7.04 Roadway flow limits and capacity

It is necessary for road flow capacity to be checked for both the minor and major design storms. Design criteria are provided in Section 7.03. Additional criteria also apply and these are outlined in the following sections.

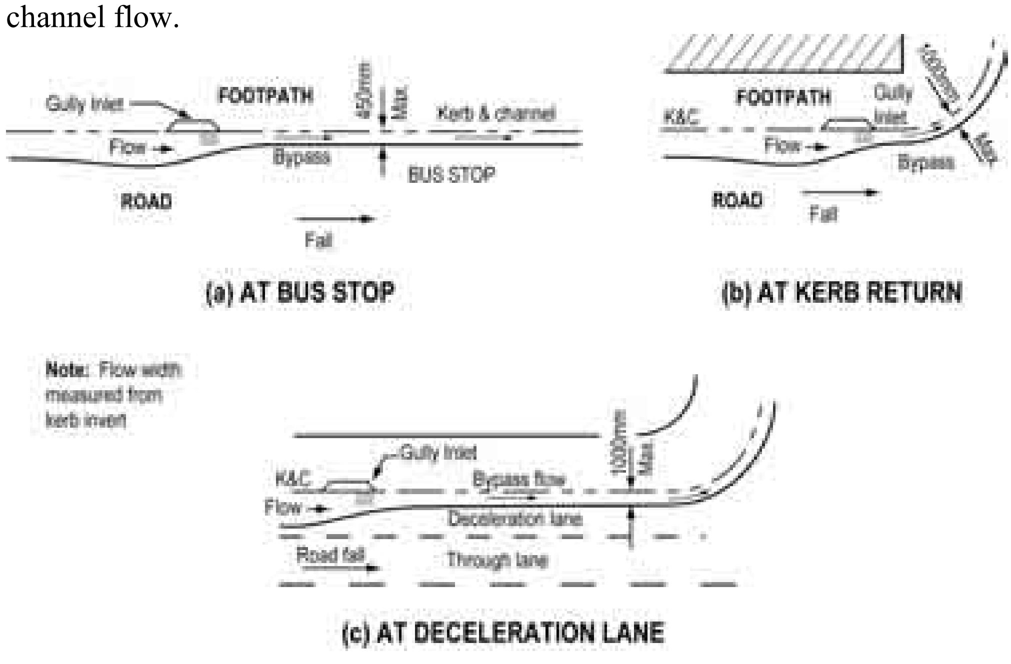
Note that in this section, and others, reference is generally made to roads with kerb and channel. This is not meant to preclude the use of grassed channels located at the verges, nor other edge treatments. The type of road edge treatment should be decided after consultation with the local authority.

7.04.1 Flow width (minor storm)

The flow width criteria for minor storms are related to the function of the road. Definitions of major and minor roads, for the purpose of this Manual, are contained in the Glossary.

It should be emphasised that flow width restrictions are dependent on the function of the road and its expected maximum traffic catchment. They are not necessarily a function of the road reserve or pavement width. Designers of drainage systems in existing areas are urged to clarify such issues with the local authority prior to design.

Relevant flow width limitations are contained in Table 7.04.1 and Figure 7.04.1. Flow width should be limited by whichever of the limitations in Table 7.04.1 is the more restrictive. The designer's attention is also drawn to the requirements of Table 7.02.1 in respect of design ARI for kerb and channel flow.



Typical Flow Width Criteria (Minor Storm)
Figure 7.04.1

Note: Flow width measured from kerb face.

Table 7.04.1 Roadway flow width [1] and depth limitations [2] (longitudinal drainage)

