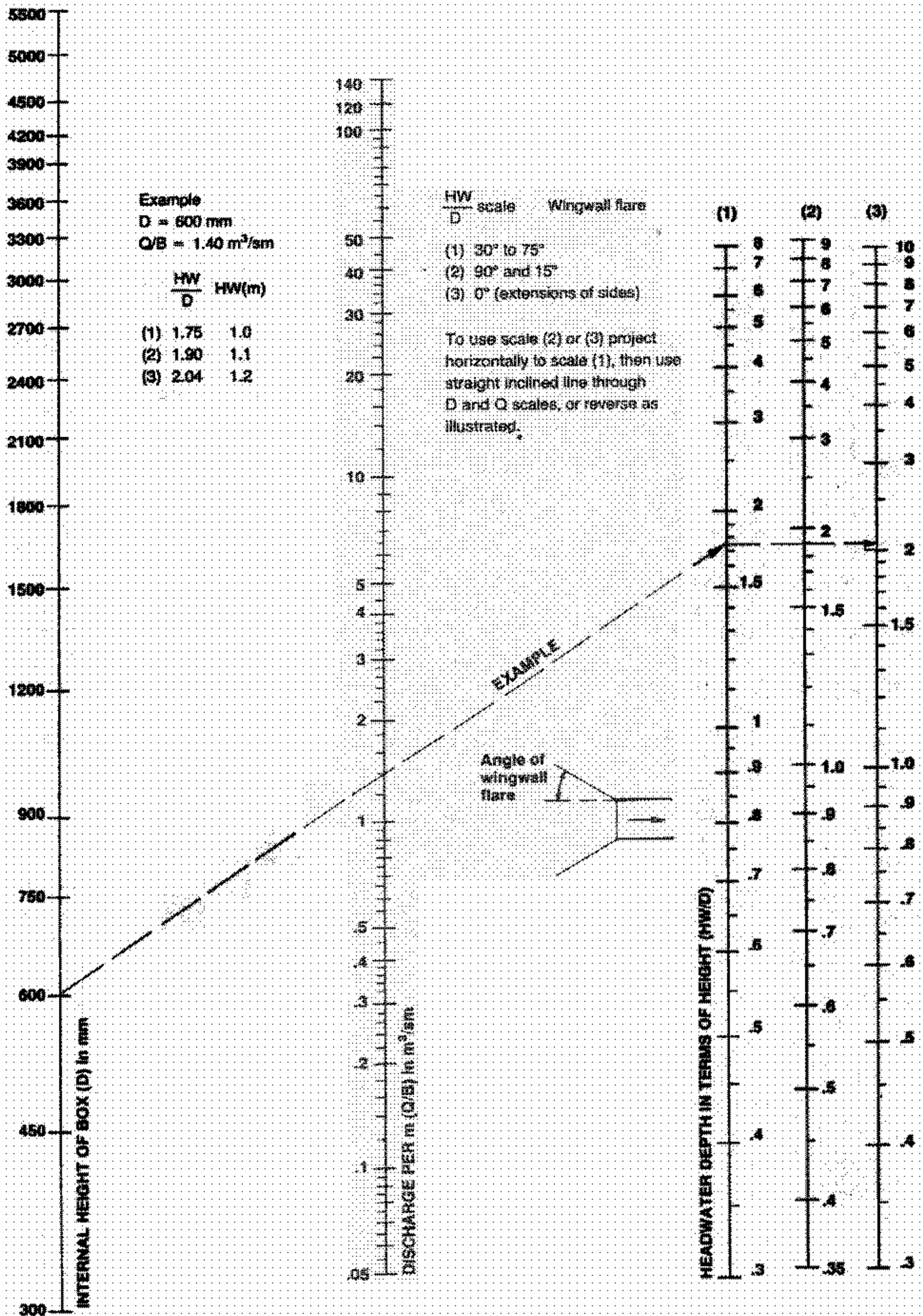
ENTRANCE LOSS COEFFICIENT k_e

TABLE 3.1 [3.4]



