# City of Armadale

# Stormwater Management Handbook

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## Amendment Record

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.

| *Specification No. and Title* | *Clause No* | *Page No* | *Amendment Date* | *Amendment Summary* | *Code* | *Comments* |
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This amendment record is to be used for project specifications and internal records only.

The AUS-SPEC annual update summary included in the 'Reference documents' in SPECbuilder Pro will provide more details of the revisions/changes made to the individual Specifications during each update.

## 1. INTRODUCTION

This handbook provides design criteria to be used in submission of drainage designs presented to the City of Armadale Council. All basic design principles shall be in accordance with the *Australian Rainfall & Runoff (AR&R) 1987 and 1998*. Consultants should read this handbook in conjunction with the AR&R. This handbook supports the City's *Subdivision and Development Guidelines* drainage design.

## 2. RAINFALL

Intensity/frequency/duration data for the Armadale local government area is presented in Table 1 and Table 2.

Table 1 - IFD Data for the Armadale LGA (Source BoM)

| *Parameter* | *Value* |
|---|---|
| 2 year, 1 hour (mm/hr) | 23.12 |
| 2 year, 12 hour (mm/hr) | 5.01 |
| 2 year, 72 hour (mm/hr) | 1.54 |
| 50 year, 1 hour (mm/hr) | 37.38 |
| 50 year, 12 hour (mm/hr) | 7.82 |
| 50 year, 72 hour (mm/hr) | 2.55 |
| Skewness G | 0.68 |
| Geographical Factor for 6 minute, 2 year storm | 4.85 |
| Geographical Factor for 6 minute, 50 year storm | 17.12 |
| Latitude | -32 ₒ 9 ' 2.54198 " |
| longitude | 116 ₒ 0 ' 51.67186 " |

Table 2 - IFD Table (mm/hr) (Source BoM)

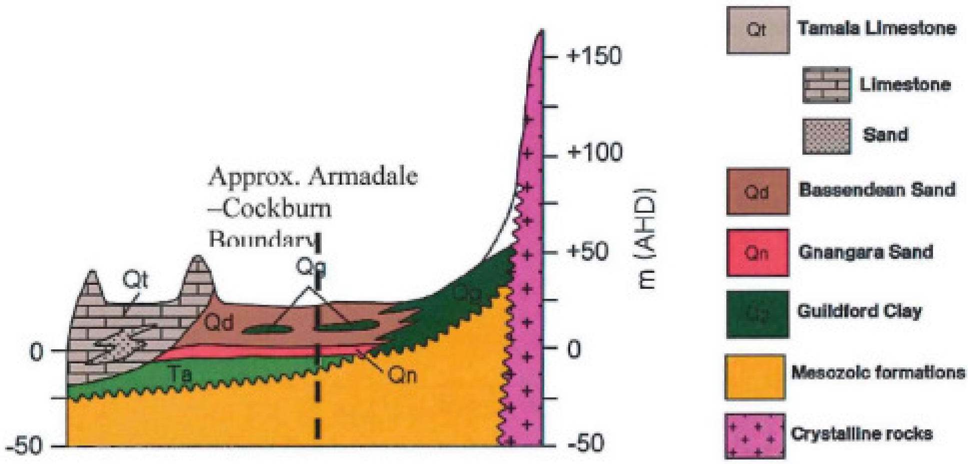
| | *Average Recurrence Interval (ARI)* | | | | | | |
|---|---|---|---|---|---|---|---|
| *Duration* | *1 Year* | *2 years* | *5 years* | *10 years* | *20 years* | *50 years* | *100 years* |
| 5 Mins | 64.7 | 84.6 | 109 | 126 | 149 | 184 | 214 |
| 6 Mins | 60.4 | 79 | 101 | 117 | 139 | 171 | 199 |
| 10 Mins | 48.3 | 62.9 | 79.8 | 91.7 | 108 | 133 | 153 |
| 20 Mins | 33.7 | 43.5 | 54.2 | 61.5 | 72 | 87 | 99.6 |
| 30 Mins | 26.8 | 34.4 | 42.4 | 47.8 | 55.7 | 66.8 | 76.1 |
| 1 Hr | 17.7 | 22.6 | 27.5 | 30.7 | 35.5 | 42.2 | 47.8 |
| 2 Hrs | 11.5 | 14.7 | 17.7 | 19.7 | 22.6 | 26.8 | 30.2 |
| 3 Hrs | 8.98 | 11.4 | 13.7 | 15.2 | 17.5 | 20.6 | 23.2 |
| 6 Hrs | 5.87 | 7.45 | 8.9 | 9.86 | 11.3 | 13.3 | 14.9 |
| 12 Hrs | 3.84 | 4.87 | 5.82 | 6.44 | 7.37 | 8.68 | 9.75 |
| 24 Hrs | 2.48 | 3.16 | 3.8 | 4.22 | 4.84 | 5.72 | 6.44 |
| 48 Hrs | 1.56 | 2 | 2.43 | 2.71 | 3.13 | 3.73 | 4.22 |
| 72 Hrs | 1.16 | 1.49 | 1.82 | 2.04 | 2.37 | 2.84 | 3.23 |

## 3. ARMADALE SOIL CHARACTERISTICS

The Armadale LOA has two distinct geological regions, with the Darling Scarp separating the elevated crystalline rocks to the east, and generally flat low lands comprised of sands and silty sands bedded with clays, ferricretes (coffee rock), limestone, silcrete and calcareous. The superficial formations are highly variable both spatially and vertically. Topsoils generally consist of dark grey brown, fine to medium grained silty non plastic sand.

Understanding of the soil characteristics is required to adopted viable stormwater management methods. Geotechnical investigations shall be carried out were necessary to adequately characterise the geology and soils of the development site. Geotechnical investigation requirements are contained in the City's *Subdivision and Development Guidelines*. Figure 1 shows a generalised cross-section of the superficial formation, from the Darling Scarp, to the coast.

Figure 1 – Generalised Cross Section showing the Superficial Geological Formations




## 4. ARMADALE GROUNDWATER HYDROLOGY

There are two aquifers that influence how stormwater is managed in Armadale, the superficial aquifer, and the semi-confined Leederville aquifer. Generally, only the superficial aquifer will have an important influence on stormwater management. The Superficial aquifer however, is in itself highly variable across the city in both thickness and hydraulic thickness.

## 5. HYDROLGICAL MODELLING

### Defining Modelling Objectives and Slope

Prior to commencement to hydrological modelling, the scope and objectives must be clearly stated to enable the most parsimonious yet informative modelling methodology to be used. When submitted any hydrological modelling, the designer must clearly state the modelling scope and objectives.

### Impervious Area Assumptions

#### Lot Scale

When estimating the design flow contribution from individual lots, due allowance should be made for possible future improvements and/or urban consolidation. For single residential lots, the total impervious shall be calculated directly, as a sum of the roofed area, in addition to directly connected impervious areas such as driveways, carports, paving etc.

### Composite areas

For catchment scale modelling of urban catchments, sub-catchments, composite or lumped design runoff impervious percentages are used. Table 3 provides typical impervious area percentages that may be adopted.

Table 3

| | $C_1$ | $C_5$ | $C_{10}$ | $C_{100}$ |
|---|---|---|---|---|
| *Residential lots* | 0.56 | 0.67 | 0.70 | 0.84 |
| *Access streets and road reserves* | 0.64 | 0.76 | 0.80 | 0.96 |
| *Group housing sites, mixed use commercial/ residential, local centre & laneways* | 0.72 | 0.86 | 0.90 | 1.00 |
| *POS basins* | 0.72 | 0.86 | 0.90 | 1.00 |
| *POS remaining areas* | 0.08 | 0.10 | 0.10 | 0.12 |

The designer shall assess whether the adoption of typical values is accurate enough for the purposes of the drainage analysis. This may be sufficient for preliminary design or master planning, however a more accurate assessment of total impervious area may be necessary for the investigation of stormwater system failures or detailed design. Where practical or necessary, runoff coefficients shall be calibrated to observed runoff data.

### Rational Method

The rational method is a simplistic method for determining the design peak flow rate, generally used to size drainage system components such as pits and pipes. Rational method calculations shall be carried out in accordance with AR&R and the requirements of this guideline. The rational method may be used only when the following criteria apply:

- The catchment area under analysis in less than 500 hectares (5km2)
- The catchment has a time of concentration less than 30 minutes, and is of a regular shape.
- The catchment does not contain significant detention basin storage or wide spread use of on-site.

### Coefficient of Discharge (C)

The value of the discharge coefficient is a statistical composite of infiltration and other losses. The process for determining the runoff coefficient is detailed in Australian Rainfall and runoff, and is outline as follows:

- Determine the fraction impervious (0)
- Determine the value of $C_{10}$ using Figure 2
- Determine the frequency factor for the required ARI (Table 4)

- Determine the 10 year C value

Figure 2 – Calculating $C_{10}$ from Fraction Impervious

*($C_{10} = 0.725f_i + 0.175$)*

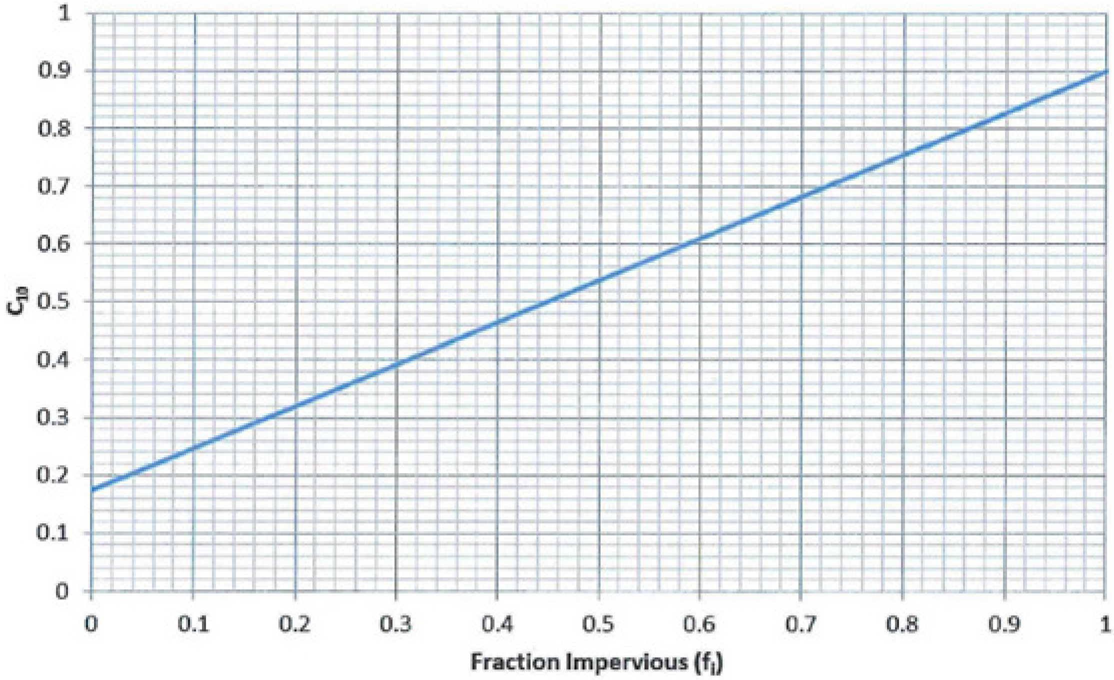



Table 4 - Frequency Factor

| *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 |

### Time of Concentration

The designer should justify the selection of time of concentration for the catchment. A minimum time of concentration shall be 6 minutes for urban areas. The maximum time of concentration shall be 20 minutes unless the sufficient evidence is supplied to justify a greater time. Partial area effects should also be taken into account by the designer. If there is doubt if the rational method is applicable for the catchment under consideration, then advice shall be sought from City of Armadale Technical Services.

### Flow Paths and Times

If the flow path is through areas having different flow characteristics or includes property and roadway, calculate the flow time of each portion of the flow path separately. Overland sheet flows shall be computed using the kinematic wave equation.

$$t = 6.94(L.n^*)^{0.6} / (I^{0.4}.S^{0.3})$$

where $t$ = overland travel time (min)
$L$ = overland sheet flow path length (m)
$n^*$ = surface roughness/retardance coefficient
$I$ = rainfall intensity (mm/hr)
$S$ = slope of surface (m/m)

Table 5 – Horton's Runoff Coefficient

| *Surface Type* | *Horton's Roughness Coefficient n** |
|---|---|
| Concrete or Asphalt | 0.010-0.013 |
| Bare Sand | 0.010-0.016 |
| Gravelled Surface | 0.012-0.030 |
| Bare Clay – Loam Soil (eroded) | 0.012-0.033 |
| Sparse Vegetation | 0.053-0.130 |
| Short Grass Paddock | 0.100-0.200 |
| Lawns | 0.170-0.480 |

# 6. HYDRAULIC DESIGN OF STORMWATER SYSTEMS

## Maintenance

The stormwater drainage system shall be designed to be readily and economically maintained by the City of Armadale, and shall incorporate appropriate access for maintenance machinery.

## Hydraulic Design Principles

Pipes shall be designed by a hydraulic grade line (HGL) method using appropriate pipe friction and drainage structure head loss coefficients.

### Minor and Major System

Recurrence intervals for minor events depend on the zoning of the land being serviced by the drainage system. The minor system design average return intervals (ARis) are detailed below;

- 10 years for commercial/industrial areas
- 5 years for residential areas
- 5 years for rural residential areas

The major system capacity, inclusive of the capacity of the underground system in addition to overland flows, shall be sized for the 100 year ARI event, except in cases as in which greater protection is required. Overland flow paths must be clearly identified and demonstrated to be safe and practical.

## Piped Drainage Systems

Damage pipe systems shall be designed as an overall system, with due regard to the upstream and downstream system and not as individual pipe lengths Drainage pipeline systems shall generally be designed as gravity systems flowing full at design discharge, but may be pressurised with the use of appropriate pits and joints.

Stormwater drainage pipes are to be designed and installed as a manhole to manhole system with trapped inlet pits (interceptors) from road gutters connecting to the manholes. Inlet pits are to be combination side entry pits.

Pipe friction loss, inlet, junction and outlet losses and other hydraulic losses shall be included when calculating pipe sizes.

### Provisions for Failure

It is important to ensure that the combined major/minor system can cope with surcharge due to blockages and flows in excess of the design ARI. If failure of cut-off drains, retarding basins or pipe system and floodway structures occurs during these periods, the risk to life and property could be significantly increased.

In establishing the layout of the pipe network, Designers shall ensure that surcharge flows will not discharge onto private property during flows up to and including 100 year ARI for flows m excess of the 100 year ARI event, designers shall ensure that the likelihood of nuisance flooding or damage to properties is minimised.

### Pipe roughness values

The following pipe roughness values shall be adopted:

Table 6 - Pipe Roughness Values

| *Pipe Material* | *Manning's n* | *Colebrook K* |
|---|---|---|
| Pre cast concrete | 0.013 | 0.6 |
| Fibre reinforced concrete | 0.011 | 0.3 |
| UPVC | 0.009 | 0.06 |

### Minimum Conduit Sizing

Minimum conduit sizes shall be as follows:

- Pipes - 375mm diameter.
- Box Culverts - 600mm wide x 300mm high

### Pipe Velocities

The velocity of flows within the stormwater pipes shall be as follows;

Table 7 – Pipe Velocity Limits

| *Flow Condition* | *Absolute Minimum [1] (m/s)* | *Desirable Minimum [1] (m/s)* | *Desirable Maximum [2] (m/s)* | *Absolute Maximum [2] (m/s)* |
|---|---|---|---|---|
| Partially full | 0.7 | 1.2 | 4.7 | 7.0 |
| Full | 0.6 | 1.0 | 4.0 | 6.0 |

Note 1: Minimum flow velocities apply to 1 in 1 year ARI design storm, and apply to all pipe materials
Note 2: Maximum flow velocities apply to concrete pipes. For other pipe materials, refer to manufacturer's advice

### Structural Pipe Design

The structural design of pipelines should be earned out in accordance with AS 3725 Loads on Buried Concrete Pipes, CPAA Pipe Class VI.I Concrete Pipe Selection Software and AS 2566.2 Buried Flexible Pipelines – Installation.

The minimum strength class for concrete pipes used within Councils system shall be class 2. Drainage lines within the road reserve shall be aligned in accordance with the Utility Providers Code of Practice for Western Australia 2000 or any revision thereto.

### Pipe cover

The minimum cover over pipes shall be as per Table 8

Table 8 – Minimum Pipe Cover

| *Location* | *Minimum Cover (mm)* | |
|---|---|---|
| | *Rigid Type Pipes e.g. Concrete, F.R.C.* | *Flexible Type Pipes e.g. Plastic or Thin Metal* |
| Residential private property, and parks not subject to traffic | 300 | 450 |
| Private property and parks subject to occasional traffic | 450 | 450 |
| Footpaths | 450 | 600 |
| Road pavements and under kerb and channel | 600 | 600 |

Reduced covers can be considered if it can be demonstrated that the reduced covers are compliant with AS 1342. The City may require greater covers to be adopted to cater for future road widening.

### Pipe Grade Limits

The maximum and minimum grades pipes shall be laid at are per table 9.

Table 9 – Pipe Grade Limits

| *Pipe Diameter (mm)* | *Maximum Grade (%)* | *Minimum Grade (%)* |
|---|---|---|
| 300 | 20.0 | 0.50 |
| 375 | 15.0 | 0.40 |
| 450 | 11.0 | 0.30 |
| 525 | 9.0 | 0.25 |
| 600 | 7.5 | 0.20 |
| 675 | 6.5 | 0.18 |
| 750 | 5.5 | 0.15 |
| 900 | 4.5 | 0.12 |
| 1050 | 3.5 | 0.10 |
| 1200 | 3.0 | 0.10 |
| 1350 | 2.5 | 0.10 |
| 1500 | 2.2 | 0.10 |
| 1650 | 2.0 | 0.10 |
| 1800 | 1.7 | 0.10 |
| 1950 | 1.5 | 0.10 |
| 2100 | 1.4 | 0.10 |
| 2250 | 1.3 | 0.10 |
| 2400 | 1.2 | 0.10 |

1. Based on maximum velocity for pipe flowing full of 60m/s
2. Based on m1mmum velocity for pipe flowing full of 10m/s except where Note 4 is applicable
3. Manning's n = 0.013 for all cases (concrete pipes)
4. The minimum grade of 10% (1 1000) is based on construction tolerance requirements
5. The m aximum grade requirement applies to both the pipe grade and the hydraulic grade
6. The mm1mum grades apply to the pipe grade only
7. Where a pipe is flowing less than half full for the design flow being considered, it is permissible to exceed the above maximum grades provided that the velocity limits specified m Table 6 are not exceeded

## Inlets and Junctions

### Inlet Capacity

Inlet pit capacity shall be designed to ensure flow widths are kept below widths outlined in section 0. Pit capacity relationships are available in Hydraulic Engineering Circular 22 of the US Federal Highway Administration (2nd Edition, 2003). On grade pits calculations are detailed in section 4.4.4 and sag inlet capacities are detailed in section 4.4.5.

A blockage factor shall be applied to inlet structures. Percentage of theoretical capacity, as outlined in Table 10 shall be applied to calculated pit inlet capacities.

Table 10 – Allowable Pit Capacities

| *Condition* | *Inlet Type* | *Percentage of theoretical capacity allowed* |
|---|---|---|
| Sag | Side Entry | 80% |
| Sag | Grated | 50% |
| Sag | Combination | Side inlet capacity only grate assumed |
| Sag | Letterbox | 50% |
| Continuous | Side Entry | 80% |
| Continuous | Grated | 50% |
| Continuous | Combination | 90% |

### Inlet and Junction Freeboard

For the design of piped systems for minor events, a freeboard shall be applied at each inlet and junction above the calculated water surface elevation (W.S E). The minimum freeboards that shall apply are as follows for inlets and junctions 150 mm.

### Inlet Spacing

The following maximum inlet spacing shall be used for inlet.

Table 11 – Maximum Inlet Spacing

| *Diameter (mm)* | *Maximum Inlet Spacing* |
|---|---|
| 225 to 450 | 100 |
| 525 to 900 | 150 |
| 1050 to 1200 | 300 |

The maximum inlet spacing does not preclude the designer to ensure flow widths checked. Inlets must be designed to ensure adequate inlet capacity to control flow width whilst providing practical and economic inlet spacing.

### Bubble-up Pit/Surcharge Pit

Bubble up pits should not be used when viable alternative outlets can be achieved if the designer feels that it is in the best interest of the system to use a bubble up, and then adequate justification shall be provided to Council.

Setting the controlling HGL (CHGL) in analysing the hydraulic performance of a stormwater system with a bubble-up pit discharge is important to analyse the performance of the system. The designer is to consider the location and invert levels in relation to the surrounding environment to appropriable adopt a CHGL. Justification of the adoption of the CHGL shall be provided when submitting engineering plans.

Bubble up pit (surcharge pit) energy loss shall be calculated by the summation of:

- Pipe to pit energy loss component (usually a 90° metre bend loss)
- Expansion loss component (if applicable)
- Screen loss component

- Exit loss component

- Function loss component (only for long chambers> 4 metres 1m depth)

## Overland Flow

An overland flow path is defined as:

- Where a piped drainage system exists, the path where stormwater exceeding the capacity of the underground drainage system would flow.

- Where no piped drainage system or other form of defined water course exits, the path taken by surface runoff from higher parts of the catchment. This does not include a watercourse or gully with well-defined banks.

Overland flow paths shall be clearly identified when submitting stormwater designs, and retained within the road reserve and public open space and not through private allotments.

### Overland flows in the road reserve

Minor and major systems rely on the use of the road to convey flows to inlet pits, or to other discharge locations. The capacity of the kerb to provide a channel for the intended flows must be considered during the design phase. Allowable Gutter flows for minor and major events are located in Appendix A

### Flow depth and width limitations

The maximum flow depth for all areas within the road reserve and foot paths shall be 0.2 meters. The velocity depth product (v.d) for all overland flow paths shall not exceed the following limits:

- Flows within the road reserve and across footpaths – 0.4m$^2$/s

- All other overland flows where only the safety of vehicles 1s affected- 0.6m$^2$/s

In all cases, the mam objective for the design of surface flows is to ensure the safety of pedestrians and vehicles. Allowable Flow widths for roads are given in table 12.

Table 12 – Allowable Flow Widths in Road Reserves

| *Situation* | *Allowable Flow Widths (m)* |
|---|---|
| Two-way cross fall | 1.5 |
| One-way cross fall | 2.0 |
| Access ways and access places | 2.5 |
| Pedestrian pram ramps and crossings | 0.5 |

# 7. OPEN CHANNELS

## Design Capacity

Open channels shall be designed to cater up to and including the 100 year ARI event without erosion or damage occurring to the channel. In calculating the design capacity for the channel, the designer must take into account the final vegetated state in deriving an appropriate surface roughness.