Roadway Flow Width and Depth Limitation	Major Road	Minor Road
1. For Minor Storm		
(a) Normal Situation	Parking Lane width (usually 2.5 m) or breakdown lane width. [3]	(i) Full pavement width with zero depth at crown; (ii) Where one way crossfall, to high side of road pavement, but not above top of kerb on low side.
(b) Where parking lane may become an acceleration, deceleration or turn lane.	1.0 m	Not applicable.
(c) Where road falls towards median.	1.0 m	Not applicable
(d) Pedestrian crossings or bus stops.	0.45 m	0.45 m
(e) At intersection kerb returns (including entrances to shopping centres and other major developments).	1.0 m [4] [5]	1.0 m [4] [5]
2. For Major Storm		
(a) Where floor levels of adjacent buildings are above road level.	(i) Total flow contained within road reserve. (ii) Freeboard ≥ 300 mm to floor level of adjacent buildings, and with maximum flow depth of 250 mm.	(i) Total flow contained within road reserve. (ii) Freeboard ≥ 300 mm to floor level of adjacent buildings, and with maximum flow depth of 250 mm.
(b) Where floor levels of adjacent building below or, less than 300 mm above top of kerb:		
(i) where 100 mm fall on footpath towards kerb;	50 mm above top of kerb.	50 mm above top of kerb.
(ii) where less than 100 mm fall on footpath towards kerb.	Top of kerb.	Top of kerb.
(c) Other.	As determined by local authority.	As determined by local authority.
3. Pedestrian Safety (Major and Minor Storms)		
(a) No Obvious Danger	$d*V \le 0.6\ m^2/s$	$d*V \le 0.6\ m^2/s$
(b) Obvious Danger	$d*V \le 0.4\ m^2/s$	$d*V \le 0.4\ m^2/s$
4. Vehicle Safety	$d*V \le 0.6\ m^2/s$	$d*V \le 0.6\ m^2/s$
Depth limit at kerb	$d \le 250$ mm	$d \le 250$ mm

Notes (Table 7.04.1):

[1] Widths are measured from channel invert for kerb and channel, and from kerb face for kerb only.

[2] Refer to Section 7.05.3 for a detailed explanation of appropriate location of kerb inlets.

[3] It may be necessary to limit discharge to 0.03 m^3/s upstream of small radius bends (less than 15m radius) to avoid flooding and traffic safety issues.

[4] Where flow is required to follow a kerb return at an intersection it may be necessary, where the longitudinal grade is steep, to check for the effect of flow superelevation upon flow spread. A procedure for the calculation of superelevation is given in Equation 9.08.

[5] When considering the 1.0m flow spread limitation at a kerb return the effect of the reduced pavement crossfall beyond the tangent point should be examined.

7.04.2 General requirements

(a) Pedestrian safety

The depth*velocity product is currently recommended as the best design measure for pedestrian safety within shallow-water overland flow paths; however, *State Planning Policy 1/03* provides an alternative design criteria which should be considered in the design and management of floodways.

Recent studies (Cox et.al., 2004) highlight that for some people, notably small children and frail older persons, there are no depth or velocity limitations that can be considered safe in all circumstances.

The product of depth d_g and velocity V_{ave} in the kerb and channel should not exceed 0.6 m^2/s (ARR-1998) to reduce hazard for pedestrians within the roadway. However, where there is an obvious risk of serious injury or loss of life, the $d_g . V_{ave}$ product should be limited to 0.4 m^2/s. This is applicable to longitudinal flow along the roadway for both Major and Minor Design Storms.

An "obvious risk of serious injury or loss of life" would include:

(i) Upstream of kerb inlets or any stormwater/pipe inlet with a clear opening greater than 90–125mm (at the discretion of the local authority–refer to Section 7.05.3(e)) where there is a risk to life resulting from small child entry into the downstream stormwater system.

(ii) Overland flow paths passing through, or discharging into flow conditions defined in Section 12.02 for Contact Classes A to D.

No definitive depth*velocity limitations can be specified for stormwater flow within childcare centres or areas frequented by elderly persons such as hospitals and retirement villages. Local governments should treat all situations on a case-by-case basis. Children with a Height*Mass product less than 20m.kg are generally of greatest risk. As a guide only, a depth*velocity product of 0.2 m^2/s may be considered acceptable within these higher risk areas.

(b) Major flows at T-junctions

Care should be taken in the design of surface flows at road T-Junctions adjacent steep hill slopes. In cases where the surface water enters a T-Junction via a steep gradient roadway, the high-velocity surface flow may fail to follow the desired flow path through the intersection. In the worst case scenario, the flow passes across the road junction—causing a traffic safety hazard—then enters the down-slope property potentially causing flooding and property damage.