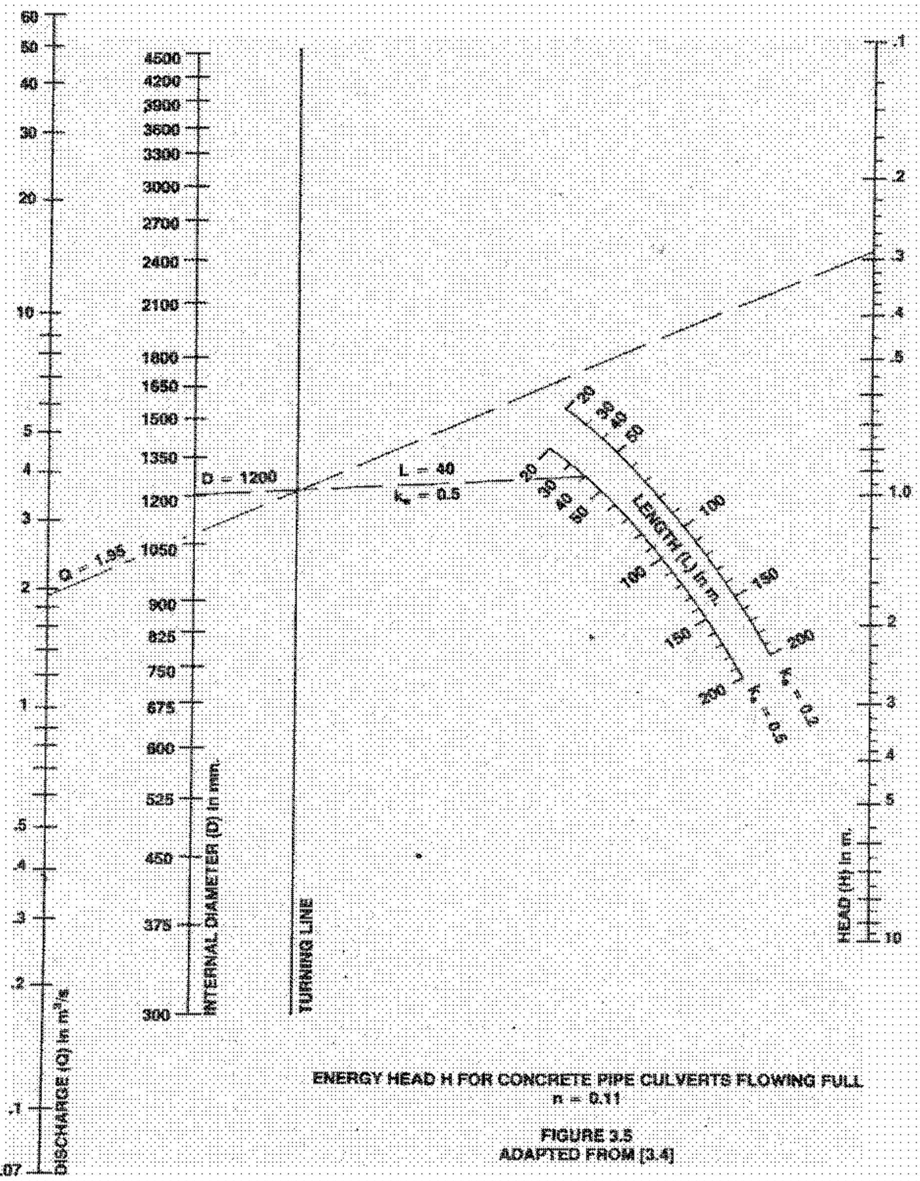
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS
WITH INLET CONTROL

FIGURE 3.3
ADAPTED FROM [3.4]



HEADWATER DEPTH FOR BOX CULVERTS
WITH INLET CONTROL

FIGURE 3.4
ADAPTED FROM [3.4]



ENERGY HEAD H FOR CONCRETE PIPE CULVERTS FLOWING FULL
n = 0.11

FIGURE 3.5
ADAPTED FROM [3.4]

NOTE:

(a) For a different value of n, use the length scale shown with an adjusted length $L_1 = L\,(n_1/n)^2$

(b) For a different value of k_e connect the given length on adjacent scales by a straight line and select a point on this line spaced from the two chart scales in proportion to the k_e values.