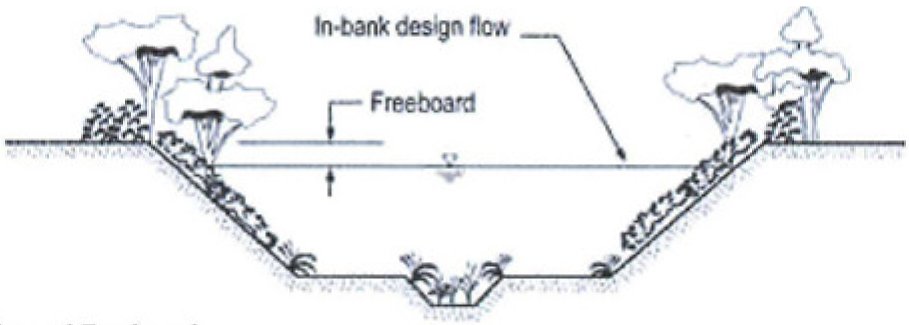
## Channel Freeboard

In situations where overtopping of a channel could result in an undesirable outcome, such as flooding of private property, then the following freeboard shall be applied to the channel as follows;

Table 13 – Channel Freeboard

| *Waterway Type* | *Freeboard (mm)* |
|---|---|
| Flood ways and natural | 300 |
| Swales | 100 |

Figure 3 – Channel Freeboard




## Minimum Channel Grades

Channels and swales shall be constructed with sufficient longitudinal grade to ensure that ponding and/or the accumulation of silt does not occur, particularity in locations where silt removal would be difficult. Longitudinal grades shall not produce velocities less than 0.8 m/s in low flow conditions flowing full. Generally channels and swales will require a minimum 0.5%, however lower values can be adopted if it is demonstrated that siltation will not occur.

## Swale Batters

The maximum batters for channels and swales shall be as per Table 14.

Table 14 – Maximum Permissible Batter

| *Swale Location* | *Maximum Permissible Batter* |
|---|---|
| POS or living stream areas | 1 in 8 for grassed areas<br>1 in 6 to 1 in 4 for landscape areas within passive POS<br>1 in 2 for terraced landscaped areas with retaining walls |
| Within verge areas of road reserve | 1 in 8 for grassed areas<br>1 in 6 for landscaped areas |
| Within median areas of road reserve | 1 in 8 for grassed areas<br>1 in 6 for landscaped areas |

## Stormwater Discharge to Channels

The design of stormwater outlets shall consider the following;

### Integration into the local character

- Appropriate integration of the outlet into the aesthetics and functions of the immediate area.
- Stormwater outlets within or adjacent to public areas should not interfere with the intended functions and management of the area.
- Outlet headwalls may be formed from materials such as precast concrete, decorated in-situ concrete, stacked rock, grouted rock, gabions, or integrated into non-related structural features such as observation decks or retaining walls.

### Safety Aspects

- Barricades installed where applicable. If the drop height exceeds 0.9 metres fencing is recommended and should be designed to sustain the imposed actions specified in AS1170.1.
- To the maximum degree allowable within the relevant codes, the choice of materials used in the construction of safety barriers (eg: tubular metal, treated timber logs, vegetative barrier) should integrate well with the character of the area.
- Wherever practical, the use of outlet screens should be avoided.
- Outlet screens shall not be used in circumstances where a person could either enter, or be swept into, the upstream pipe network. In this context, the term "outlet" refers to stormwater discharge points, not to outflow systems in water storage structures such as detention/retention basins.
- Maximum 150mm clear bar spacing for outlet screens. Bar screens should also be set a maximum 150mm above the pipe/channel invert.
- Appropriate access must be provided to the screen for dry weather maintenance including the removal of debris.
- Outlet screens should have a removable feature for maintenance access.

- Outlet screens on pipe units up to 1800mm in width should be designed such that the full width of the outfall pipe/box can be accessed for periodic maintenance.

### Location of Outlets

- All screens should be secured with tamper-proof bolts or locking device.

- Outlet screens should be structurally designed to break away under the conditions of 50% blockage during the pipe's design storm event.

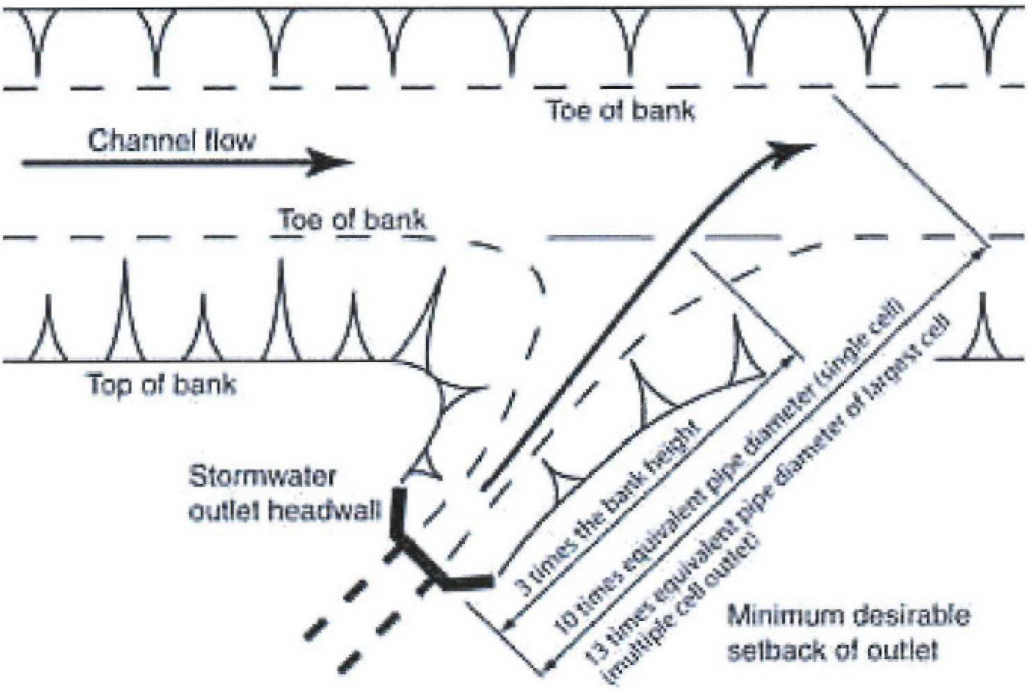
- Consideration should be given to the hydraulic consequences, including upstream flooding, resulting from debris blockage of outlet screens.

- Where practical, stormwater outlets should be recessed into the banks of any watercourse that is likely to experience bank erosion, channel expansion, or channel migration. Typically the minimum desirable setback is the greater of:

    - 3 times the bank height from the toe of the bank, and

    - 10 times the equivalent pipe diameter (single cell) or 13 times the equivalent diameter of the largest cell (multiple outlets) measured from where the outlet jet would strike an erodible bank (Figure 4)

Figure 4 – Minimum Desirable Outlet Setback




- Prior to recessing an outlet into a waterway bank, consideration should be given to the long-term impact on the riparian zone.

- Where it is not practical to recess the outlet into the bank, and outlet jetting from the pipe is likely to cause erosion on the opposite bank, then consideration should be given to measures that would reduce the outlet velocity.

- Where practical, stormwater outlets should be located away from highly mobile or erodible stream banks, or the outside of channel bends where turbulence generated by the outlet structure could initiate or aggravate bank erosion.

### Direction of Outlets

- Outlets that discharge into a "narrow" receiving channel should be angled 45 to 60 degrees to the main channel flow. A receiving channel is considered narrow if:
    - The channel width at the bed is less than 5 times the equivalent pipe diameter; or
    - The distance from the outlet to the opposite bank (along the direction of the outlet jet) is less than l 0 times the equivalent pipe diameter; and
    - The inflow is more than 10% of the receiving channel flow.

- Stormwater outlets that discharge in an upstream direction need to be avoided wherever practical.

### Elevation of Outlets

- If the outlet discharges into a permanent sedimentation basin or other stormwater treatment system, then the outlet should discharge above the designated sediment clean-out level.

- Submerged outlets shall be avoided for reasons of maintenance, including inspections and de-silting operations.

### Sedimentation and pollution control

- To the maximum degree practical, the outlet should not provide suitable habitat for the breeding of biting or nuisance insects. This may be achieved through appropriate design of the outlet, and/or by controlling sedimentation within and immediately adjacent to the outlet.

- To minimise sedimentation within the pipe, a minimum 1 year ARI flow velocity of 1.2 m/s is desirable.

- If significant sedimentation problems are expected at, or within the outlet, then the City shall be consulted in regards to their preference for an open channel or piped outlet.

### Maintenance requirements

Consideration should be given to the requirements for safe inspection and maintenance access

### Erosion control

- To the maximum degree practical, stormwater discharge from the outlet shall not cause bed or bank erosion within the receiving waterway/channel

- If outlet flow velocities are to be reduced by lowering the gradient of the small length of pipe immediately upstream of the outlet, then this length of pipe should be at least 15 times the hydraulic depth (partial full flow)

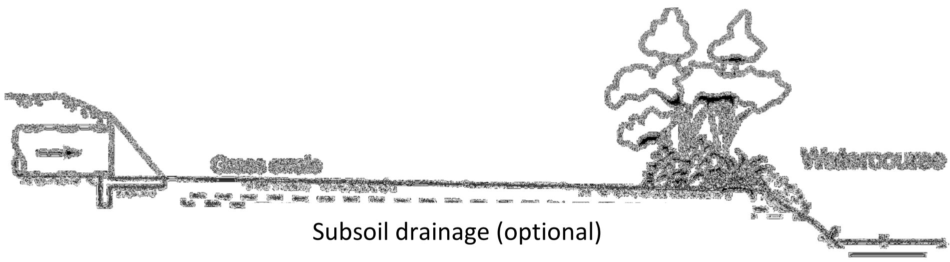
- Nominal scour protection should be included for a minimum distance of three pipe diameters from the face of the outlet if exit velocities do not exceed 2m/s

- If exit velocities exceed 2m/s, then a site-specific outlet scour control/energy dissipater will be required

### Discharge to Grass Swales

Reference is made here to the design of outlets that discharge to drainage swales, grass channels, or spoon drains as shown in figure 5.

Figure 5 – Discharge to Swale or Spoon Drain




- Outlet's invert level at least 50mm above the design invert of the grass swale to allow for normal grass growth

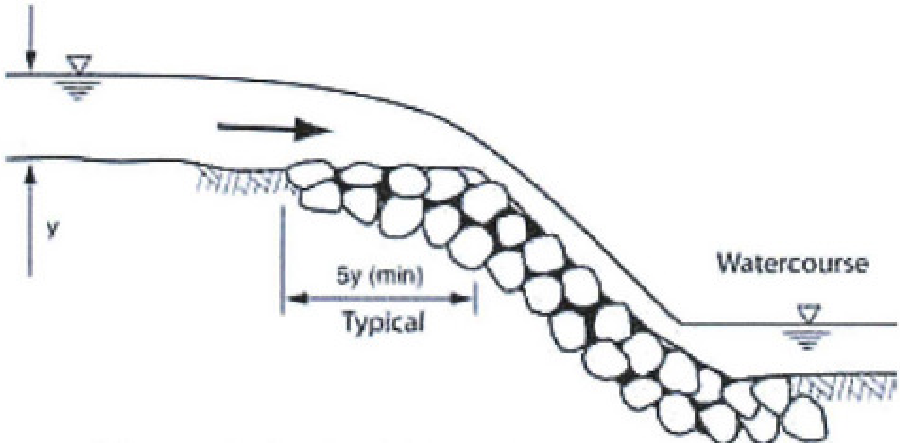
- 50 year ARI depth times velocity product (dB) within the swale should not exceed 0.4 to 0.6 depending on safety risk

- Hydraulic analysis must consider total flow within the swale, including flows that enter the swale as overland flow

- Subsoil drainage (including suitable pervious bed materials) may be required to minimise long-term soil saturation along the swale invert to facilitate regular maintenance activities.

- Final discharges from the swale into a waterway or open channel must incorporate adequate scour protection. Scour protection may include a loose rock chute, or stepped spillway. In general this scour protection should extend at least five times the nominal flow depth upstream of the chute crest to protect the swale from accelerating flow velocities as shown in figure 6

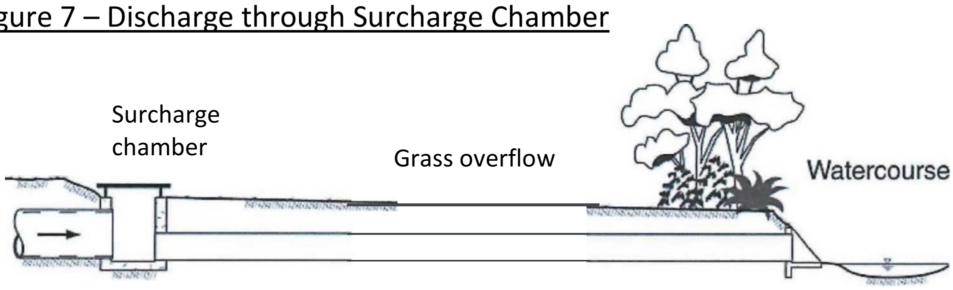
Figure 6 – Recommended Scour Protection at Crest of Drop Chutes




### Discharge through Surcharge (Bubble-up) Pits

Surcharge chambers are commonly used when stormwater systems discharge through a park or open space where a lower drainage standard is allowed compared to the upstream drainage system as shown in Figure 7.

Figure 7 – Discharge through Surcharge Chamber




Prior to incorporating a surcharge chamber into a drainage design, the following should be considered:

- The potential for a person (that has been swept into the upstream drainage system) being trapped inside the surcharge chamber and unable to exit through the chamber or the outlet pipe.
- Potential upstream flooding problems caused by debris blockage of the outlet screen.
- Structural integrity of the outlet screen and concrete coping, and its ability to withstand high outflow velocities and high bursting pressures caused by partial debris blockage.

- Safe maintenance access to allow removal of debris trapped within the surcharge chamber.

### Natural Channel Design

The design procedure for natural channel design is described in the *West Australian Stormwater Management Manual.*

## 8. WATER SENSITIVE DESIGN

### Water Sensitive Urban Design (WSUD) Design Criteria

All WSUD devices and structures shall be designed in accordance with the *Stormwater Management Manual for Western Australia (Department of Water, 2004)*, and must take into consideration any relevant information from WSUD research. The following general requirements are made of water quality treatment systems/devices:

- Discharges for an average 1.5 year Average Recurrence Interval be maintained at pre-development levels for stormwater treatments.

- Treatment types shall be determined by the Developer, subject to approval by the City after satisfying the City's requirements for (1) maintenance (2) ongoing costs and (3) stormwater quality standards achieved.

- Developers of industrial estates will be required to contribute to treatment off site if the City has whole of catchment treatment. Developers may also need pre-treatment within the proposed development in accordance with the requirements of the City's Stormwater Management Plan. Where whole of catchment treatment is not available Developers will be required to provide separate treatment for the development.

- Staging and construction of developments is to be considered. Only construct treatments when there are sufficient houses built to generate sufficient runoff to keep plants alive. The City is to bond the value of the plantings in preference to have planting at inappropriate time

- No manual handling is to be involved for the cleaning and maintenance of structures and equipment associated with the treatment of stormwater. Routine maintenance must be earned out without need for access of confined spaces.

- Developers shall undertake a risk assessment for all treatment sites. The assessment shall include fencing, grates across drams, wetlands, retarding basins, pumping stations, and other associated structures

- Operational documentation and manuals to be provided prior to the commencement of Defects Liability Period

### Vegetated Swales/Grass Swales/Buffer Strips

The design requirements for vegetated swales/buffer strips are as follows:

- The longitudinal slope of a swale is the most important consideration swales are most efficient with slopes of 1% to 4%. Lower than this, swales become waterlogged and/or have stagnant pooling, while steeper slopes may have high velocities (with potential erosion and vegetation damage risks). Check banks (small porous walls) may be constructed to distribute flows evenly across the swale if they are identified as the most suitable treatment option in such areas.

- Swale side slopes are typically 1 in 8. For maintenance, grassed swales requiring mowing must not have side slopes exceeding 1 in 4

- Vegetated swales, grass swales and buffer strips shall be designed and constructed m accordance with the guidelines set out in *Western Australian Stormwater Management Manual*

Vegetated swales are generally not acceptable in industrial subdivisions.

### Bio-retention Basins and Rain Gardens

The design requirements for bio-retention basins and rain gardens are as follows:

- Water ponding at entry points to the swale should not occur for longer than 1 hour after the cessation of rainfall
- Bio-retention basins and rain gardens shall be designed and constructed m accordance with the guidelines set out in *Western Australian Stormwater Management Manual*

### Gross Pollutant Traps (GPT) and Sediment Traps

#### General

Determine the best location(s) for GPT(s)/sediment trap(s) and its catchment size in conformance with ARQ clause 8.4 and the following:

- Complementary with the strategic catchment treatment objectives

- Topography

- Available space

- Proximity to pollutant source areas

- Outlet approach use a single device to treat a whole catchment (up to 200 ha or more)

- Distributed approach. Target smaller individual catchments with many traps

- Site constraints including topography, soils and geology, groundwater, space, access, odour problems, visual impacts, safety concerns and vermin

### GPT/sediment trap performance and type

Determine the performance for GPT and sediment traps in conformance with ARQ clause 8.5 including the following:

- Treatment objectives define the objectives for the project. Remove litter and vegetation larger than 5mm. Sediment Remove particles larger than 0.125mm.

- Operating design flows select the design flow in conformance with ARQ chapter 7.

- Flood capacity analyse hydraulics of the drainage system including the head-loss of the GPT and diversion weir under flood conditions. Check the design of the bypass system for impacts on the local drainage system and consequences on flooding.

- Trapped pollutant storage. Assess the pollutants that are likely to be collected and determine the holding capacity w1th respect to the maintenance operations and frequency.

- For design purposes the cleaning frequency of GPT's is six (6) months maintenance requirements. Design the GPT for maintainability and operability including the following considerations. Ease of maintenance and operation access to the treatment site frequency of maintenance disposal.

### Assessment of GPT performance

Include in the maintenance program requirements for validating the GPT performance by field monitoring, physical laboratory models or computer simulation.

### Selection of the GPT

Design the GPT with consideration of the following and the checklist available in ARQ Appendix 8A:

- life cycle costing

- Footprint and depth of the unit

- Hydraulic impedance and requirements

- Disposal costs

- Occupational health and safety

### Hydrocarbon management

Where required, design and size water/oil separators or interception devices in conformance with ARQ clause 9.7.

## 9. DETENTION AND INFILTRATION BASINS

Detention basins are generally used to reduce downstream flow rates as not to produce flooding or other detrimental effects to the downstream environment. Detention basins should be comprehensively planned and designed as a part of an overall catchment drainage strategy. This should occur early on in the planning process to allow optimal safe and practical design of the basin whilst integrating well into the built environment.

Infiltration Basins dispose of stormwater through movement of the soil strata, allowing stormwater to be an input into a local or regional unconfined aquifer. Infiltration basins are not to be considered water quality treatments unless they incorporate water quality treatment methods within the basin. Generally, soils in Armadale are not suitable for infiltration basin, either due to low soil permeability and/or high groundwater tables. If infiltration basins are proposed by the designer, then all supporting site investigation information shall accompany the proposal.

### Analysis

The Designer shall test the performance of the basin using a range of 'design' storms or a long term record of rainfall to determine the maximum storage requirements and the size of outlets for the basin. It must demonstrate that the basin is designed for the critical storm duration for the basin.

A hydrograph estimation technique shall be used to estimate appropriate inflow hydrographs to the basin. Inflow hydrographs shall be routed through the basin using a full reservoir routing calculations to determine the basin characteristics and resultant outflow hydrographs.

### Outlets

Outlet configuration will generally be designed to meet flow control requirements, and often the hydraulics for such devices is complicated and difficult to analyse. The designer is to ensure that the outlet is checked to operate efficiently and safely under the range of flow regimes that the structure will operate under.

Outlet structures shall be designed to reduce the risk of blockage. The consequences of blockage shall be investigated, and if found to be significant, mitigating measures shall be incorporated into the final design. The consequence of a rainfall event overtopping a basin shall be considered. If there is a potential for damage to downstream property, than a secondary outlet shall be provided for the basin. The secondary outlet allows a non-catastrophic means of failure above the 100 year ARI.

### Emptying Times

Emptying times for detention basins and infiltration basins shall be analysed to ensure that detention or retention volume is available for future rainfall events. Whilst detention basins generally tend to return to pre-storm state quickly, emptying times become important for infiltration basins. In lieu of detailed 'continuous simulation' modelling as described in 3.5.3 WSUD: Basic Procedures for 'source control' of stormwater (Argue2008), the required emptying time for minor events shall be 0.5 days, and for major events shall be 3.0 days. For all detention and infiltration basins, the designer must demonstrate emptying times are compliant.

### Grades

Basins shall embankment slopes shall have a maximum batter of 1 in 6. Slopes up to 1 in 4 may be approved is special circumstances. The basin floor shall be designed to ensure positive drainage to minimise the likelihood of ponding. The absolute minimum grade to be adopted for basin floors is 1 in 50, however the designer must take into account the surface roughness and construction tolerances when determining basin floor grades. For embankments with grades greater than 1 in 6, a detention basin risk mitigation assessment shall be submitted to the city. The Detention Basin Risk Management is located in Appendix B.

### Maintenance Access

A maintenance access shall be provided to each basin to enable appropriate access to vehicles and equipment during maintenance activities. The Designer is to ensure the access location has a safe access point from the road reserve. The maintenance access shall have minimum width of 4 metres.

### Maximum Depths

Maximum depths shall be, where practical, less than 1.2 metres for the 20 year ARI. For basin depths greater than this, a detention basin risk mitigation assessment shall be submitted to the city The Detention Basin Risk Management is located in Appendix B.

### Freeboard

In situations in which a breach of the top of bank of a basin may result in damage to property or endangers pedestrians or vehicles, then a freeboard shall be applied to the basin of at least 100mm. The designer shall consider the risk and consequence when determining the freeboard requirement.

## 10. ON-SITE STORMWATER DESIGN

### Legal and Practical Point of Discharge

Each proposed development must have both a legal point of discharge and a practical point of discharge. The former consists of a location to which the owner has the legal right to discharge water to or through each lot may be permitted via approval to discharge into council's stormwater system or to Water Corporation managed drain. A practical point of discharge refers to a location that the subject site can physically drain to.

The developer of a site must demonstrate that the development has a legal point of discharge and a practical point of discharge. It is the developer's responsibility to seek downstream easements if required.

### Permissible Discharge Rate

Unless detailed otherwise, stormwater discharge to the City's drainage system must be at or below pre-development rates. Post-development flow rates are to be attenuated to pre-development flow rates through the provision of adequate temporary detention storage or onsite disposal methods such as infiltration. "Pre-development" state of infill developments refers to the original undeveloped state of the block.