(c) Flow capacity calculation for roadways with kerb and channel

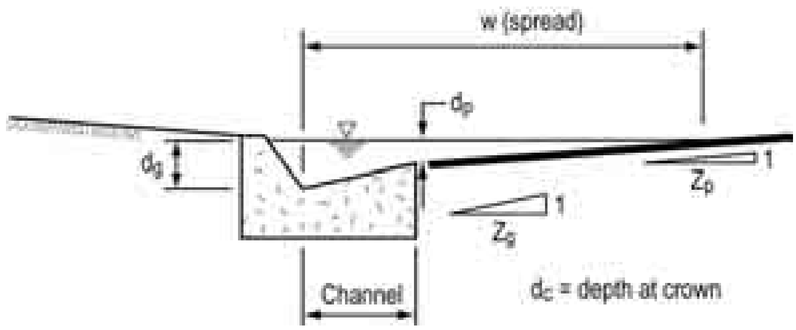
Roadway flow capacity may be calculated using Izzard's Equation (Refer Technical Note 4, Book 8, ARR-1998). The values outlined in Table 7.04.2 are recommended for Manning's Roughness Coefficient (*n*) and Flow Correction Factor (*F*).

Izzard's Equation provides a solution to flow determination in a triangular channel as follows:

$$Q = 0.375\ F.(Z/n).S^{0.5}.d^{2.667} \qquad (7.02)$$

For composite flow as in a half road where the pavement and channel have different roughness and crossfall, the equation becomes:

$$Q = 0.375\ F.\ [(Z_g/n_g).(d_g^{2.667} - d_p^{2.667}) + (Z_p/n_p).(d_p^{2.667} - d_c^{2.667})].S^{0.5} \qquad (7.03)$$



Half Road Flow
Figure 7.04.2

where
- Q = Longitudinal flow down kerb (m^3/s)
- F = Flow Correction Factor
- Z = Cross slope gradient
- Z_g = Cross slope gradient of kerb

Z_p = Cross slope gradient of pavement

n = Manning's roughness

n_g = Manning's roughness of kerb

n_p = Manning's roughness of pavement

S = longitudinal slope of kerb

d = maximum depth of flow

d_g = depth of flow at kerb invert

d_p = depth of flow at edge of pavement

d_c = depth of flow at crown

Table 7.04.2 Recommended values of Manning's roughness coefficient and flow correction factor for use in Izzard's equation [1]

Surface Type	***n***
Concrete	0.013
Hot Mix Asphaltic Concrete	0.015
Sprayed Seal	0.018
Kerb and Channel Type	***F***
Semi-mountable Type	0.9
Barrier Type (300 mm channel)	0.9
Barrier Type (450 mm channel)	0.9

Note:

[1] No recommendation is given in respect of the roughness on footpaths, it being normal practice to exclude the flow on the footpaths because of the likely presence of utility poles, landscaping etc.

(d) Resurfacing allowance

It is recommended that consideration be given to the effect of future resurfacing of roadways. Where such provision is to be included, allowance for a standard 25mm (asphaltic concrete) resurfacing is recommended unless directed otherwise by the local government.

Note that the construction of a 25mm thick asphaltic concrete overlay can reduce the waterway area to 45 to 65 percent of that available prior to overlay for the same depth at invert. Some increase in flow depth for the same flow must inevitably occur following an overlay.

7.05 Stormwater inlets

7.05.1 Kerb inlet types

(a) Four types of kerb inlets are in common use, they are:
  (i) Grate only e.g. field inlets and anti-ponding gullies on kerb returns.
  (ii) Side inlet – these inlets rely on the ability of the opening under the backstone or lintel to capture flow. They are usually depressed at the invert of the channel to improve capture capacity.
  (iii) Combination grate and side inlet – these inlets utilise the backstone arrangement of the side inlet with the added capacity of a grate in the channel.
  (iv) Special site specific designs for high inflow.

(b) Local authorities may determine appropriate kerb inlet types for a particular installation and should make available relevant standard drawings showing dimensions and set out details along with inlet capacity charts for those inlets.

7.05.2 Provision for blockage

Local authorities may indicate the percentage of blockage that is to be applied to the theoretical inflow capacity of inlets.

Where such guidance is not provided the recommendations in Table 7.05.1 should be adopted. Where the invert of the kerb is depressed at the inlet the capacity of the inlet should be adjusted accordingly.