## On-site Stormwater Management Methods

### Sites with Pervious Soils

Sites with pervious soils are sites which can support the management of stormwater via infiltration methods. Soils with a saturated hydraulic conductivity of 3.6 mm/hr to 360 mm/hr are preferred for infiltration applications. Soils with a low hydraulic conductivity (0 36 - 3 6 mm/hr) do not necessarily preclude the use of infiltration systems even though the required infiltration/ storage area may become prohibitively large.

For sites that can support infiltration measures, the following criteria must be met to achieve City of Armadale approval.

- Demonstrate course free draining sand is to a depth of 10 metre below the lowest invert of the proposed infiltration device used. If the infiltration device is unknown, then solid testing to a minimum depth of 2.8 metres shall occur. A geotechnical report shall be submitted to the City for approval.

- Confirm that the area is not susceptible to high ground water or high winter groundwater levels. The City may request geotechnical analysis to determine maximum water groundwater levels. Adequate separation between maximum water groundwater levels and any structures is required generally 10 metres clearance from peak groundwater levels to the base of the infiltration device is applied, however this can be reduced if the designer demonstrates that the infiltration device is designed to operate in close proximity to the groundwater level.

- The proposed system shall demonstrate that the required discharge rate is achieved via submission of engineering calculations.

### Soakwells

There is a prolific use of soakwells in Western Australia, and currently is the dominant method of infiltration disposal. When soakwells are proposed to be used, the following requirements need to be demonstrated:

- Soakwells need to be a minimum 1m above the maximum groundwater level as described by a geotechnical report or site investigations.

- Comply with minimum separation from building pads, Table 15.

- Sized accordingly to the requirement of both infiltration capacity and storage volume as dictated by the site allowable discharge

Table 15 – Minimum Separation between Infiltration Devices and Buildings

| *Soil Type* | *Hydraulic Conductivity* | *Minimum Distance to Footings (m)* |
|---|---|---|
| Sand | >180mm/hr | 1.0 |
| Sandy clay | 180 to 36mm/hr | 2.0 |
| Medium clay | 36 to 3.6mm/hr | 4.0 |
| Reactive clay | 3.6 to 0.036mm/hr | 5.0 |

Table 16 – Typical Soakwell Volumetric Capacity ($m^3$)

| *Depth (mm)* | *600mm dia.* | *900mm dia.* | *1050mm dia.* | *1200mm dia.* | *1500mm dia.* | *1800mm dia.* |
|---|---|---|---|---|---|---|
| 600 | 0.17 | 0.38 | 0.52 | 0.68 | 1.06 | 1.53 |
| 900 | 0.25 | 0.57 | 0.78 | 1.02 | 1.59 | 2.29 |
| 1200 | 0.34 | 0.76 | 1.04 | 1.36 | 2.12 | 3.05 |
| 1500 | | 0.95 | 1.30 | 1.70 | 2.65 | 3.82 |
| 1800 | | 1.15 | 1.56 | 2.04 | 3.18 | 4.58 |

### Other Infiltration Methods

Where appropriate, the City will support the use of other infiltration devices other than soakwells. A range of infiltration methods are outlined in the Stormwater Management Manual of Western Australia along with appropriate calculation methods. Design calculations shall be submitted to Council when proposing an infiltration device.

Design requirements for infiltration and aquifer recharge systems. Submit calculations demonstrating the effectiveness of the infiltration device for successions of storms and hydrological effectiveness to ARQ clause 11.4.

System design: Conform to ARQ clause 11.3.4 for the following:

- Unsuitable soils. Test soils for permeability and assess for suitability.
- Clearance distances to building footings and boundaries conform to ARQ clause 11.3.1 with regard to the soil classification.
- Rock and shale. Test for permeability and assess for suitability.
- Shallow soil cover over rock. Test for permeability and assess geology for weathered or fractured rock.
- Steep terrain. Check soil depth on a downslope and assess suitability.
- Water table interaction with infiltration systems. Check water table stability and salinity for suitability and the presence of any aquifers that may interact.
- Water table affected by upstream infiltration devices. Assess geology for any likely upstream infiltration devices dev1ces that may limit retention.

- Aquifer recharge/retrieval annual balance. Assess for continual equilibrium of local potentiometric levels

- Water quality inflows to infiltration devices. Provided treatment is required for all water running directly into soakwells in conformance with ARQ clause 11.2.3

### Sites with Impervious Soils

If site conditions do not permit on-site infiltration then stormwater systems should be designed to balance the pre and post development 5 year and 100 year critical flows with suitable attenuation measures before discharge to an adequate outlet system - Local authority or Water Corporation managed drain. The city may impose greater restrictions on discharge rates in to the Council stormwater system with due regard to the upstream and downstream conditions.

## On Site Detention Methods

On site detention methods shall be applied to reduce discharge to Councils system. The site storage is required to temporarily store rainwater during a storm, while the flow out of the storage is controlled. Generally, storage is to be provided underground, however the City may accept some storage above ground (for example, in a suitably shaped driveway, or paved area) for events greater than the 5 year ARI. Temporary above ground storage of stormwater must not exceed a depth of 150mm for the critical 100 year ARI storm event, and the designer is to ensure there is sufficient freeboard to finished floor levels from this maximum ponding depth.

For small sites (< 1000m2), the modified rational method can be employed to calculate:

- Permissible site discharge (PSD) based on pre-developed site characteristics.

- Post-developed site discharge.

- Site storage requirements based on design storm requirements and critical storm duration

For larger sites, (>1000m2), onsite storage requirements shall be calculated by a practicing engineer, using applicable methods. For all cases drawings and calculations must be prepared by a practicing and practicing Civil Engineer to the satisfaction of the City of Armadale and submitted for approval prior to construction.

## Re-use of stormwater

Rainwater tanks can have the potential to control peak flows m a similar manner to other on-site detention methods; however there is an added benefit to providing an alternative water source for household use. By incorporating the re-use of stormwater in the design it is possible to:

- Reduce flood risk;

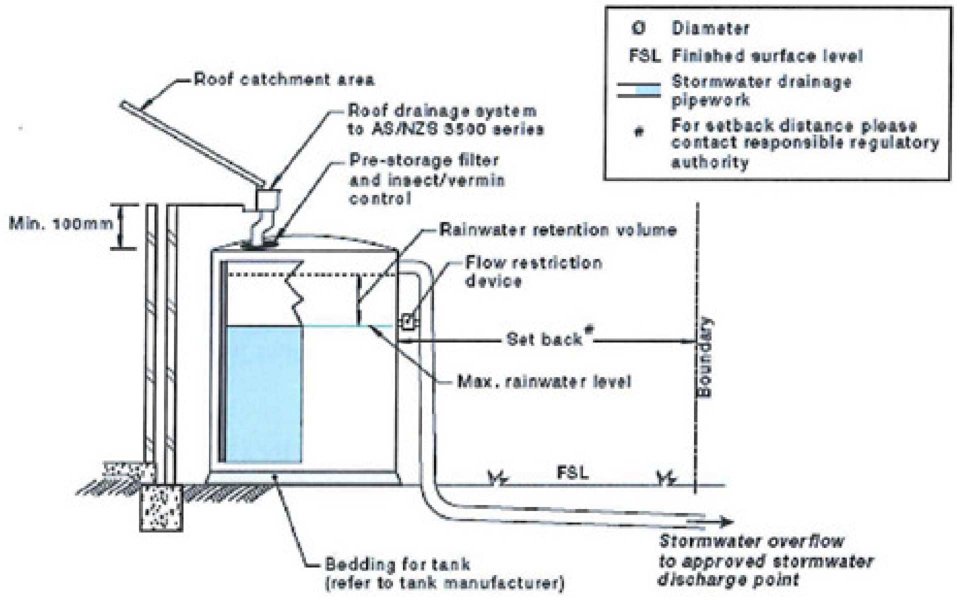
- Prevent erosion of waterways, slopes and banks;

- Improve water quality in streams and groundwater;

- Protect ecosystems and habitats, and
- Protect the scenic, landscape and recreational values of streams.

The City supports the re-use of stormwater via rainwater tanks with an appropriate overflow, see Figure 8.

Figure 8 – Rainwater Tank Detail




### Design criteria for connection to Council's system

Discharge to the City's system is to be via a silt trap located within the property and must comply with the following;

- Connection to the City's system is to be via a 90mm Stormwater Grade PVC pipe to a manhole located in the verge fronting the lot.
- Should a suitably located manhole not be available in the verge fronting the lot, the existing City of Armadale drainage system shall be extended and a manhole installed at the cost of applicant. Refer to section 12.3.1.3 for detailed requirements.
- Where no piped drainage system exists the City's Subdivision Engineer may allow discharge to the street via a pipe through the kerb.

# 11. SPECIFIC DEVELOPMENT REQUIREMENTS

## Residential Developments (R code)

Residential lots shall provide soakwells within the property in accordance with the following;

Table 17 – Minimum Storage Requirements for Small Residential Sites (<1000$m^2$) with Pervious Soils

| *Total Land Area ($m^2$)* | 300 | 400 | 500 | 600 | 700 | 800 | 900 | 1000 |
|---|---|---|---|---|---|---|---|---|
| *Required Storage Volume ($m^3$)* | 0.8 | 1.1 | 1.3 | 1.6 | 1.8 | 2.1 | 2.4 | 2.6 |

Table 18 – Minimum Storage Requirements for Small Residential Sites (<1000$m^2$) with Impervious Soils

| *Total Land Area ($m^2$)* | 300 | 400 | 500 | 600 | 700 | 800 | 900 | 1000 |
|---|---|---|---|---|---|---|---|---|
| *Required Storage Volume ($m^3$)* | 1.5 | 2.0 | 2.5 | 3.0 | 3.5 | 4.0 | 4.5 | 5.0 |

## Higher Density Developments

High density developments such as grouped housing sites tend to have higher impervious fraction ratios. Runoff from these developments needs to be managed in a way not to cause problems downstream whilst not producing issues for the site.

Suitable stormwater management for infill developments is dependent on the groundwater levels and the soil characteristics of the specific site. Determining the soil characteristics and the soil capacity to provide infiltration is necessary to define the most appropriate method of stormwater management for the site.

Any existing dwellings/structures that are retained as part of the subdivision/strata development must be included in the calculations and the design of the stormwater management system.

## Strata Subdivisions

Stormwater management of strata subdivisions require design by a competent and qualified engineer in accordance with the following:

- Stormwater discharge from the site must be limited to pre-development flows for all events up to and including the 100 year ARI.
- The capacity of the receiving system must be checked to ensure discharge does not contribute to downstream flooding.
- Water sensitive urban design principles shall be employed.

- Commercial/industrial – nutrient/hydrocarbon removal. Treatment devices are to be sized to treat the 3 months ARI event.

## 12. GREENFIELD DEVELOPMENTS

### District Water Management Strategy Requirements

The City assesses District Water Management Strategies (DWMS) which support the following:

- Regional or District Structure Planning
- Rezoning under a Regional Scheme
- Rezoning of Urban Deferred

Generally, state or local authorities are tasked with preparing a DWMS, however on occasion a private entity may wish to prepare a DWMS to support a rezoning or structure plan. In this case, prior to conducting an assessment the City requires a letter of support from the Department of Water to carry out a DWMS. This should include an outline of the required scope of the DWMS, including:

- The required DWMS study area.
- Detailed requirements and responsibilities for pre-development monitoring.
- Regional water quality targets.
- All relevant water management documents that are required to be referenced.

DWMS assessed by the City shall be compliant with the *Better Urban Water Management (BUWM), 2008* and the following City of Armadale Requirements:

- Acid Sulphate Assessment – Identification of potential risks to infrastructure through potential or real acid sulphate soils. This should include an assessment of the constraints and requirements for future infrastructure.
- In areas of known or suspected high groundwater tables, presentation of district level information (physical and chemical) if available. If no information is available, or current level of information is unable to provide the level of information required, then monitoring will be required to be undertaken.
- Identification & assessment of the potential impacts on surface and groundwater systems due to the proposed land use change. This should consider potential changes (physical and chemical) to groundwater conditions, surface water bodies, and drainage conditions of the study area (changes to how the land is drained), changes to upstream environments and changes to downstream environments. This should be carried out on a scale that is conducive to make

recommendations for water management strategies and to inform an implementation framework for future water management strategies.

- Presentation of district scale water balance modelling or other informative modelling should be presented.
- Identification and assessment of potential impacts of predicted climate change on water management.
- Identification of stormwater quality improvement methodologies available for adoption for the study area. Discuss and assess the success or failure of water quality improvement devices adopted in similar locations or nearby locations.
- Identification of potential point source of diffuse source that currently, or after development may require management.

When submitting a DWMS to the City for review, the *City of Armadale DWMS Checklist* shall also be submitted.

## 13. LOCAL WATER MANAGEMENT STRATEGIES REQUIREMENTS

The City assesses Local Water Management Strategies (LWMS) which support the following:

- Local Structure Plans
- Local Planning Scheme Amendments

For most developments, a LWMS will be preceded by Regional and/or District water management strategies. In these cases, the LWMS will be form a contiguous series of water management strategies, that a refined with each level.

In instances of developments were little or no water management strategies have been developed, then it is essential that defined principles and objectives are agreed to by the City prior to commencing the LWMS. These principles and objectives shall also have support from the Department of Water.

Local Water Management Strategies (LWMS) assessed by the City shall be compliant with the *Better Urban Water Management (2008)*, and the following City of Armadale requirements;

### Landscaping

The nature of current developments is such that public open space (POS) and drainage feature are co-located, or one in the same. As noted in *Liveable Neighbourhoods (WAPC, 2009),* the area required for drainage and water sensitive design will largely depend on terrain, soil type and climate. Given the intimate relationship between POS and Water Sensitive Urban Design (WSUD), the following items are required to be submitted and an inclusion of the LWMS;

- Identified POS areas, broken down into;
  - POS not burdened by a WSUD or drainage function
  - POS burdened by a major storm control function
  - POS burdened by a minor storm control function
  - POS burdened by a WSUD (water quality function)

- Details of any constructed water bodies predominantly used as water features.

- POS irrigation requirements and bore locations and applicable current water extraction licences. A 'fit for purpose' assessment for all POS irrigation water sources shall be carried out with regard to the suitability of the water source for irrigation, impacts on the irrigative systems, and any impacts on surface water quality.

### Acid Sulphate Soils

Actual acid sulphate soils (AASS) or potential acid sulphate soils (PASS) have the potential to cause short term and long term damage to both natural assets and councils infrastructure assets proper identification of AASS or PASS is critical at the LWMS to properly assess the impact on the development, and will inform the urban design process. The following cases will require the LWMS to address and further investigate AASS and PASS.

- Any instance the Department of Environment require an Acid Sulphate Management Plan,

- Any site that is identified as having a moderate to high ASS risk, and

- Any site that requires installation of council infrastructure that potentially will be below or at groundwater level for any period of time.

All test results shall be submitted as an annex, and results reported in the LWMS testing shall be carried out by a NATA accredited lab.

A risk based assessment of the ASS or PASS shall be carried out if it is determined that there is potential for construction activities or urbanisation of land to mobilise acids.

### Pre and Post Development Water Balance Modelling

Asses the risk of bulk changes to the pre-developed water cycle, and assessment of potential impacts on;

- Water dependant ecosystems

- Groundwater levels

- Flooding both upstream and downstream and within the study area

- Existing Council infrastructure

Water balance and hydrological modelling shall be used to determine the impacts of the developments. The LWMS must include a summary of the modelling methodology used, with presentation of critical parameters and assumptions. Result summary sheet shall be included as an appendix.

## 14. URBAN WATER MANAGEMENT PLAN REQUIREMENTS

### Lot Scale Management

Overview of the Lot level stormwater management required to be adopted by builders and owners. This should be a short, simple description of stormwater requirements, with a method of discriminating lot requirements. This information will be given to future land holders, builders and will be placed on the City's website for easy reference for residents.

Where lots have frontages either equal to or less that 11m in width or where the total lot area is equal to or less that 350m², then lot connection pits to the City's stormwater drainage are to be provided. Re-use of stormwater is encouraged.

### Street Scale Stormwater Systems

- The street scale stormwater system shall be demonstrated at UWMP stage to be able to comply with this guideline.

- The Urban Water Management Plan (UWMP) shall include an outline of urban drainage methodology and requirements of future detailed urban drainage modelling. This should include a description on how the adopted stormwater management strategies will impact on the design of the stormwater system at subdivision stage. For example, the following design parameters shall be summarised for subdivision stormwater detailed design;
    - Minor system requirements to be adopted
    - Major system requirements to be adopted
    - Detention basin design parameters
    - Catchment scale modelling parameters used

- All detention storage areas shall undergo detailed engineering design support with calculations. This enables future subdivision works and public open space works to be carried out with greater certainty.

### Water Quality

All water quality structures need to undergo detailed design with proper assessment of site conditions and constraints, in addition to expected and modelled hydrologic conditions.

### Post Development Monitoring

Post development monitoring shall be sufficient to assess the adequacy of the following:

- Water quality improvement device performance & condition
- Adequacy of routine maintenance for water quality improvement devices
- Post Developed water level measurements. The scope and time length of monitoring will be determined by the City, and will be dependent on the risk of flooding due to increases of groundwater via hydrological changes to the catchment.
- Adequacy of the trunk and major drainage system

### Stormwater drainage management

Stormwater management for Greenfield sites shall be in accordance with the DOW Stormwater Management Manual and address the following:

- Re-use of stormwater
- Treatment of the 1 year ARI event
- Sizing of pipe systems to accommodate the 5 year ARI event
- Provision of storage infrastructure sized to attenuate the 10 year ARI event to pre-development flow rates to protect the downstream drainage system
- Overland flood route for the 100 year ARI event
- Specific requirements of downstream receiving waters; and
- Existing upstream catchment.

## 15. APPENDIX A – DETENTION BASIN RISK MANAGEMENT

# DETENTION BASIN
# RISK MITIGATION ASSESSMENT

## 1 DEFINITIONS

*Consequence*
The outcome of an event expressed qualitatively or quantitatively, being a loss, injury, disadvantage, or gain. There may be a range of possibility outcomes associated with an event. (AS/NZS 4360:1999)

*Hazard*
A source of potential harm or situation with a potential to cause loss. (AS/NZS 4360:1999)

*Risk*
The chance of something happening that will have an impact upon objectives. It is measured in terms of consequences and likelihood. (AS/NZS 4360:1999)

*Risk Assessment Rating*
The rating given to the consequence of the hazard occurring.

*Risk Mitigating Treatment*
Process or actions taken to reduce the Risk or the Severity of the Hazard.

*Severity (of the hazard)*
How bad or the acuteness of the consequence.

## 2 INTRODUCTION

The City of Armadale (CoA) currently many detention / sedimentation basins / wetlands that were constructed as part of the development process for newly created subdivisions or improvements to existing drainage networks. These basins reduce the intensity of stormwater flows into our drainage systems, improve water quality through the process of trapping sediments, and aid in minimising the change in stormwater flow from a natural to a developed state.

For the drainage network to effectively service the community, drainage infrastructures such as detention basins are located in or near residential areas. Not only do these basins add open space, aesthetic appeal, and are an area for natural habitat to reside, basins in the proximity of residential areas increase the potential risk exposure to Council. Community safety concerns with detention basins predominately relate to the steepness of the batter slope, depth of water in the basin (temporary and permanent), and the proximity of the basin to a populated area.

This guideline shall ensure that a consistent approach is adopted when evaluating existing and new detention basins for hazards and the associated risk. The evaluation shall be used to implement risk mitigating measures to reduce the risk of injury to the public, and hence reduce Council's exposure to the possibility of a claim should an injury occur.

This guideline shall adopt differing approaches to separate existing and new basins. New basins shall mitigate risks at the design stage, with existing basins undergoing treatments to reduce current risks. Hence a program needs to be implemented which is funded appropriately to overcome any deficiencies in safety of existing basins.

# 3 ASSESSMENT

## 3.1 Aim

The aim is to eliminate or reduce the risk of injury to the public should persons accidentally or deliberately enter the basin, and consequently reduce Council's exposure to the possibility of a claim should an injury occur.

## 3.2 Objectives

- Provide minimum risk for persons assessing the area whether access gained was deliberate or accidental
- identify hazards and their severity by conducting a consistent formal assessment of all basins
- Determine risk and an appropriate risk mitigating treatment

# 4 EVALUATION

Detention basins are evaluated for their Hazard, Severity (of the hazard), and the associated Risk. Physical and Environmental characteristics of the basin are used m evaluating the hazards and risks respectively. The evaluation of these characteristics is used to produce a Risk Mitigating Treatment.

Basins with varying natural/ constructed surrounding topography may have a number of components with differing hazards and associated risks. Therefore several Risk Mitigating Treatments for one basin may be adopted. For example, a basin with differing batter slopes with a walkway on only one side of the basin.

## 4.1 Basin Properties for Evaluation

Australian Rainfall and Runoff (ARR 1987) has been used to determine what type of physical conditions requires mitigation treatments to reduce Council's exposure to risk. Clause 14 10.4 of ARR notes that:

- Slopes 6H 1V or flatter are recommended for grassed lined channels
- Rails or fences should be provided at transitions and other locations where steep slopes occur such as slopes greater than 4H 1V
- Maximum height of fence or rails to be 1 0 to 1 2m and should not impede potential rescuers
- Grassed lined channels are preferable to hard lined channels
- Warning signs and depth gauge boards are recommended for installation
- Maximum preferable depth at 20yr storm is 1 2m
- Trees and mounds within basins are desirable as refuges

Based on Australian Rainfall and Runoff (ARR 1987) this risk mitigation assessment shall examine the following physical characteristics to be determined as a hazard to the public

- Batter slope
- Height to drop from the top of batter to toe of batter
- Depth of water
- Proximity and level of public use
- Whether a basin is wet or dry

To determine a risk mitigating treatment for the corresponding hazard the environmental property of level of public use shall be examined.

# 5 PROCESS OF EVALUATION- NEW DETENTION BASINS

All new detention basins shall have a batter slope of 6H:1V or flatter. This includes the batter slope below the top water level. A batter slope of 6H:1V will result in no Risk Mitigating Treatments. This batter slope shall lower the presence of hazards and minimise future maintenance costs associated with steeper batter slopes.

Should this physical constraint not be possible to obtain, the risk assessment for exiting detention basins may be used.

## 5.1 Process of Evaluation – Existing Detention Basins

The following steps are employed to determine the Hazard, Severity (of the hazard), Risk (of the hazard), and recommended Risk Mitigation Treatment for each hazard component of the basin. This detention basin risk mitigation assessment is based on the City of Armadale's Risk Treatment Plan-Risk Assessment table as below.