Table 7.05.1 Provision for blockage at kerb inlets [1]

Condition	Inlet Type	Percentage of Theoretical Capacity Allowed
Sag	Kerb inlet	80%
	Grated	50%
	Combination	[2]
Continuous Grade (On-Grade)	Kerb inlet	80%
	Longitudinal bar grated	60%
	Transverse bar grate or longitudinal bar grate incorporating transverse bars	50%
	Combination	90% [3]

Notes:

[1] This table does not prevent local authorities from setting alternative blockage factors for site specific inlet designs.

[2] In a sag the capacity of a combination inlet should be taken to be the theoretical capacity of the kerb opening, the grate being assumed to be blocked.

[3] On a continuous grade the capacity of a combination inlet should be taken to be 90% of the combined theoretical capacity of the grate plus kerb opening.

This Manual does not include inflow capacity charts for kerb inlets. These charts should be obtained from the relevant local authority. Such charts should reflect the theoretical or measured capacity of the inlet, to which the above percentages should be applied to allow for blockage.

7.05.3 Kerb inlets in roads

(a) General

Kerb inlets should be provided at the following locations in kerb and channel:

(i) In the low points of all sags in kerb and channel.

(ii) On grades, to ensure compliance with the flow width limitations discussed in Section 7.04.

(iii) At the tangent point of kerb returns or small radius convex curves (kerb radius less than 15m) such that the flow width around the kerb return (i.e. beyond the kerb inlet) during the Minor Design Storm does not exceed 1.0m measured from the invert of kerb and channel. This limitation will also be applicable at important vehicular turnouts or footpath crossovers, where high traffic volumes are anticipated, such as at entrances to shopping centres.

(iv) Immediately upstream of potential pedestrian crossing and bus stops such that the flow width does not exceed 450mm from invert of kerb and channel during the Minor Design Storm.

(v) Immediately upstream of any reverse crossfall pavement to prevent flow across the road during the Minor Design Storm (i.e. at the start of crossfall transition from normal to reverse crossfall).

(vi) Where superelevation or reverse crossfall results in flow against traffic islands and medians. Kerb inlets shall be provided along the length of the island or median as necessary to meet the flow width limitations as stated in Section 7.04 and at the downstream end of the island or median to minimise the flow continuing along the road (see also (vii) below). Where sufficient width of island or median is available, grated kerb inlets should be recessed so that the grate does not project onto the road pavement. Alternatively side entry inlets with no grate should be installed.

(vii) Where reverse crossfall on a road pavement causes flow onto the pavement. The extent to which such flow onto the pavement is permissible depends upon the catchment area involved and the risk of vehicle aquaplaning. The question of aquaplaning is addressed in *Road Drainage Design Manual* (Department of Main Roads, 2001).

(viii) Where it is anticipated that a parking lane may become an acceleration, deceleration or turn lane in accordance with Table 7.04.1.

(ix) Consideration should be given to the positioning of kerb inlets relative to the side property boundaries. In residential and industrial locations, a kerb inlet located near the side property boundary may cause difficulties with driveway access. In commercial areas and those where there is likely to be a high volume of pedestrian traffic, kerb inlets should be located to avoid set down points or locations where pedestrian movements are likely to be highest.

(b) Kerb inlets on grade

(i) Designers should be aware that kerb inlet capacity is controlled by the crossfall of the road pavement and the longitudinal grade.

(ii) Bypass flow from a kerb inlet must be accounted for in the design of the downstream kerb inlet which receives the bypass flow. There is no limitation to the amount of flow which may be bypassed from a kerb inlet provided that the flow width criteria discussed in Section 7.04 are adhered to. Note that a number of road flow capacity calculations may be required, using actual crossfalls at the intersection, to check that all bypass flows are contained within the 1.0m flow width limitation at kerb returns, under minor storm conditions.