**Table 3.1 - Section from the CoA Risk Treatment Plan - RISK ASSESSMENT TABLE**
**(Non-relevant items to this assessment ruled out)**

| RISK ASSESSMENT TABLE | | | | |
|---|---|---|---|---|
| HOW BAD / HOW LIKELY | VERY LIKELY Could happen at any time | LIKELY Could happen sometime | UNLIKELY Could happen, but rarely | VERY UNLIKELY Could happen, but probably never will |
| Kill or cause permanent disability | A | A | B | B |
| Long term illness | A | B | B | C |
| Injury requiring medical attention | B | B | C | C |
| Minor injury requiring First aid | B | C | C | D |
| PRIORITY ACTION | A Immediate action – Stop/prevent activity<br>B Initiate treatment immediately<br>C Treatment initiated as soon as practicable | | D To be done by nominate/agreed date<br>OK Acceptable | |

For usual work practice, scenario assessments the CoA Risk Assessment table is used for each Hazard. For each Hazard the Severity and Risk (labelled - How Bad and How Likely respectively) is considered to determine the Risk Assessment Rating. This rating is shaded on the table and noted as A, B, C, or D. This Risk Assessment Rating then determines the recommended Risk Mitigating Treatment.

The Risk Mitigating Treatments from the CoA Risk Treatment Plan – Treatment Hierarchy table as below denotes a graded set of objectives for generic treatments.

**Table 3.2 - Section from the CoA Risk Treatment Plan - TREATMENT HIERARCHY TABLE**

The treatment hierarchy provides a graded set of objectives for treatments Best solution number one (1) to last resort number six (6)

**1. ELIMINATE**
Remove risk from the process by eliminating the step in the process – i e do not do it
**2. SUBSTITUTE**
Reduce risk by changing process, materials or equipment to something that does the job more safely – i e change from a hazardous substance to a non-hazardous substance
**3. ISOLATE**
Put in place physical preventive mechanisms – i e signs, alarms, lights, ventilation, guards & barriers
**4. ENGINEER**
Minimise the risk by engineering means – i e use a mechanical lifting device rather than manual handling techniques
**5. ADMINISTRATE**
Develop & implement safe work procedures – i e Work Method Statements, training, direction, and supervision
**6. PPE**
Accept the initial hazards and protect personnel by using personal protective equipment to reduce the risk – i e Safety glasses etc

Adopting the steps taken from the City's Risk Treatment Plan, this detention basin risk mitigation assessment is broken into the assessment segments Hazard, Severity, Risk, and Risk Mitigating Treatment.

## 5.2 Hazard

The hazards included for analysis such as batter slope, height of drop, and depth of water, when placed in combination result in differing severities. Hence, combinations of these hazards shall be examined to gain a Severity Rating.

*Determine if the basin is dry or permanently wet*

A basin is considered wet if the permanent water depth is greater than 300mm from the base to the invert of the outlet. Dry basin only carries a flow of water during a storm event and dissipates within a short period of time such as a day

*Measure batter slope*

For the purpose of evaluation five batter slopes have been adopted. Interpolations are not permitted, hence adopt next worse case when measured values are not represented by these five options.

- Vertical Face
- 1H 1V batter
- Stepped
- 3H 1V batter
- 6H 1V batter or flatter

*Measure the depth*

The depth of batter measurement shall be dependent on the wet or dry status of the basin and the batter slope. The water is considered more of a hazard than the batter slope for basins with a batter slope less than 3H:1V.

**Table 3.3** – Depth Calculation

| | | |
|---|---|---|
| Depth for Dry Basin (Diagram 3.1) | = | depth from surrounding area to the base of the basin |
| Depth for Wet Basin greater than 3H:1V batter (Diagram 3.2) | = | depth from surrounding area to the base of the basin |
| Depth for Wet Basin lesser than 3H:1V batter (Diagram 3.3) | = | depth of water from top of water to base of basin |

**Diagram 3.1** - Dry Basin, all batter slopes

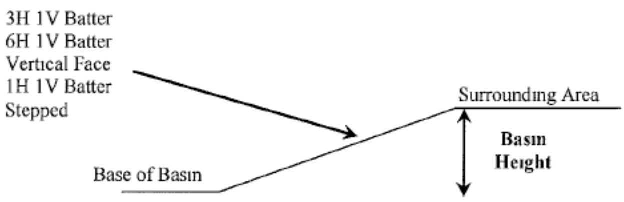


**Diagram 3.2** - Wet Basin, batter slopes steeper than 3H:1V

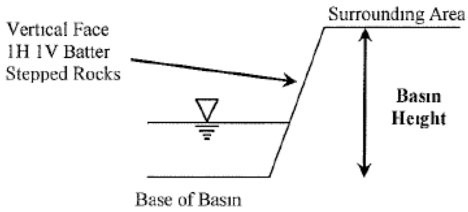
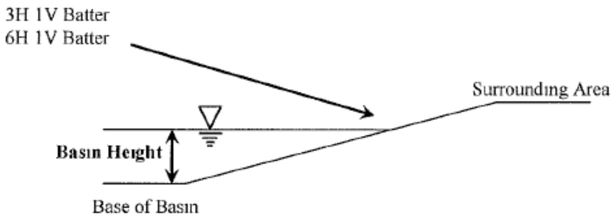



**Diagram 3.3** - Wet Basin, batter slopes flatter than 3H:1V




## 5.2 Severity

*Determine Severity Rating*

The Severity Rating is the acuteness of an incident as a result of the occurrence of the hazard. The Severity Rating is determined in Table 4.1 and is defined in Table 4.2. It should be noted that an additional rating, "Insignificant" has been added to the City's Risk Treatment Hierarchy to account for the range of possible outcomes as a result of a range of physical properties that a Hazard may have.

**Table 4.1** – Severity Rating

| | | Batter Slope (Dry / *Wet*) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Vertical Face | | Stepped | | 1H:1V | | 3H:1V | | 6H:1V | |
| Height (m) | >1.2 | 2 | *1* | 2 | *5* | 3 | *1* | 5 | *5* | 5 | *5* |
| | 1.2 | 3 | *1* | 2 | *5* | 3 | *1* | 5 | *5* | 5 | *5* |
| | 0.9 | 3 | *1* | 3 | *5* | 4 | *1* | 5 | *5* | 5 | *5* |
| | 0.6 | 4 | *4* | 4 | *5* | 4 | *5* | 5 | *5* | 5 | *5* |
| | ≤ 0.3 | 5 | *5* | 5 | *5* | 5 | *5* | 5 | *5* | 5 | *5* |

**Table 4.2** – Severity Rating Definitions

| Rating | Descriptor | Description |
|---|---|---|
| 1 | Catastrophic | Could cause death |
| 2 | Major | Could cause extensive injuries requiring hospitalisation |
| 3 | Moderate | Could cause injuries requiring medical treatment |
| 4 | Minor | Could cause minor injuries |
| 5 | Insignificant | No injuries likely |

## 5.4 Risk

*Determine public use*

The Risk (or probability) of an event occurring has been assumed to be proportional to the number of people frequenting the hazard. The greater the number of persons visiting the area should increase the probability of the number of persons entering the detention basin.

**Table 5.1** – Risk Level Description

| Public Use Level | Frequency | Persons per day |
|---|---|---|
| A | Most frequent | >100 |
| B | Frequent | 20 to 100 |
| C | Less frequent | 5 to 20 |
| D | Rare | < 5 |

*Specific type of public use.*

The Risk (or probability) of an event occurring has also been assumed to be proportional to the number of young person's frequenting the basin. Where the frequency of Public Use Level contains a high percentage of young persons. Medium Fencing shall be adopted for areas of the basin that have gained a Severity Rating of at least 4 and are directly adjacent to the general trafficable path.

# 6 RISK MITIGATING TREATMENT

## 6.1 Select a Risk Mitigating Treatment.

The Risk Mitigating Treatment is established from the calculated Severity Rating and the likelihood.

**Table 6.1** – Risk Mitigating Treatment

| | | Public Use Level | | | |
|---|---|---|---|---|---|
| | | **A** (> 100) | **B** (20 – 100) | **C** (5 - 20) | **D** (<5) |
| Consequence Rating | 1 Catastrophic | Exclusion Fencing | Exclusion Fencing | Exclusion Fencing | Exclusion Fencing |
| | 2 Major | Exclusion Fencing | Exclusion Fencing | Exclusion Fencing | Exclusion Fencing |
| | 3 Moderate | Exclusion Fencing | Medium Fencing | Vegetation | Vegetation |
| | 4 Minor | Nothing Required | Nothing Required | Nothing Required | Nothing Required |
| | 5 Insignificant | Nothing Required | Nothing Required | Nothing Required | Nothing Required |

**Note:**
- Any vertical face over 1m shall have at a minimum a barrier railing as indicated below
- Refer to Specific Type of Public Use section under assessment segment **Risk** in Section 3 of this document

## 6.2 Risk Mitigating Treatments

Design of the risk mitigating treatments should incorporate required access for the purpose of basin maintenance. Treatments should also not impede potential rescuers.

*Exclusion Fencing*

- Galvanised and powder coated steel or powder coated aluminium pool fencing
- 1.2m in height
- Colour in black, oyster grey or light blue to match existing City of Armadale assets
- Or approved custom made public art fencing

*Medium Fencing- Barrier Railing/Dense Vegetation*

- Standard cable fencing design with 100mm NB galvanised steel posts (powder coated)
- Galvanised ball-tube hand rail
- Or approved custom made public art fencing

OR

- Vegetation 4m wide planted at maximum 0.5m centres
- 0.75 minimum height

*Vegetation*

- Vegetation placed to denote a physical boundary line at least 1m wide

*Nothing Required*

- Grassed area

### 6.3 Ancillary Items

*Inlet and Outlets*

To avoid entry in pipes and pits:

- Outlet pipes – 300mm shall be covered with either a grate or prevention bar
- Pits shall be covered with a grate

The grate or prevention bar should be designed to withstand the force exerted by the flow of stormwater. Care should be taken not to partially protect the drainage network so that small persons or children are able to enter into one part of the drainage system and not exit from another.

*Signage*

Each detention basin shall have a minimum of one sign to notify the public of the dangers associated of entering the basin and if required, the purpose of the basin. The sign shall include the symbols such as:

- No swimming
- No diving
- Do not enter

Each detention basin shall also have depth indicators indicating maximum depth in the basin.

*Refuge Mounds*

Trees and mounds within the basin are allowed for the purpose of refuge.

*Vegetation*

No planting of trees shall be allowed m the basin wall where the wall is an acting levee bank.

## detention basin risk mitigation proforma

# DETENTION BASIN RISK MITIGATION ASSESSMENT

**WAPC Number**........................ **Subdivision Name**........................................................ **Stage** ....................................

**Name of Assessing Engineer** ........................................................ **Consultancy Name**............................................... **Date** ..............................

| | Component Description | Step 1<br>Wet / Dry | Step 2<br>Batter Slope | Step 3<br>Depth for Basin | Step 4<br>Severity Rating | Step 5<br>Public Use (Risk Level) | Step 6<br>Type of Public Use | Step 7<br>Risk Mitigating Treatment | Comments |
|---|---|---|---|---|---|---|---|---|---|
| 1 | | | | | | | | | |
| 2 | | | | | | | | | |
| 3 | | | | | | | | | |
| 4 | | | | | | | | | |
| 5 | | | | | | | | | |
| 6 | | | | | | | | | |
| 7 | | | | | | | | | |
| 8 | | | | | | | | | |
| 9 | | | | | | | | | |
| 10 | | | | | | | | | |
| 11 | | | | | | | | | |
| 12 | | | | | | | | | |

***(Tick box where applicable)***

Outlet pipes (greater than 300mm ☐ / less than 300mm ☐) Refuge Mound Present ☐ Signage Present ☐

Other Comments

.......................................... . ..................................... . . . ................. .............. . ............. .................................

# Pressure Head Change Coefficient Data and Water Surface Elevation Coefficient Data for Sumps and Manholes

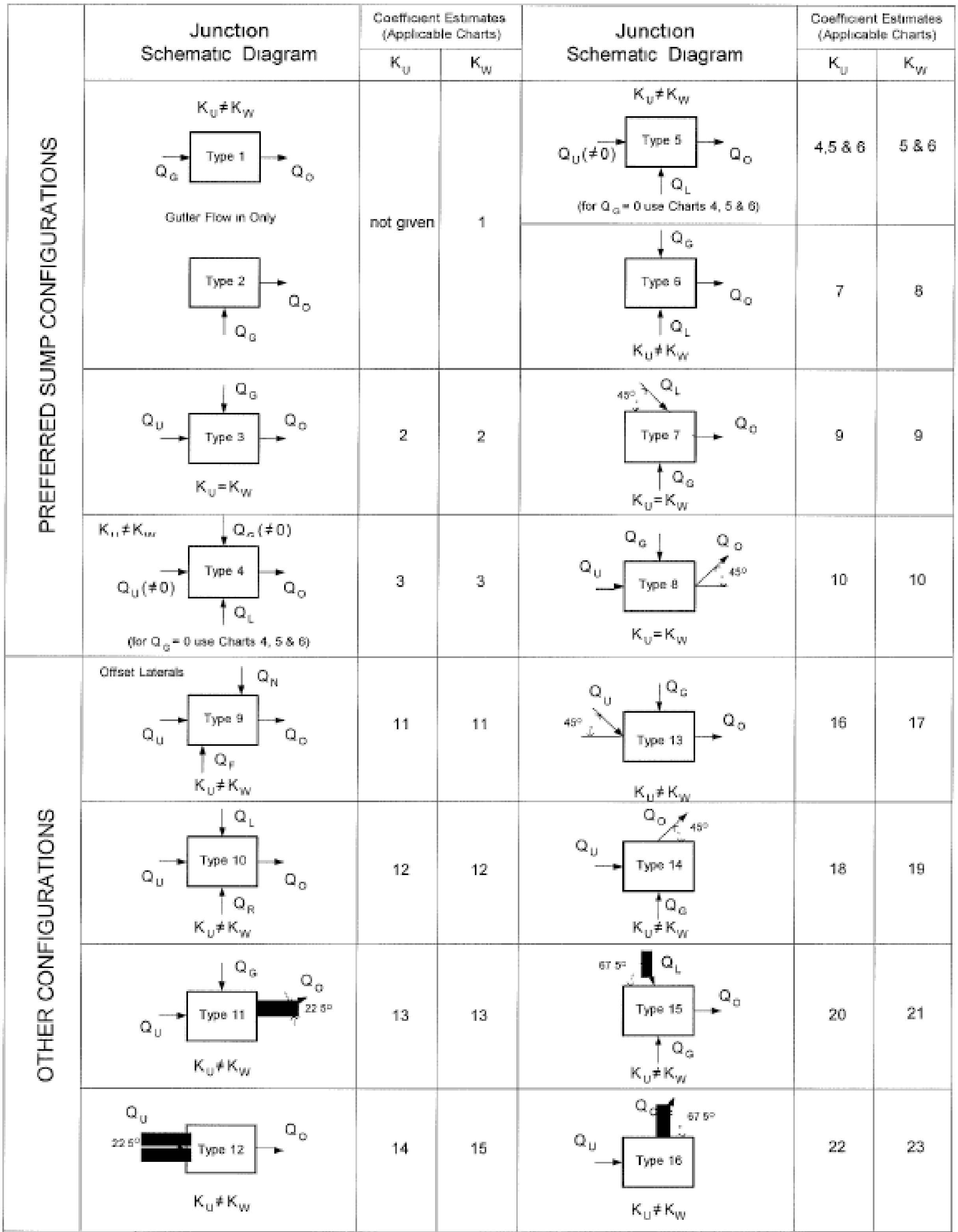
## S o u r c e s


M Sangster W. M. et al. 'Pressure Changes at Storm Drain Junctions', Engineering Series, Bulletin No. 41, Engineering Experimental Station, University Of Missouri, 1958

Charts 1, 3 to 6, 11 and 12

N Hare C. M. 'Energy Losses in Pipe Systems', Advances in Urban Drainage Design, Insearch Ltd, NSW Institute of Technology, 1981

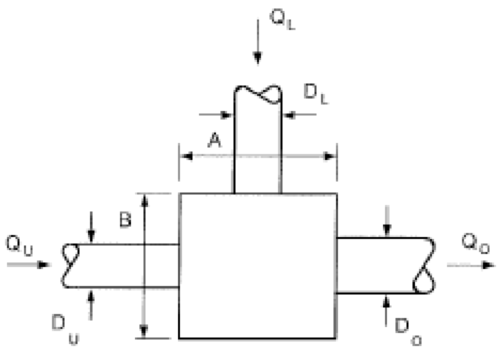
Charts 2, 7 to 10, 13 to 23




| | Junction Schematic Diagram | Coefficient Estimates (Applicable Charts) $K_U$ | $K_W$ | Junction Schematic Diagram | Coefficient Estimates (Applicable Charts) $K_U$ | $K_W$ |
|---|---|---|---|---|---|---|
| PREFERRED SUMP CONFIGURATIONS | Type 1, Type 2 ($K_U \neq K_W$) | not given | 1 | Type 5 ($K_U \neq K_W$) | 4,5 & 6 | 5 & 6 |
| | | | | Type 6 ($K_U \neq K_W$) | 7 | 8 |
| | Type 3 ($K_U = K_W$) | 2 | 2 | Type 7 ($K_U = K_W$) | 9 | 9 |
| | Type 4 ($K_U \neq K_W$) | 3 | 3 | Type 8 ($K_U = K_W$) | 10 | 10 |
| OTHER CONFIGURATIONS | Type 9 ($K_U \neq K_W$) | 11 | 11 | Type 13 ($K_U \neq K_W$) | 16 | 17 |
| | Type 10 ($K_U \neq K_W$) | 12 | 12 | Type 14 ($K_U \neq K_W$) | 18 | 19 |
| | Type 11 ($K_U \neq K_W$) | 13 | 13 | Type 15 ($K_U \neq K_W$) | 20 | 21 |
| | Type 12 ($K_U \neq K_W$) | 14 | 15 | Type 16 ($K_U \neq K_W$) | 22 | 23 |

Junction Types Covered by the Charts

FIGURE 1




Junction Dimensions

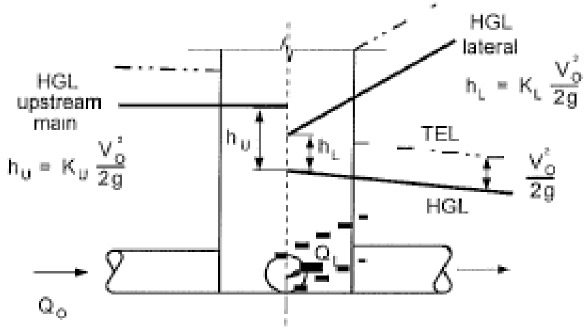



Diagram of HGL elevations at junctions of a lateral with a through main

| | |
|---|---|
| Q | rate of flow |
| D | diameter of pipe |
| A | dimension of junction in direction of outfall pipe |
| B | dimension of junction at right angles to outfall pipe |
| S | depth of water in sump |
| HGL | hydraulic grade line |
| TEL | total energy line |
| $Q_G$ | flow into sump through top inlet |
| $D_O$, $Q_O$ | dia and flow in outfall |
| $D_U$, $Q_U$ | dia and flow in upstream main |
| $D_L$, $Q_L$ | dia and flow in left lateral |
| $D_R$, $Q_R$ | dia and flow in right lateral |
| $D_N$, $Q_N$ | dia and flow in near lateral |
| $D_F$, $Q_F$ | dia and flow in far lateral |
| $D_{hv}$, $Q_{hv}$ | dia and flow in lateral with higher-velocity flow |
| $D_{lv}$, $Q_{lv}$ | dia and flow in lateral with lower-velocity flow |

($D_R$, $Q_R$; $D_N$, $Q_N$; $D_F$, $Q_F$:) of opposed laterals, designation with reference to position relative to outfall end of junction

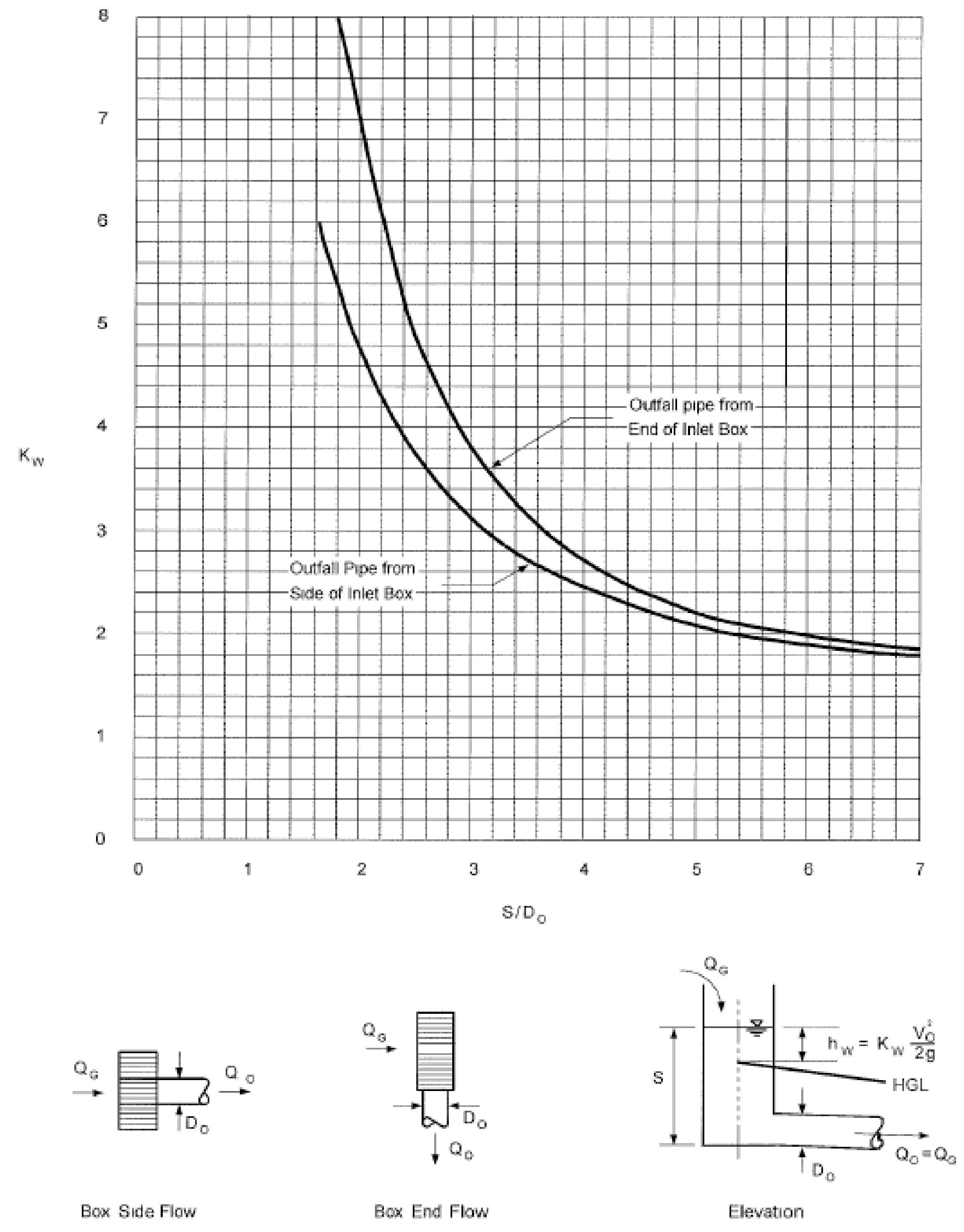
($D_{hv}$, $Q_{hv}$; $D_{lv}$, $Q_{lv}$:) for in-line opposed laterals only