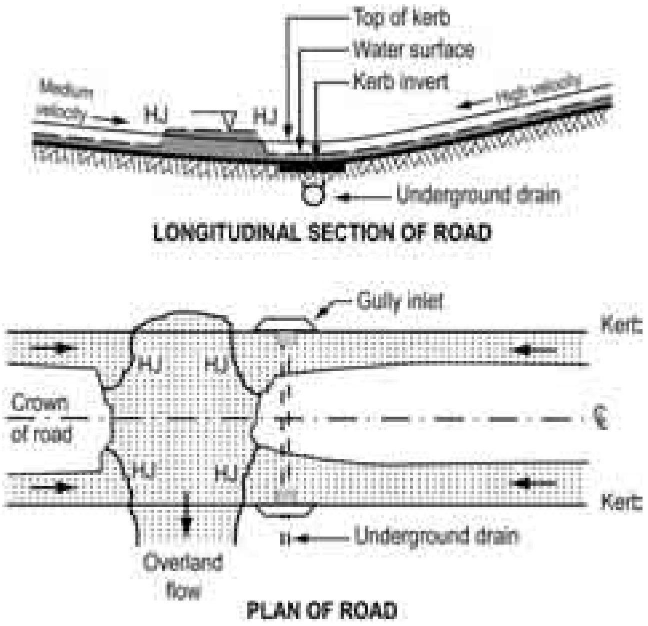
(iii) Where bypass flow from a kerb inlet is required to follow a kerb return at an intersection it may be necessary, where the longitudinal grade is steep, to check for the effect of flow superelevation upon flow spread. A procedure for the calculation of superelevation is given in Equation 9.08.

(iv) The procedure detailed in Figure 7.05.3 is recommended for determining the location of kerb inlets on grade.

(c) Kerb inlets in sags

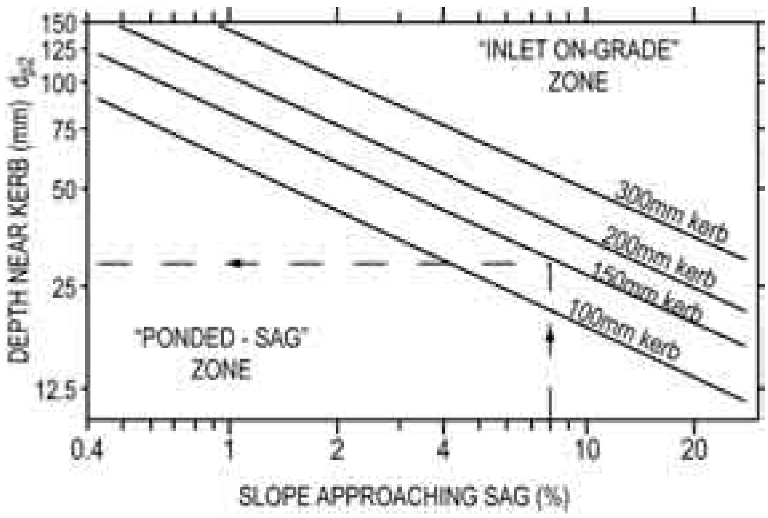
Kerb inlets in sags must have sufficient inflow capacity to accept the total flow (including bypass flows from upstream) reaching the inlet. Ponding of water at sag inlets should be limited to the widths discussed in Section 7.04 particularly at intersections where turning traffic is likely to encounter ponding water.

Where the longitudinal grades on either side of the sag are different, or where the flow from one direction is dominant, the location of the effective sag may move from the true sag and a hydraulic jump may form beyond the sag. Care should be taken, by the provision of extended or additional inlets, to ensure that capture capacity is maintained and that the water level does not cause flow over the footpath into the adjacent property. A procedure for checking whether this effect is occurring has been proposed by Black (1987a) and is detailed in Figures 7.05.1 and 7.05.2.



A sag in a road with supercritical approach flows (“HJ” indicates a hydraulic jump)

Figure 7.05.1



Limiting condition for a sag inlet to act as an on-grade inlet (n = 0.013) (Source: Black, 1987a)

Figure 7.05.2

Notes:

[1] e.g. for kerb height = 150mm and approach slope = 8%

[2] "Inlet on grade" conditions will apply for flow depth > 30mm

[3] i.e. ponding may exceed kerb height after hydraulic jump unless kerb inlets are extended towards the flatter side of the sag.

Guidance for the use of this Figure is contained in Volume 2

(d) Intersections

(i) Consideration shall be given to the steepness of grade of the road and the possibility of momentum carrying water past the stormwater inlet/s, across the road and into properties opposite the intersection. Solution to such problems may require extra inlets to be installed. Also refer to Section 7.04.2 (b).