Pressure change coefficients for inlet water depth and an upstream pipe pressure relative to the outfall pipe pressure

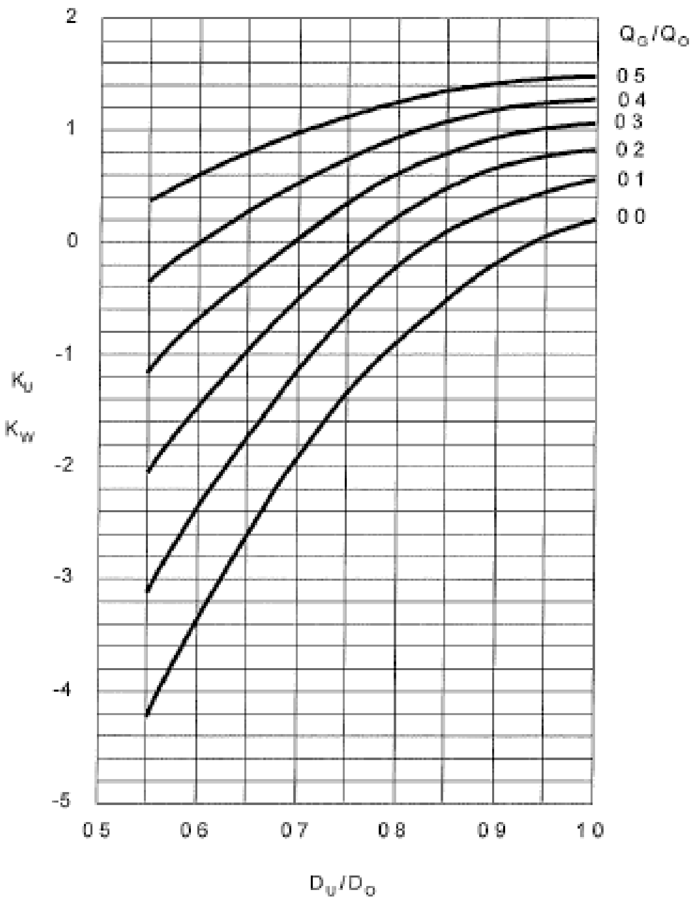
| | |
|---|---|
| $K_G$ | water depth with all flow through grate |
| $K_U$ | upstream main pressure |
| $K_R$ or $K_L$ | lateral pipe pressure |
| $K_N$ | near lateral pipe pressure (offset opposed laterals) |
| $\underline{K_F}$ | far lateral pipe pressure (offset opposed laterals) |
| $K_U$ , $K_L$ | pressure coefficient $\underline{at}$ $Q_L = Q_O$ |
| $M_U$ , $M_L$ | multipliers for $K_U$ or $K_L$ to obtain $K_U$ or $K_L$ |

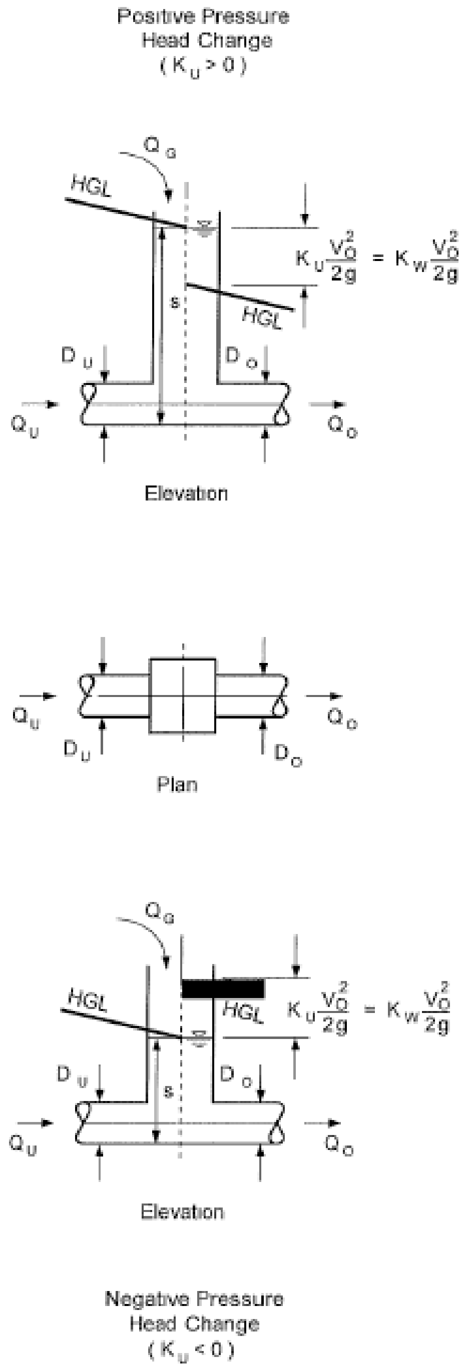
Nomenclature used in the Charts

FIGURE 2




| | | |
|---|---|---|
| Source M | Water surface elevation coefficient ($K_W$) for rectangular inlet with grate flow | CHART 1 |

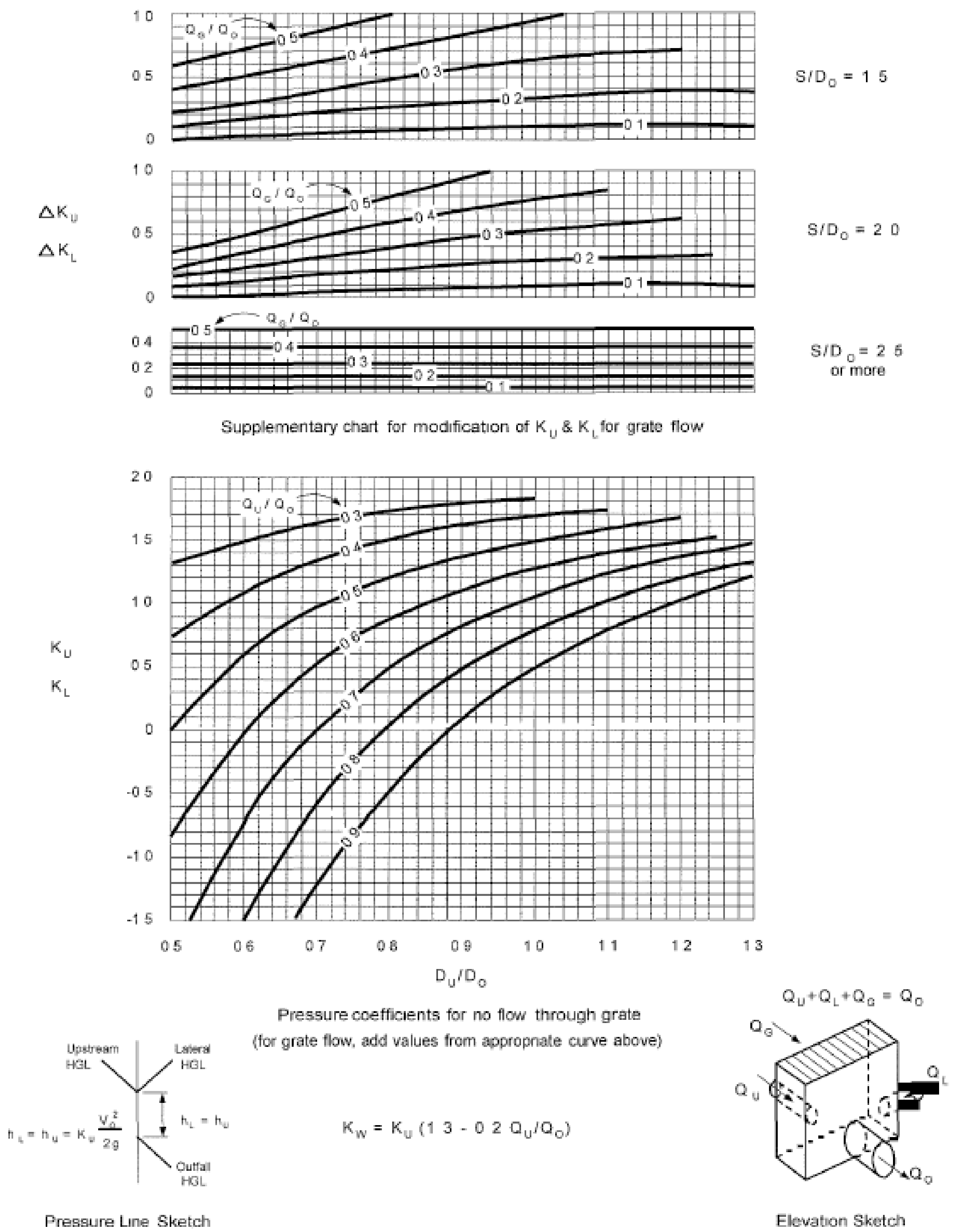


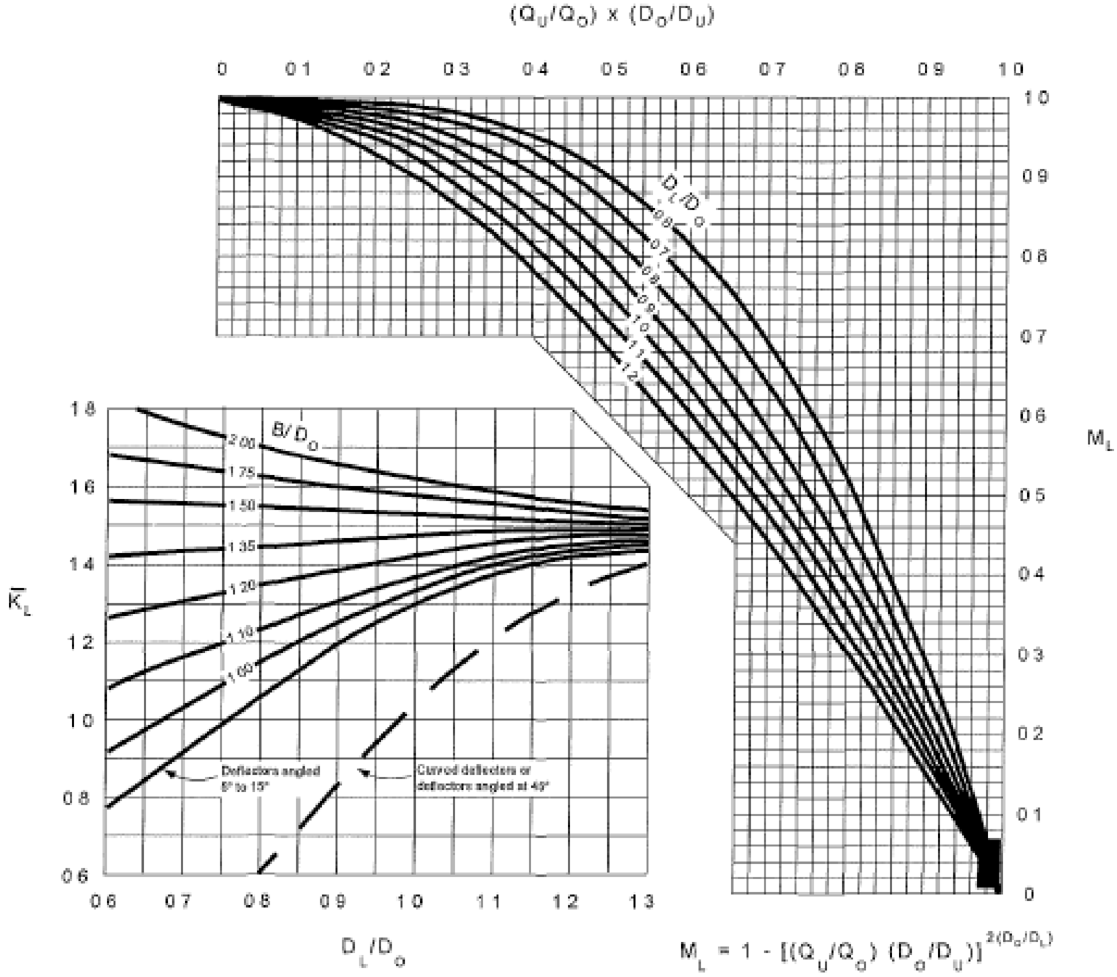

| $Q_G/Q_O$ \ $S/D_O$ | 0 00 | 0 10 | 0 20 | 0 30 | 0 40 | 0 50 |
|---|---|---|---|---|---|---|
| 1 5 | 0 00 | 0 11 | 0 22 | 0 33 | 0 44 | 0 55 |
| 2 0 | 0 00 | 0 04 | 0 08 | 0 12 | 0 16 | 0 20 |
| 2 5 | 0 00 | 0 00 | 0 00 | 0 00 | 0 00 | 0 00 |
| 3 0 | 0 00 | -0 03 | -0 06 | -0 09 | -0 12 | -0 15 |
| 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 |

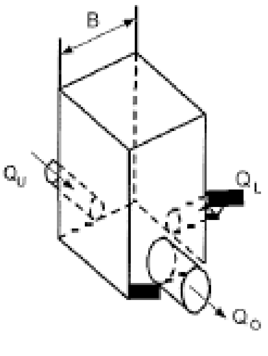
Increment $K_U$ and $K_W$ by the above values for $S/D_O$ ratios other than 2 5

| Source N | Coefficients for straight through flow | CHART 2 |
|---|---|---|




| | | |
|---|---|---|
| Source M | Pressure head change coefficient ($K_U$) for rectangular inlet with in-line upstream main and 90° lateral pipe (with or without grate flow) | CHART 3 |




Elevation Sketch

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

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

Dashed curve for curved or 45° angle deflectors applies only to manholes without upstream inline pipe

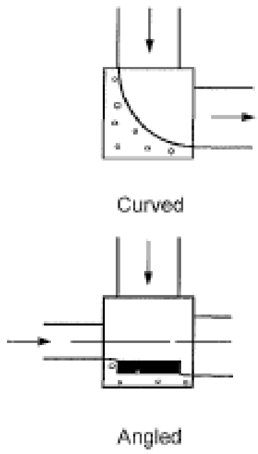
Use this chart for round manholes also

For rounded entrance to outfall pipe, reduce chart values pf $K_L$ by 0 2 for combining flow

For $(Q_U/Q_O) \times (D_O/D_U) > 1$ use Chart 6

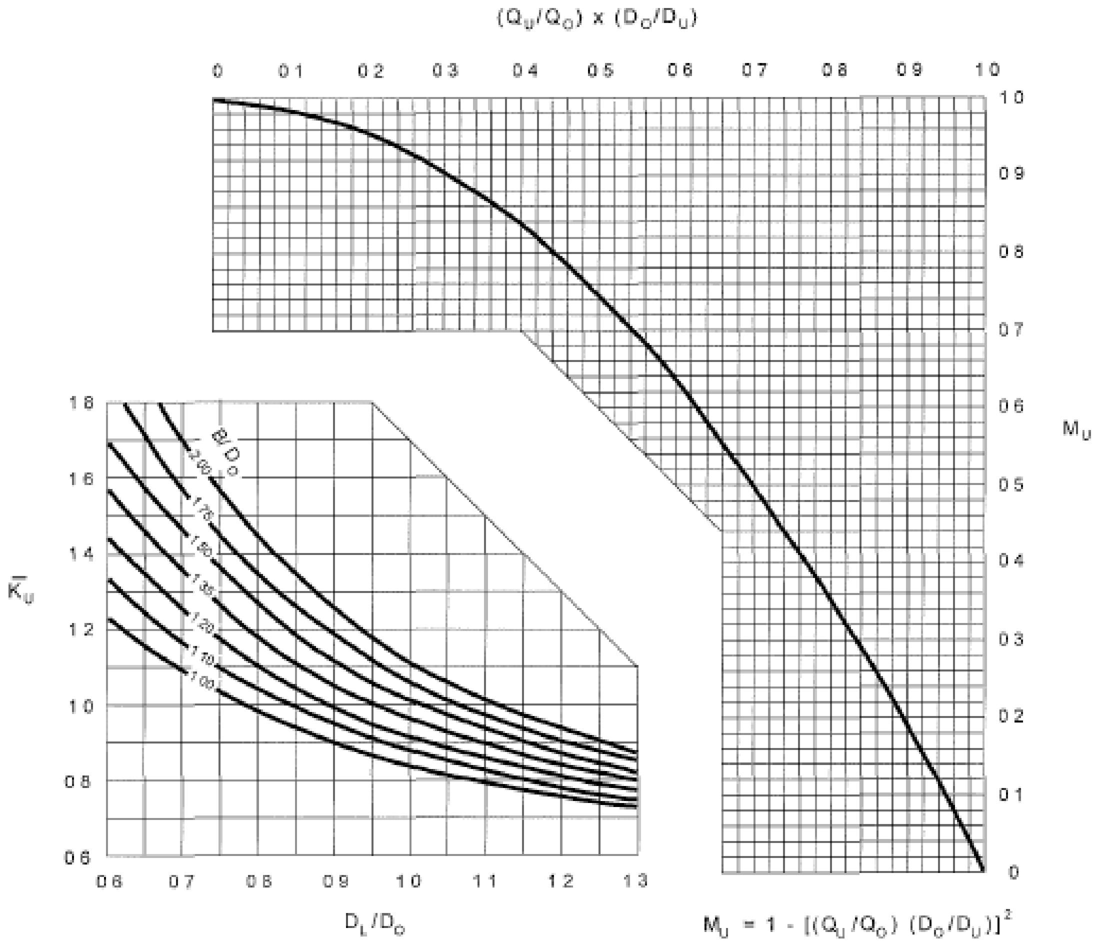
For $D_L/D_O < 0\ 6$ use Chart 6

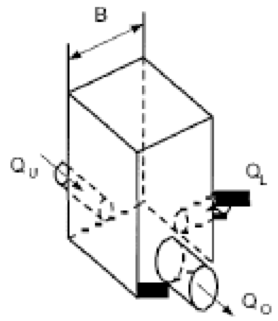
$$h_L = K_L \frac{V_O^2}{2g}$$




Plan of Deflectors

| | | |
|---|---|---|
| Source M | Pressure head change coefficient ($K_L$) for square or round manhole at 90° deflection or on through pipeline at junction of 90° lateral pipe (lateral coefficient) | CHART 4 |



Elevation Sketch

To find $K_U$ for the lateral pipe, first read $\overline{K}_U$ from the lower graph. Next determine $M_U$. Then,

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

$$K_W = K_U\,(1.3 - 0.2\,Q_U/Q_O)$$

For manholes with deflectors at 0o to 15o read KU on curve for B/DO = 1

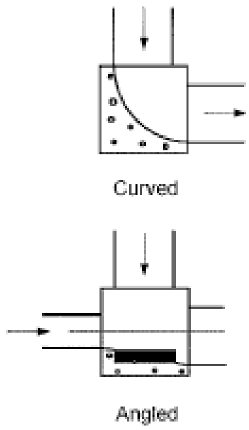
Use this chart for round manholes also

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

For $(Q_U/Q_O) \times (D_O/D_U) > 1$ use Chart 6

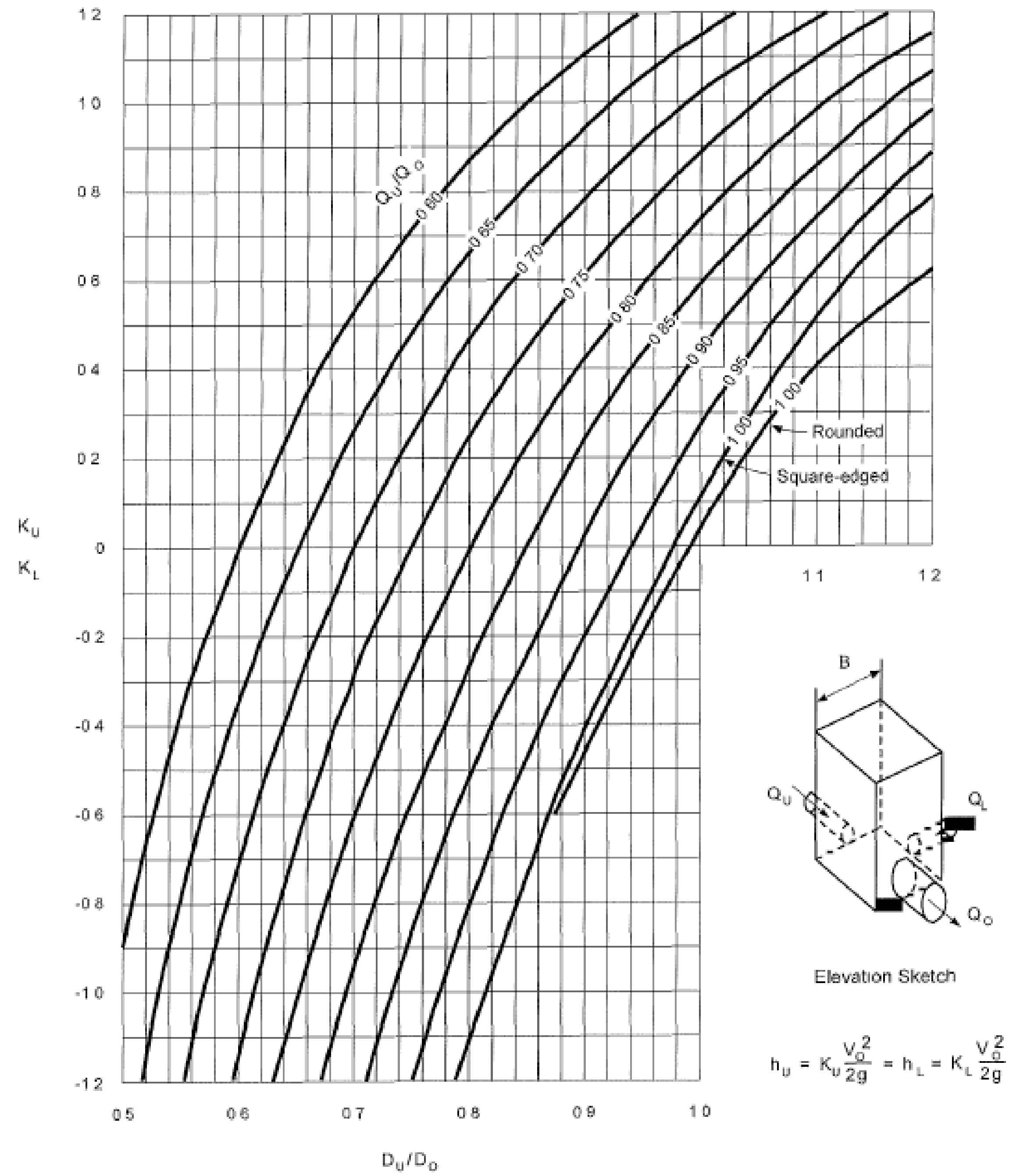
For $D_L/D_O < 0.6$ use Chart 6

$$h_U = K_U \frac{V_O^2}{2g}$$



Plan of Deflectors

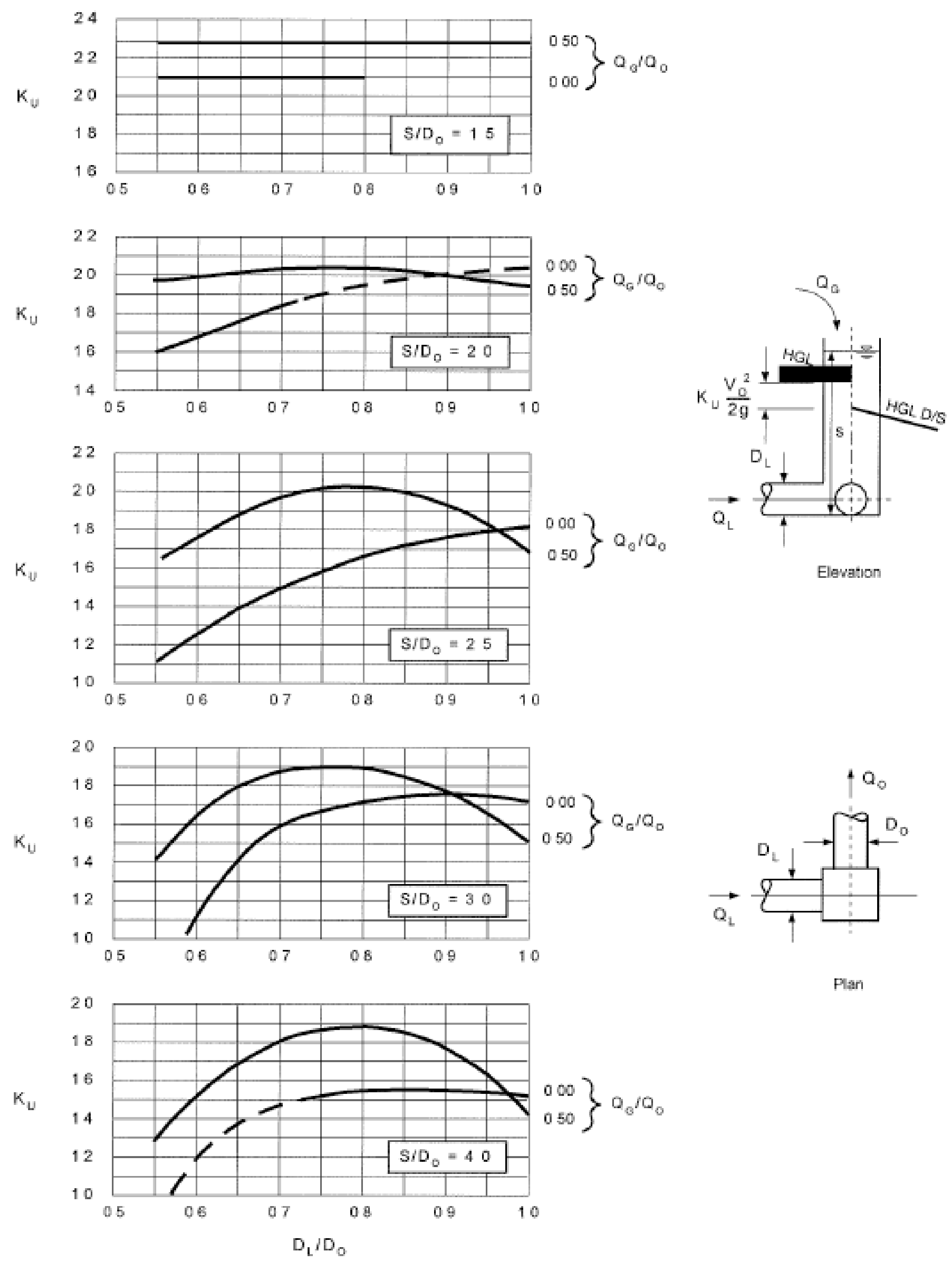
| | | |
|---|---|---|
| Source M | Pressure head change coefficient ($K_U$) for square or round manhole on through pipeline at junction of 90° lateral pipe (in-line pipe coefficient) | CHART 5 |



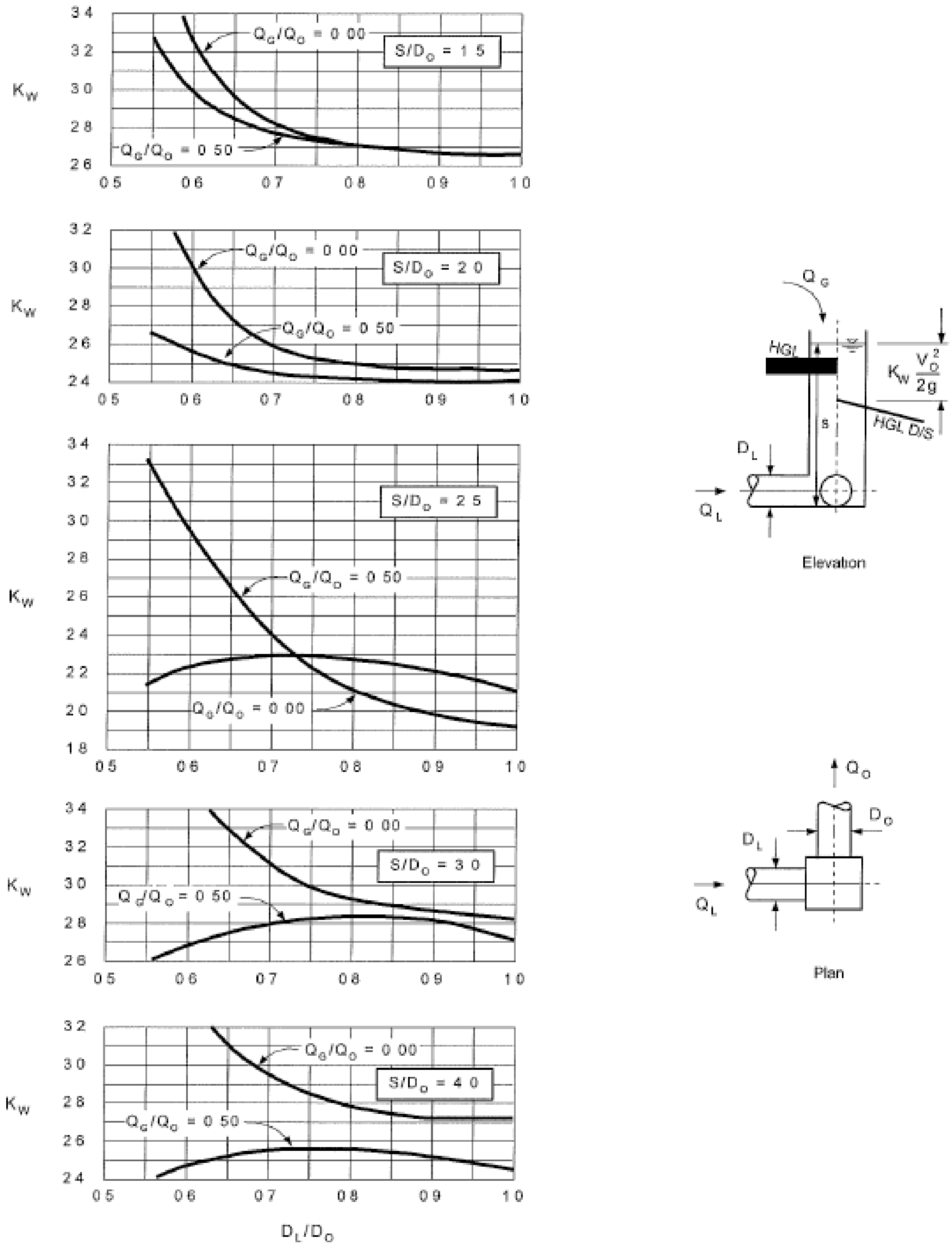

$$h_U = K_U \frac{V_O^2}{2g} = h_L = K_L \frac{V_O^2}{2g}$$

$$K_W = K_U (1.3 - 0.2\ Q_U/Q_O)$$

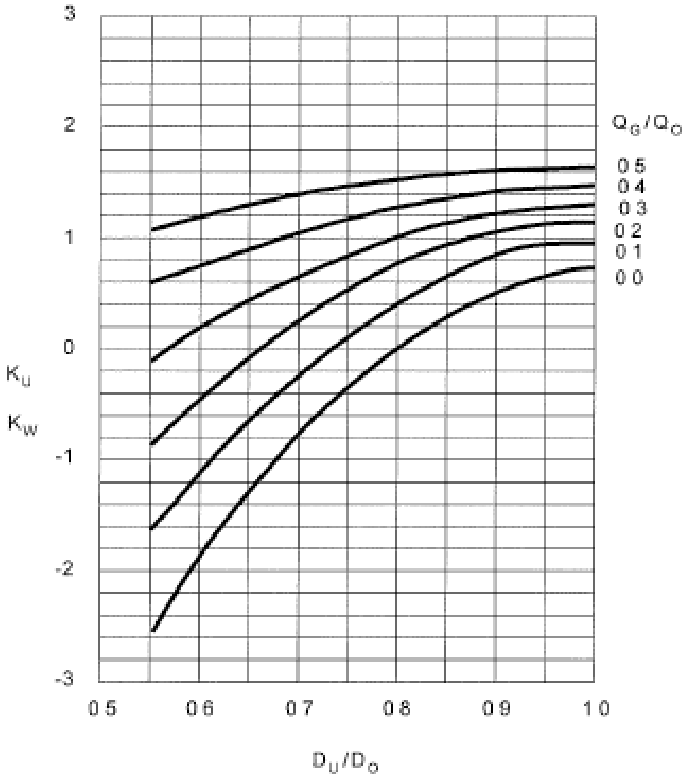
| | | |
|---|---|---|
| Source M | Pressure head change coefficient ($K_U$) for square or round manhole on through pipeline at junction of 90º lateral pipe (for conditions outside the range of Charts 4 & 5) | CHART 6 |

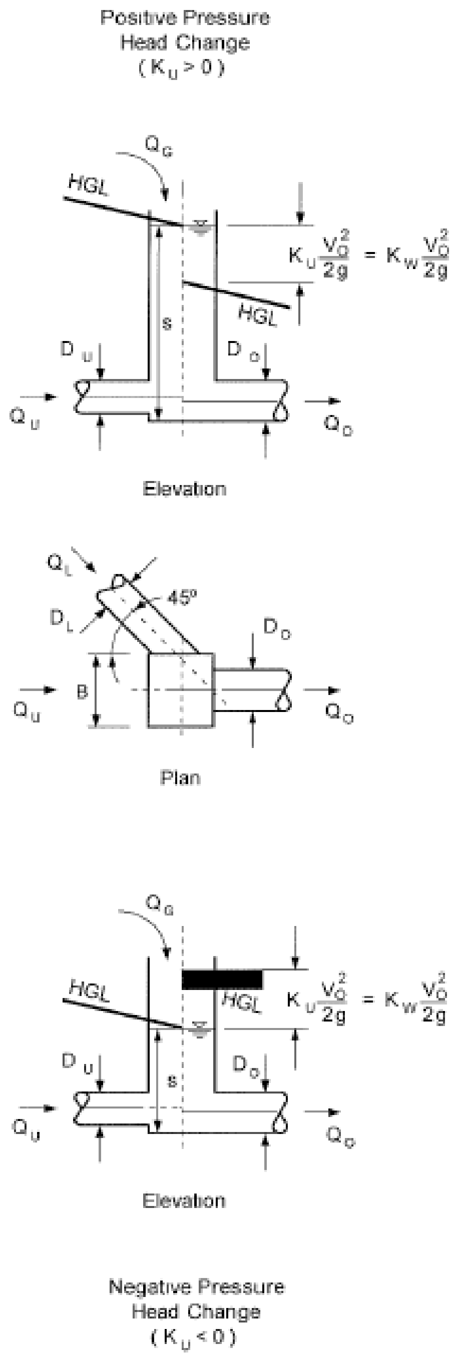



| | | |
|---|---|---|
| Source N | Pressure head change coefficients ($K_U$) for 90° bends at sump junctions | CHART 7 |




| | | |
|---|---|---|
| Source N | Pressure head change coefficients ($K_W$) for 90° bends at sump junctions | CHART 8 |

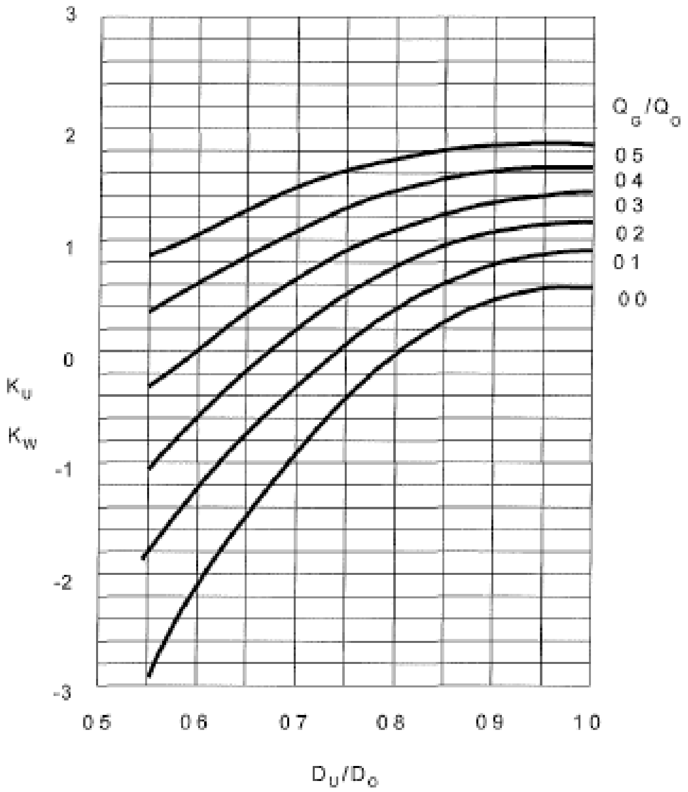


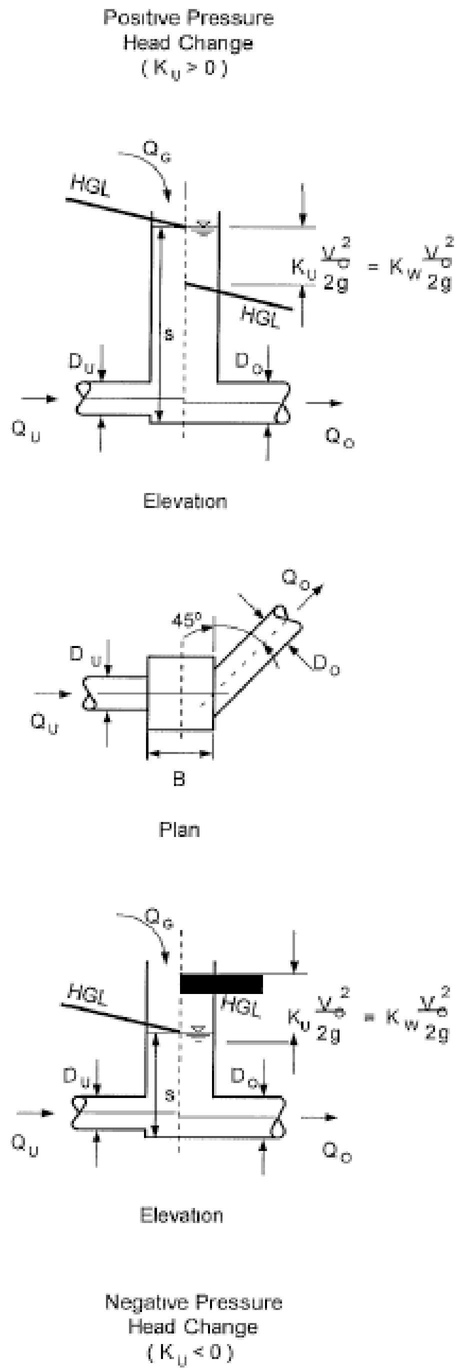




| $S/D_O$ \ $Q_G/Q_O$ | 0.00 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
|---|---|---|---|---|---|---|
| 1.5 | 0.00 | 0.11 | 0.22 | 0.33 | 0.44 | 0.55 |
| 2.0 | 0.00 | 0.04 | 0.08 | 0.12 | 0.16 | 0.20 |
| 2.5 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 3.0 | 0.00 | -0.03 | -0.06 | -0.09 | -0.12 | -0.15 |
| 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 |

Increment $K_U$ and $K_W$ by the above values for $S/D_O$ ratios other than 2.5

| Source N | Coefficients for 45° bends at sump junctions with branch point located on downstream face of sump | CHART 9 |
|---|---|---|

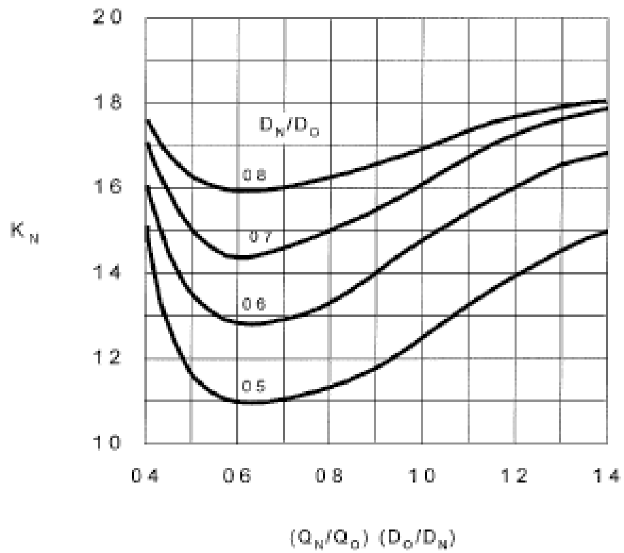



| $S/D_O$ \ $Q_G/Q_O$ | 0 00 | 0 10 | 0 20 | 0 30 | 0 40 | 0 50 |
|---|---|---|---|---|---|---|
| 1 5 | 0 00 | 0 11 | 0 22 | 0 33 | 0 44 | 0 55 |
| 2 0 | 0 00 | 0 04 | 0 08 | 0 12 | 0 16 | 0 20 |
| 2 5 | 0 00 | 0 00 | 0 00 | 0 00 | 0 00 | 0 00 |
| 3 0 | 0 00 | -0 03 | -0 06 | -0 09 | -0 12 | -0 15 |
| 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 |

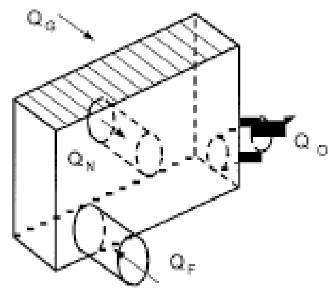
Increment $K_U$ and $K_W$ by the above values for $S/D_O$ ratios other than 2 5

| Source N | Coefficients for 45° bends at sump junctions with branch point located on downstream face of sump | CHART 10 |
|---|---|---|



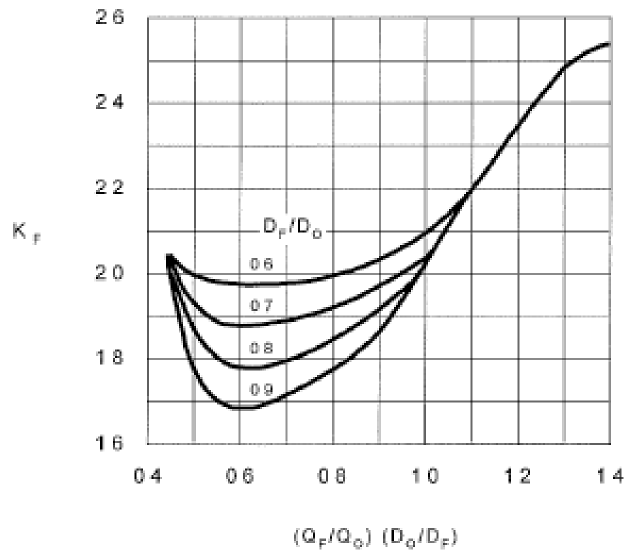

Near Lateral

$$h_N = K_N \frac{V_O^2}{2g}$$




$Q_F + Q_N + Q_G = Q_O$

Elevation Sketch




Far Lateral

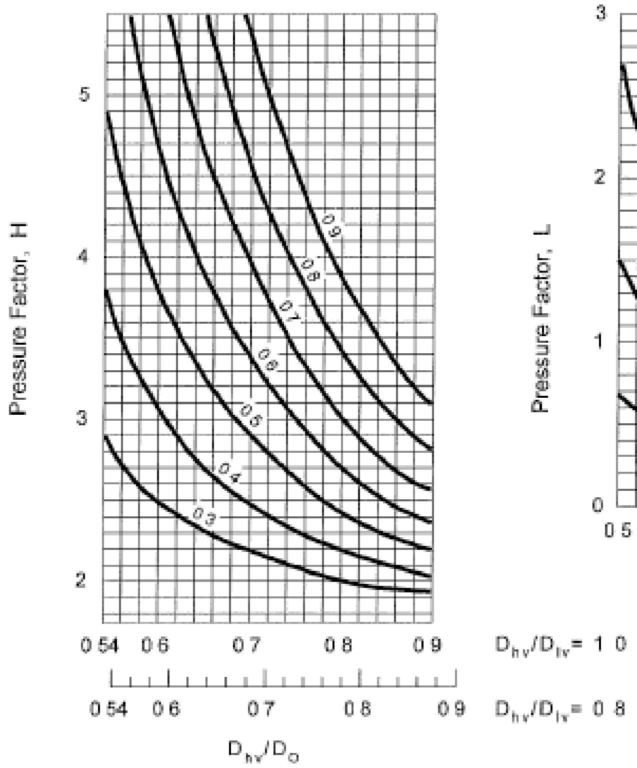
$$h_F = K_F \frac{V_O^2}{2g}$$

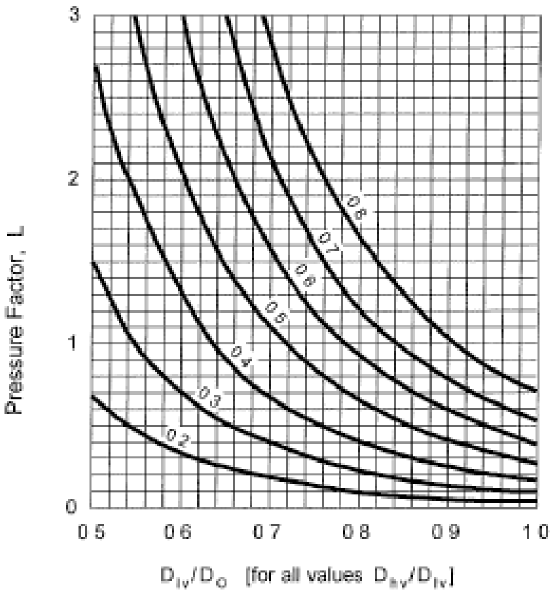
Note

1 For $Q_G = 0$ reduce $K_N$ & $K_F$ by 0 2

2 $K_W$ = 1 3 $K_F$

| | | |
|---|---|---|
| Source M | Pressure head change coefficients ($K_N$ & $K_F$) for rectangular inlet with offset opposed lateral pipes each at 90° to outfall (with or without grate flow) | CHART 11 |