(ii) Where two falling grades meet at an intersection, every endeavour should be made to locate the low point of the kerb and channel at one of the tangent points of the kerb return.

(iii) Where both grades are steep it may not be practicable to locate the low point at a tangent point. In this case, kerb inlets should be provided at both tangent points, with additional inlets provided upstream of the tangent points, if necessary, designed to limit the flow width beyond the kerb return. An anti-ponding kerb inlet (grate only) installed within the width of the channel—nominally 450mm long by 300mm wide with no kerb inlet should be provided at the low point.

The location of a kerb inlet, or a grated inlet that protrudes onto the pavement within a kerb return is considered unsatisfactory because of the risk of damage by and to vehicles.

(e) Safety issues

In locations where the kerb inlet is accessible by a small child, whether deliberate or as a result of a child being swept down the flooded kerb, then the maximum clear opening height for a kerb inlet shall not exceed 125mm.

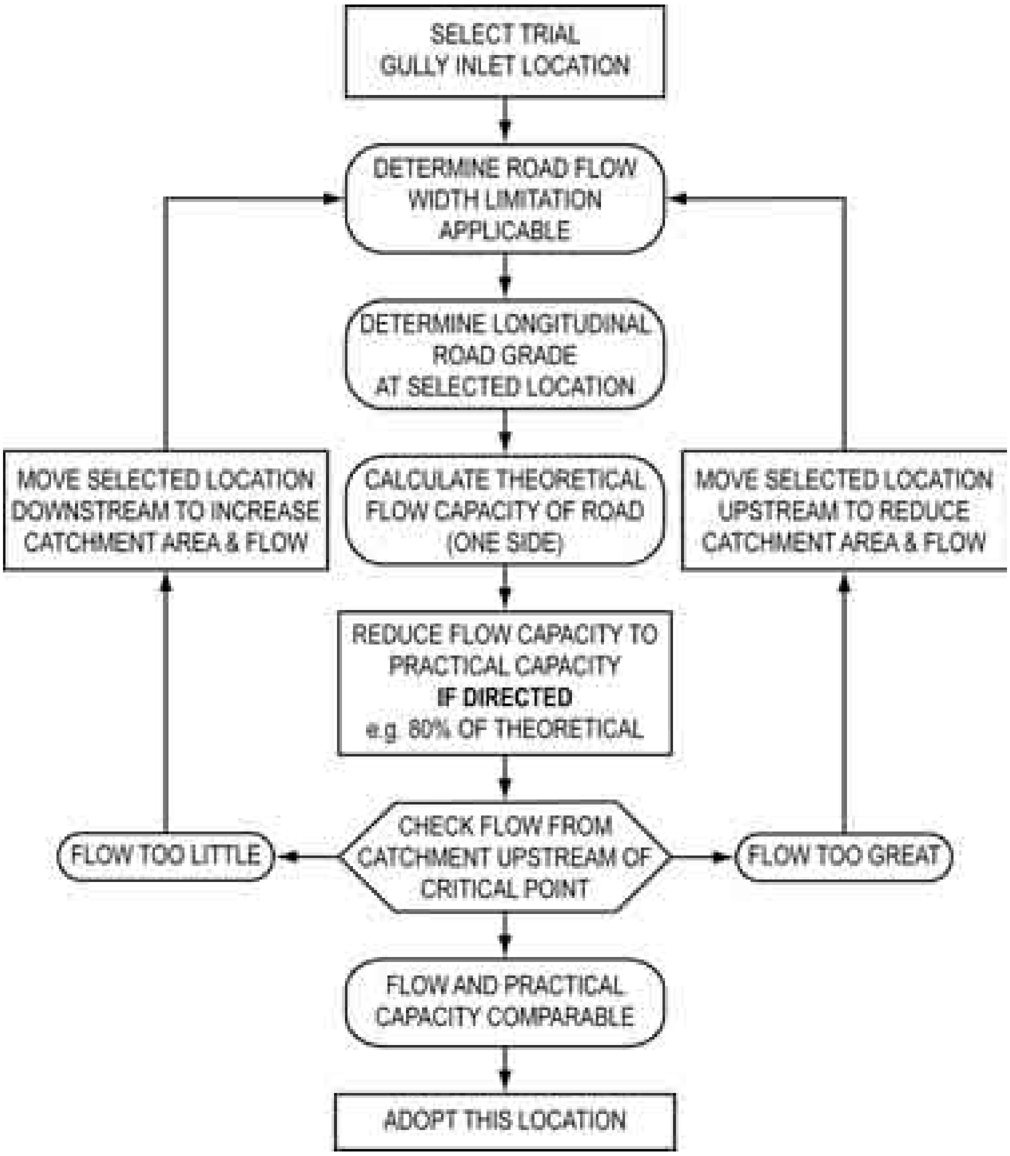
Local authorities may choose to reduce this maximum clear opening to 90 or 100mm if the increase risk of a 125mm opening is considered unacceptable (refer to Technical Note 7.05.1).