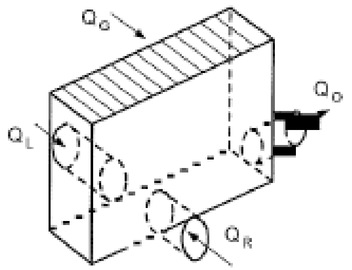




$D_{hv}$ = diameter of lateral with higher-velocity flow

$Q_{hv}$ = 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




$Q_F + Q_N + Q_G = Q_O$

Elevation Sketch

To find KR or KL for the right or left lateral pipe with flow at a lesser velocity than the other lateral, read H for the higher velocity lateral D and Q, then read L for the lower velocity lateral D and Q,

Then,

$$K_R (\text{or } K_L) = H - L$$

$K_R$ or $K_L$ for the lateral pipe with higher velocity flow is always 1 8

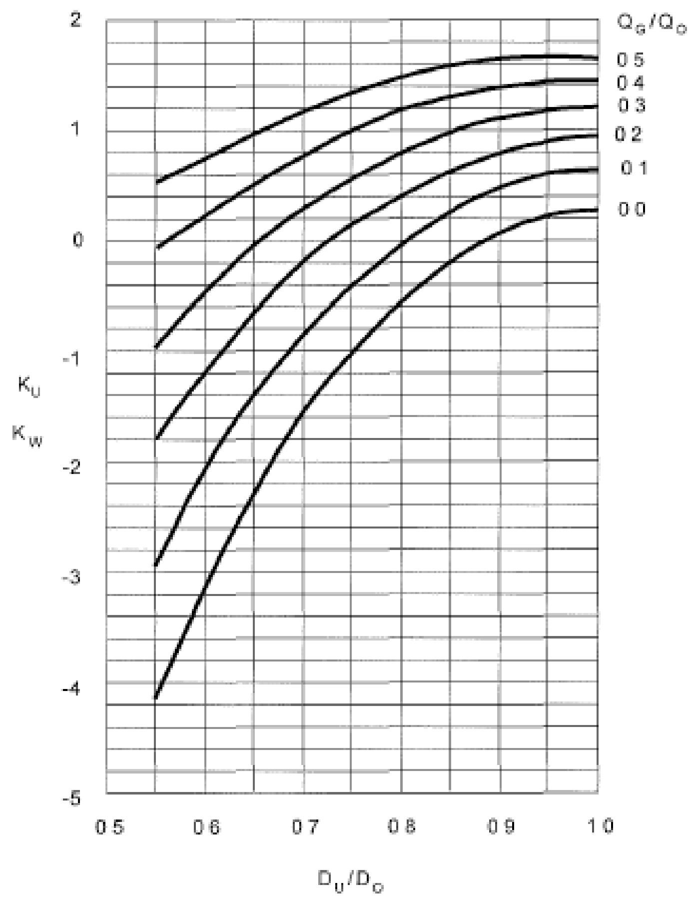
$$h_L = K_L \frac{V_O^2}{2g} \qquad h_R = K_R \frac{V_O^2}{2g}$$

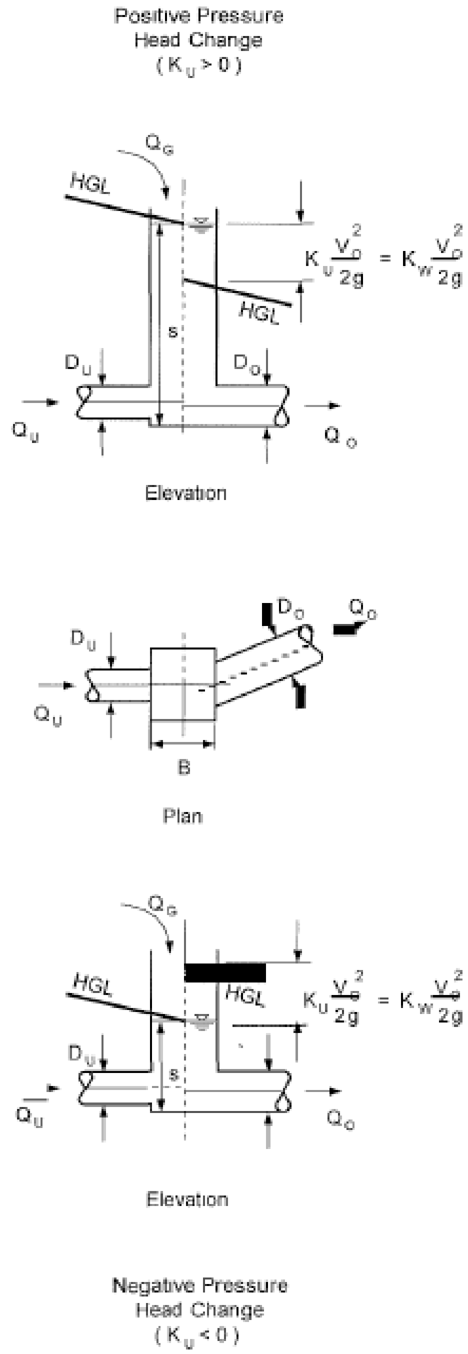
For $Q_G = 0$ reduce $K_R$ & $K_L$ by 0 2

$K_W = 1\ 3\ (K_R \text{ or } K_L)$

Use coefficient for highest velocity lateral

| | | |
|---|---|---|
| Source M | Pressure head change coefficients ($K_L$ & $K_R$) for rectangular inlet with in-line opposed lateral pipes each at 90° to outfall (with or without grate flow) | CHART 12 |

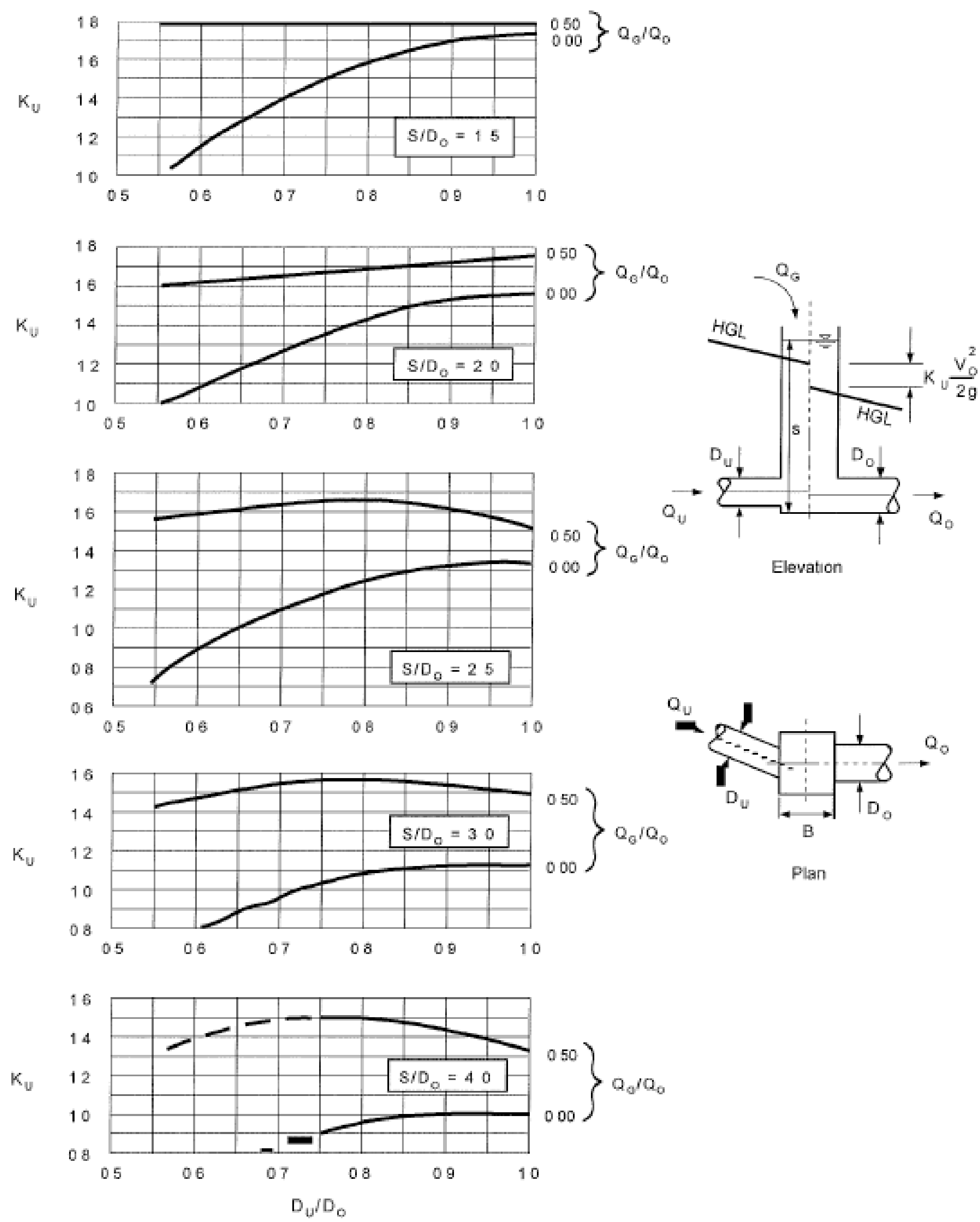


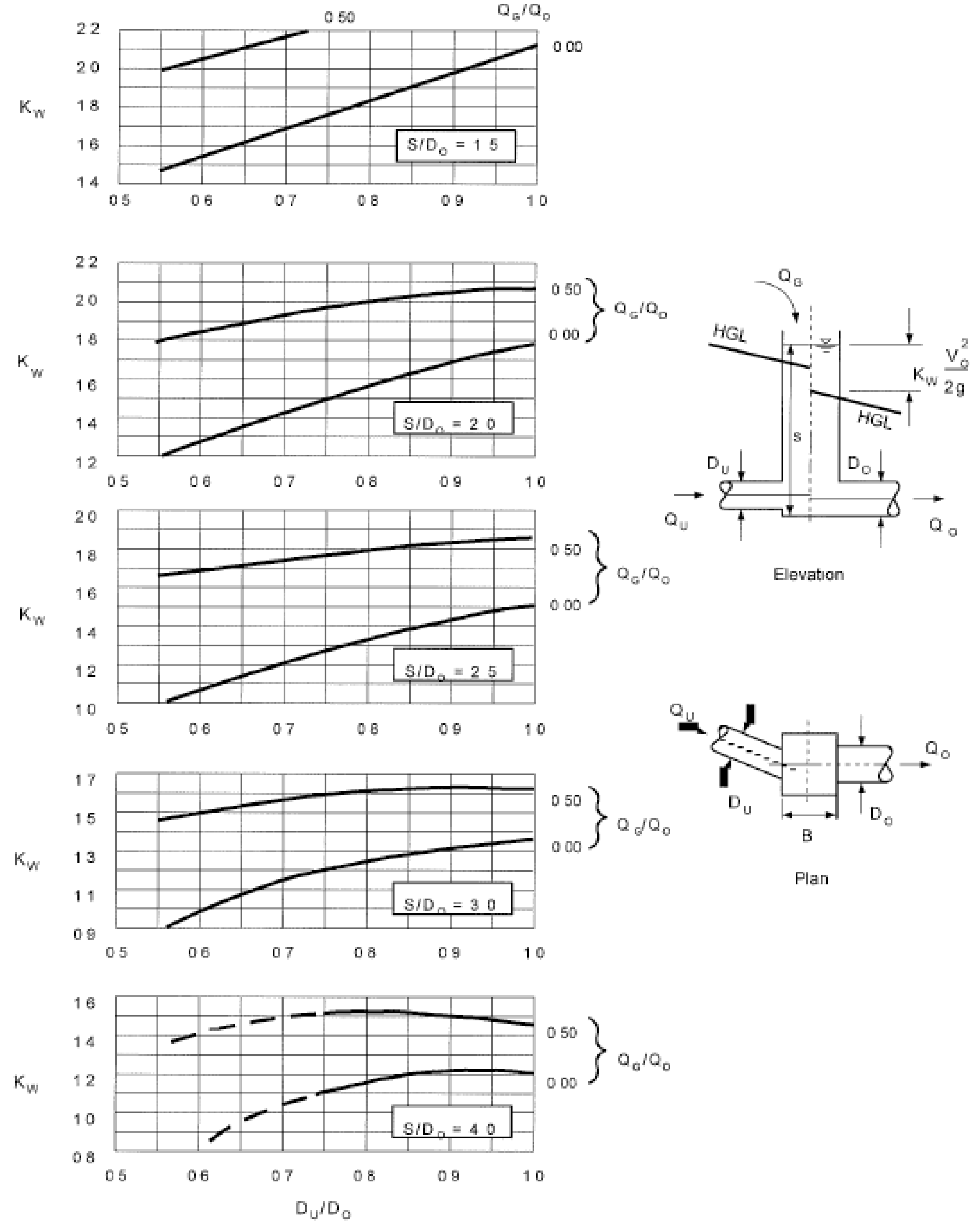

| $S/D_O$ \ $Q_G/Q_O$ | 0 00 | 0 10 | 0 20 | 0 30 | 0 40 | 0 50 |
|---|---|---|---|---|---|---|
| 1 5 | 0 00 | 0 11 | 0 22 | 0 33 | 0 44 | 0 55 |
| 2 0 | 0 00 | 0 04 | 0 08 | 0 12 | 0 16 | 0 20 |
| 2 5 | 0 00 | 0 00 | 0 00 | 0 00 | 0 00 | 0 00 |
| 3 0 | 0 00 | -0 03 | -0 06 | -0 09 | -0 12 | -0 15 |
| 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 |

Increment $K_U$ and $K_W$ by the above values for $S/D_O$ ratios other than 2 5

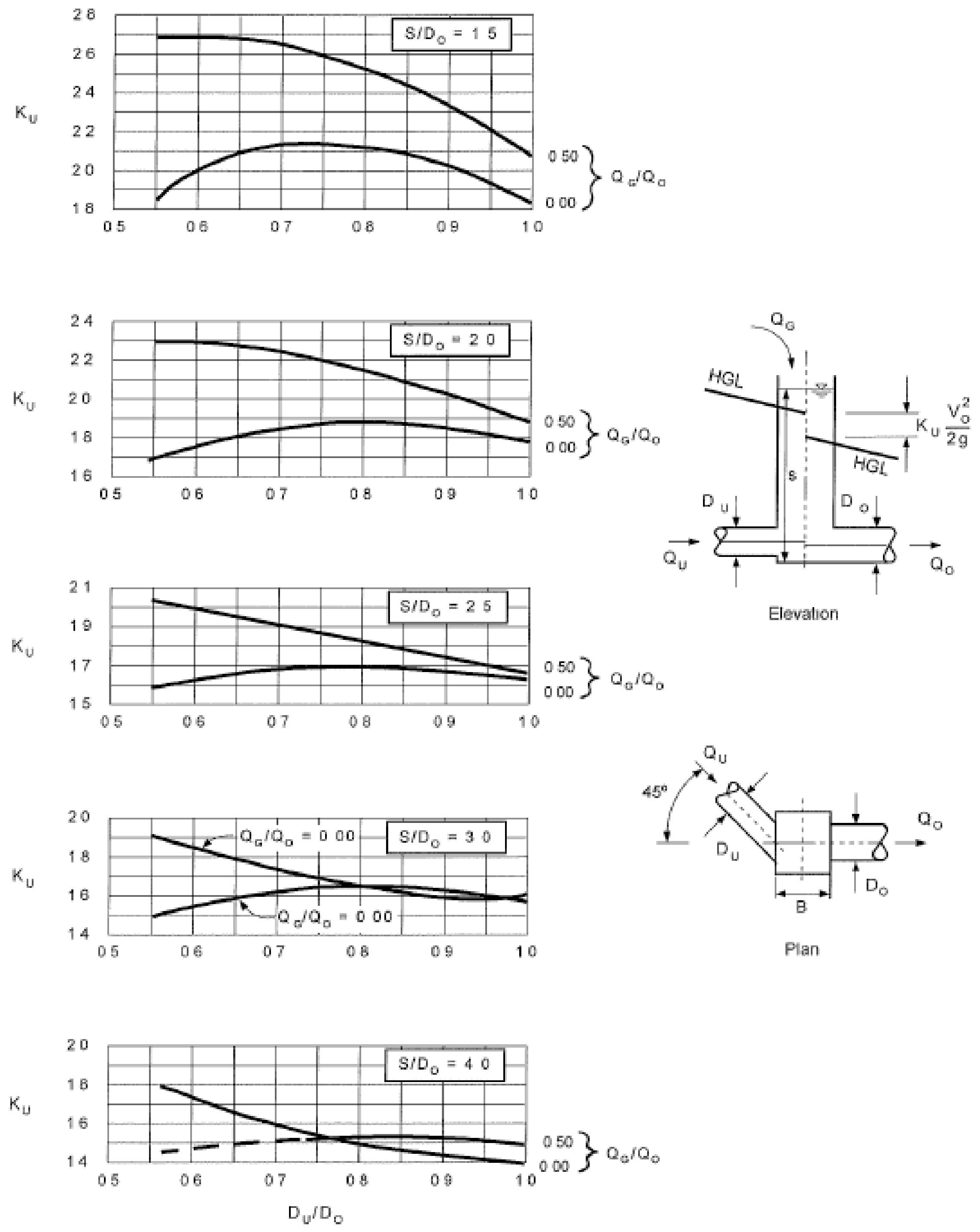
| Source N | Coefficients for 22 5° bends at sump junctions with branch point located on downstream face of sump | CHART 13 |
|---|---|---|



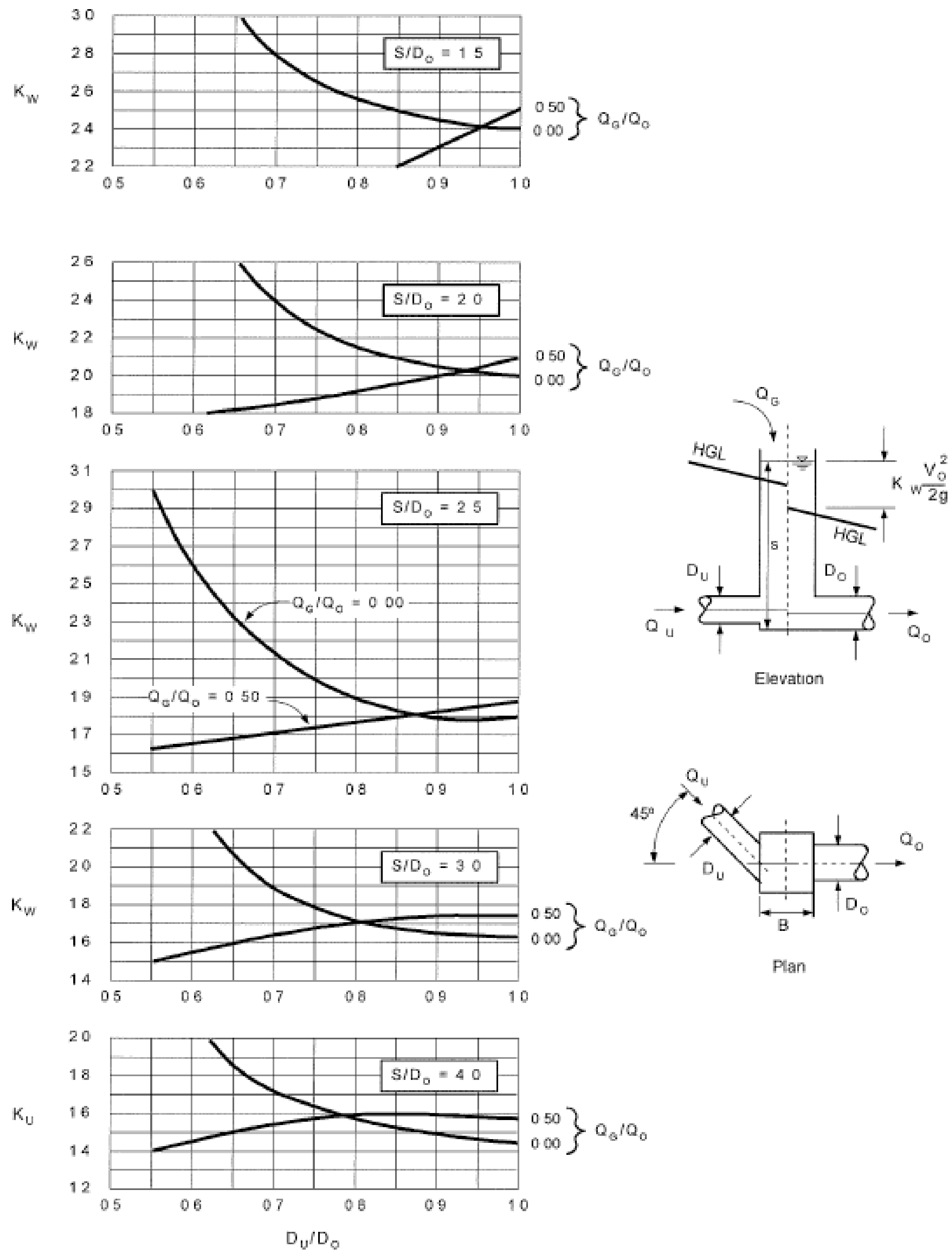

| Source N | Pressure change coefficients ($K_U$) for 22 5° bends at sump junctions with branch point located on upstream face of sump | CHART 14 |
|---|---|---|



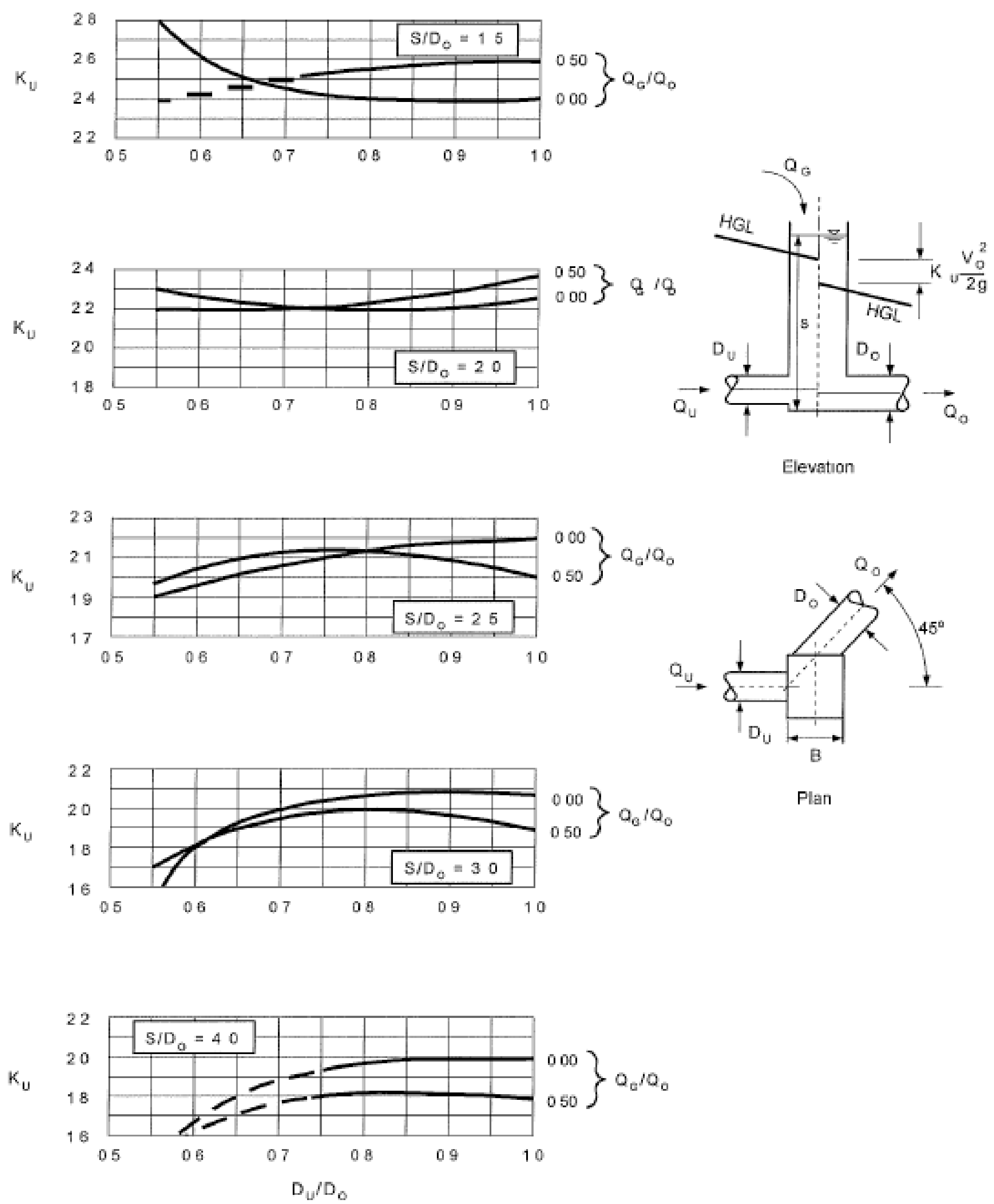

| | | |
|---|---|---|
| Source N | Pressure change coefficients ($K_W$) for 22 5° bends at sump junctions with branch point located on upstream face of sump | CHART 15 |



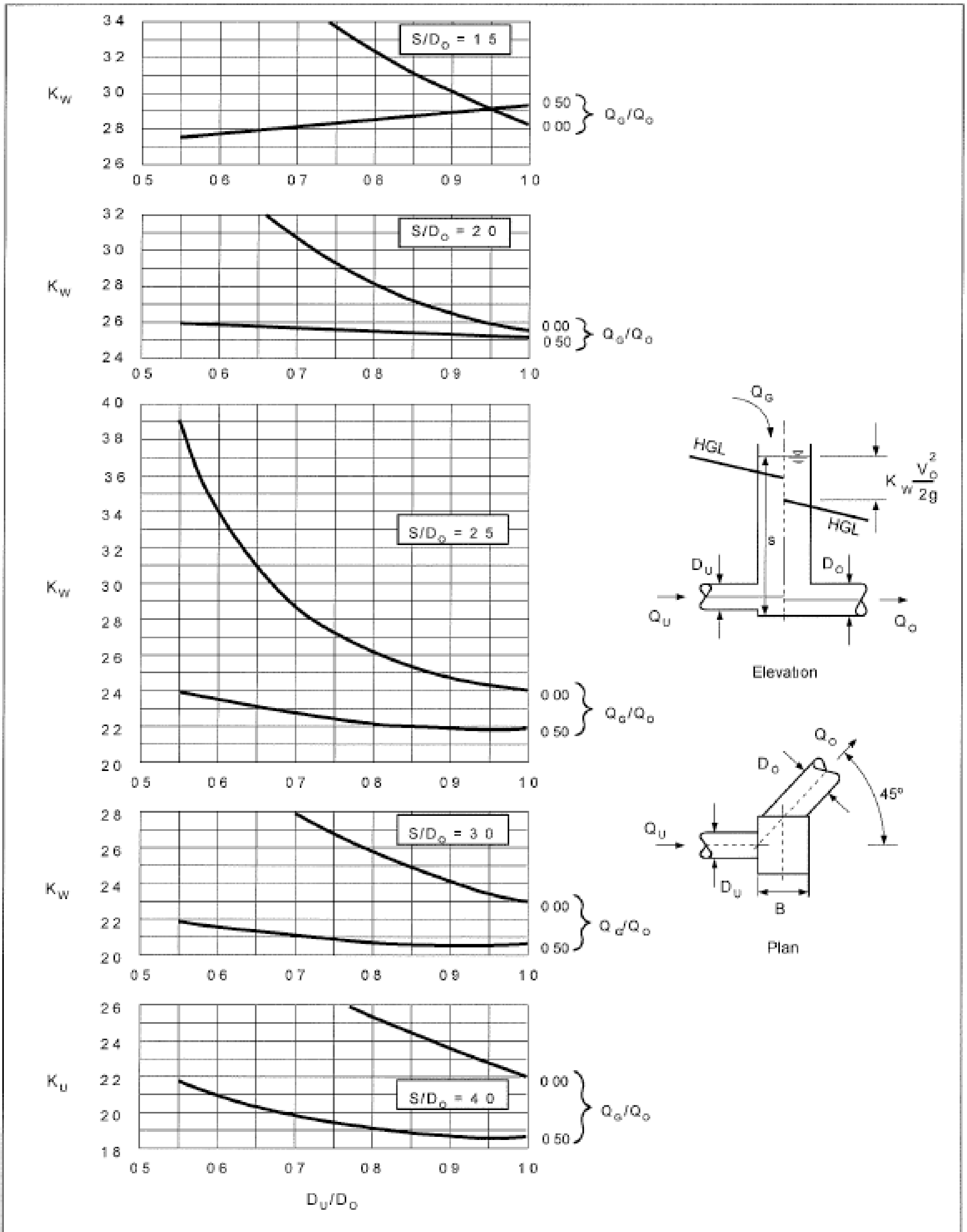

| | | |
|---|---|---|
| Source N | Pressure change coefficients ($K_U$) for 45° bends at sump junctions with branch point located on upstream face of sump | CHART 16 |



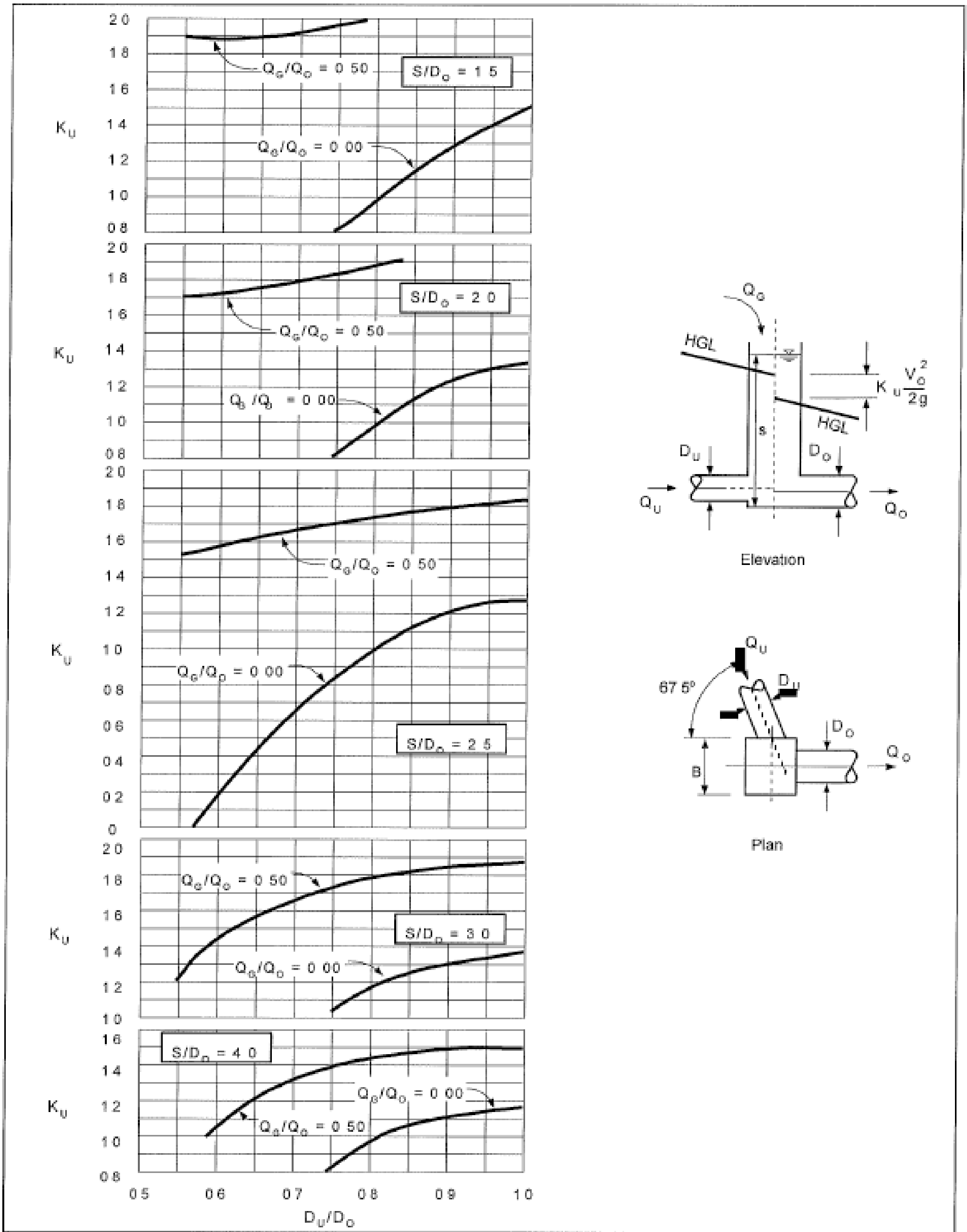

| | | |
|---|---|---|
| Source N | Pressure change coefficients ($K_W$) for 45° bends at sump junctions with branch point located on upstream face of sump | CHART 17 |




| | | |
|---|---|---|
| Source N | Pressure change coefficients ($K_U$) for 45° bends at sump junctions with branch point located on upstream face of sump | CHART 18 |



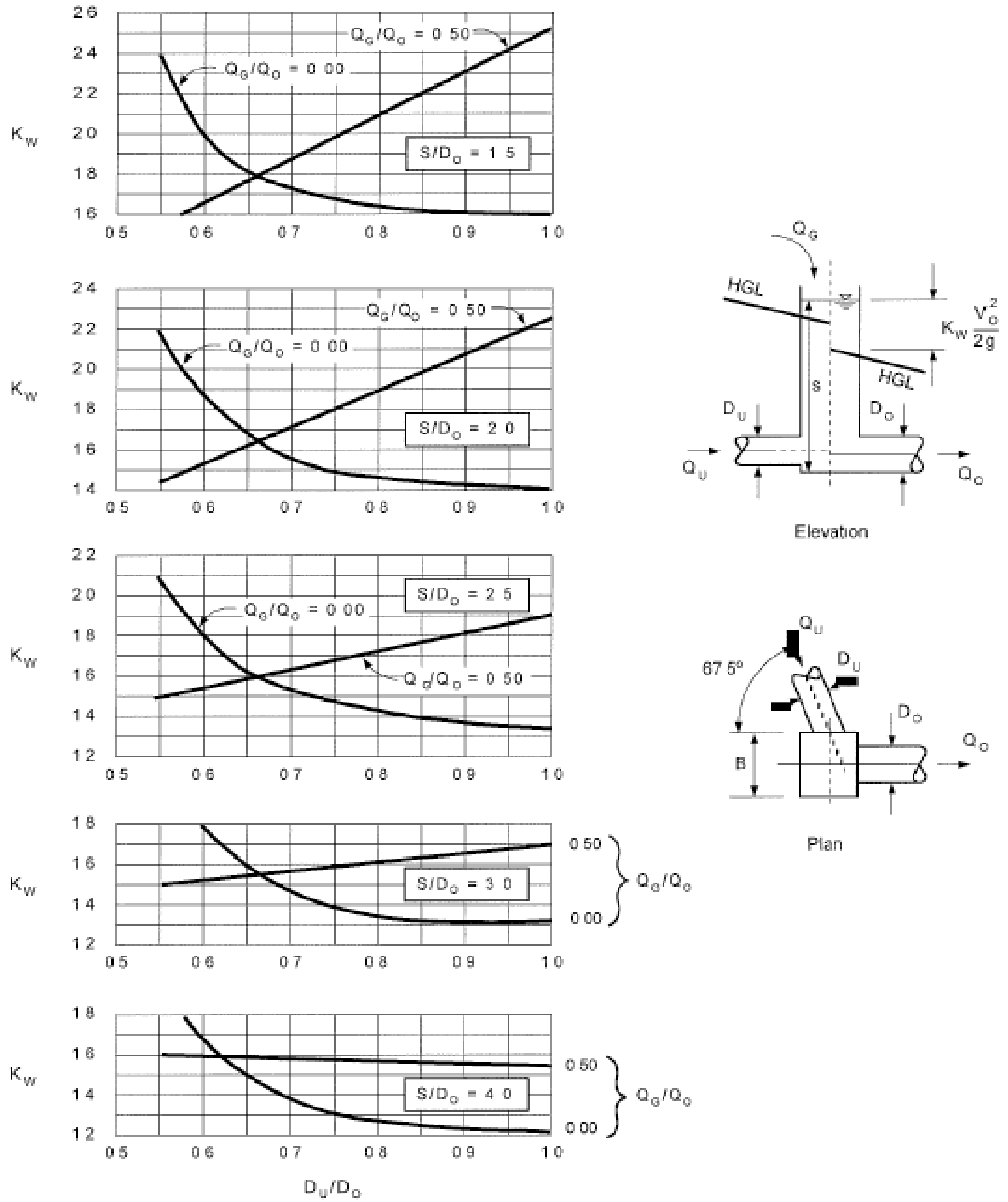

| | | |
|---|---|---|
| Source N | Pressure change coefficients ($K_W$) for 45° bends at sump junctions with branch point located on upstream face of sump | CHART 19 |



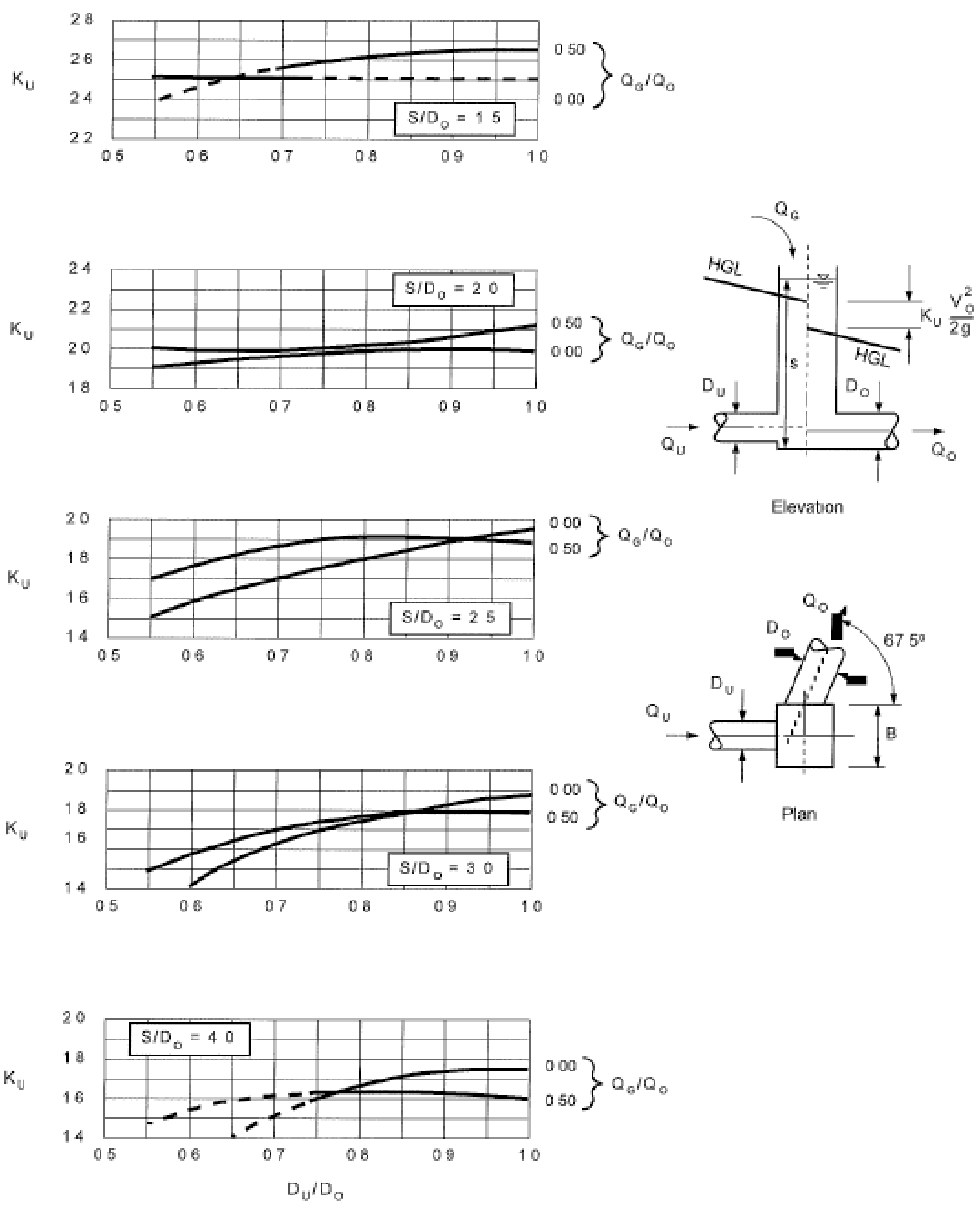

Source N

Pressure change coefficients ($K_U$) for 67 5º bends at sump junctions with branch point located near downstream face of sump

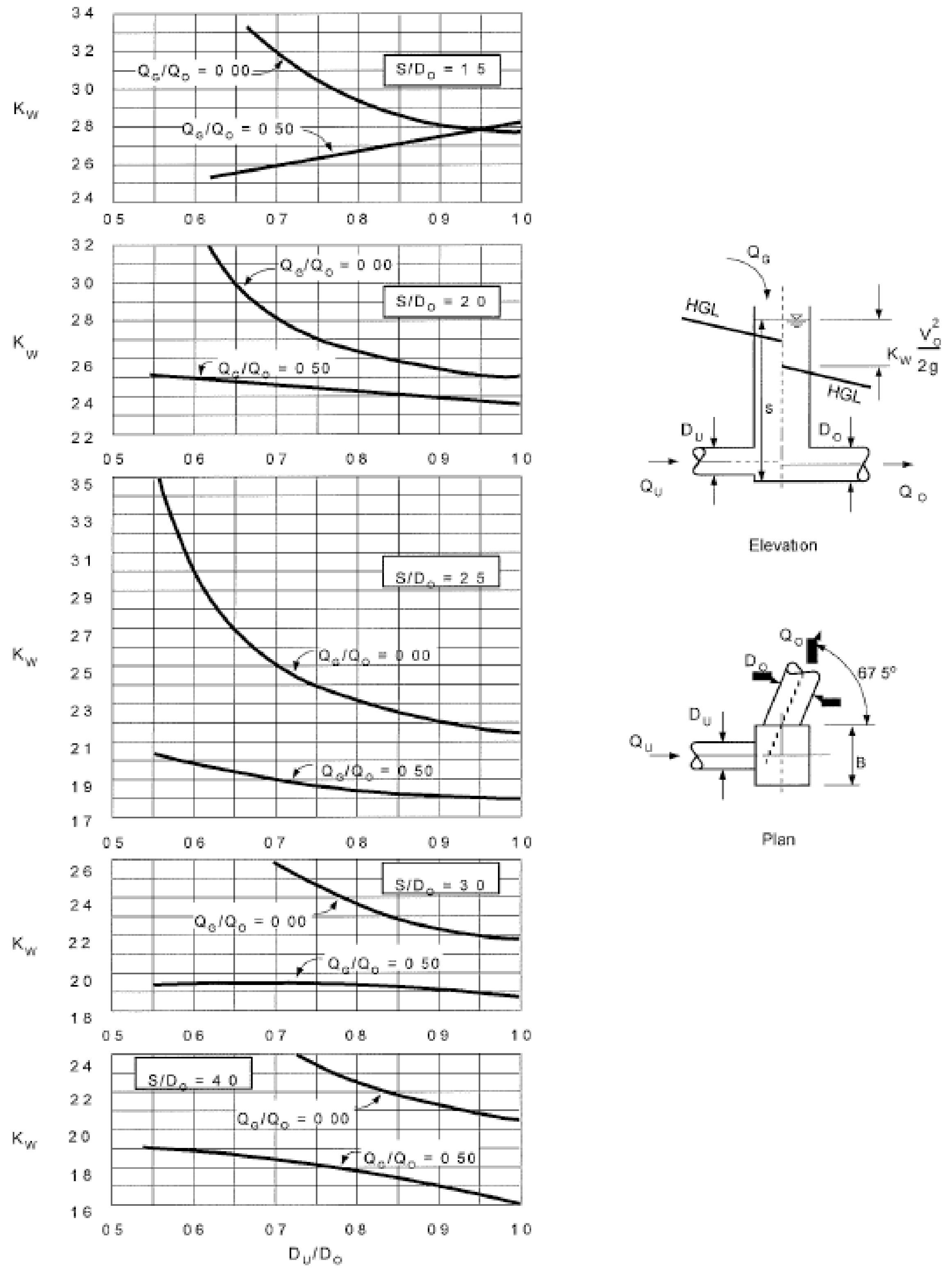
CHART 20




| | | |
|---|---|---|
| Source N | Pressure change coefficients ($K_W$) for 67 5° bends at sump junctions with branch point located near downstream face of sump | CHART 21 |




| | | |
|---|---|---|
| Source N | Pressure change coefficients ($K_U$) for 67 5° bends at sump junctions with branch point located near upstream face of sump | CHART 22 |




| | | |
|---|---|---|
| Source N | Pressure change coefficients ($K_W$) for 67 5° bends at sump junctions with branch point located near upstream face of sump | CHART 23 |