Technical Note 7.05.1:

Considerable debate exists regarding the recommended maximum clear opening for kerb inlets to provide safety for small children. Even though past history has shown the “likelihood” to be low, the “consequences” of a child being swept down a flooded kerb and into a stormwater inlet can be extreme.

After consideration of the various arguments presented to the QUDM Reference Group, the recommendation for 125mm maximum clear opening was accepted. However, the 125mm opening still presents a risk of a small children partially entering (i.e. feet first) the inlet.

A maximum clear opening of 90mm is recommended where it is necessary to exclude the entry of the torso of a 2-year-old child. Such consideration may apply in parks, schools and childcare centres.



Flow chart for determining kerb inlet positions on grade

Figure 7.05.3

Notes:

[1] Changes in catchment area may result in changes in *time of concentration* for a catchment.

[2] The above procedure is iterative.

[3] Selection of the initial trial kerb inlet location may be based on changes in road grade (e.g. steep to flat), physical restrictions in road (e.g. median or Residential Street Management devices), or by driveways, entrances or intersections etc.

7.05.4 Field inlets

Field inlets (also known as drop inlets) should be provided in parks, footpaths, medians etc. as necessary, to drain all low points, and should be provided within allotments in accordance with Section 7.13.

Where there is considerable pedestrian traffic adjacent to a field inlet e.g. in a footpath, a grate with close bar spacing should be used—recommended bar spacing is provided in section (d) below. Elsewhere a grate with wide bar spacing is preferable, because of the reduced risk of blockage by debris.

In all situations an allowance for blockage of 50% of the clear opening area of the grate should be made.

(a) Inflow Capacity

The inflow capacity of a field inlet depends upon the depth of water over the inlet. For shallow depths the flow will behave as for a sharp crested weir. For greater depths the inlet will become submerged and inflow will behave as for an orifice. **It is recommended that the capacity of the inlet be checked using both procedures and the lesser inlet capacity adopted.**

(i) Under weir flow conditions (Figure 7.05.4):

$$Q_g = BF.\ x\ 1.66\ L.h^{3/2} \qquad (7.04)$$

where Q_g = flow into field inlet (m^3/s)

$BF.$ = blockage factor = 0.5

1.66 = weir coefficient

L = weir length (m) (see note below)

h = depth of water upstream of inlet (relative to weir crest) where flow velocity is low (i.e. velocity head is insignificant) otherwise use the height of energy level above the weir crest(m)

Note: The length referred to in this case is the effective weir length. Thus for a grated inlet adjacent to a kerb, the side along the kerb should be ignored. For a side inlet the length referred to is the length of the inlet.

(ii) Under orifice flow conditions (Figure 7.05.5):

The orifice flow equation depends on the pressure gradient across the orifice. The standard orifice flow equation applies when “atmospheric” pressure conditions exist downstream of the grate, such as would exist if the design Water Surface Elevation (WSE) is 150mm below the grate (as per Table 7.16.1 and Figure 7.05.5).

Equation 7.05 is based upon a pressure change coefficient of $K_g = 2.75$.

$$Q_g = BF. \times 0.60\, A_g\, .(2g.h)^{1/2} \qquad (7.05)$$

where Q_g = flow into field inlet (m^3/s)

$BF.$ = blockage factor = 0.5

A_g = clear opening area of grate (m^2)

h = depth of approaching water relative to the orifice (m)

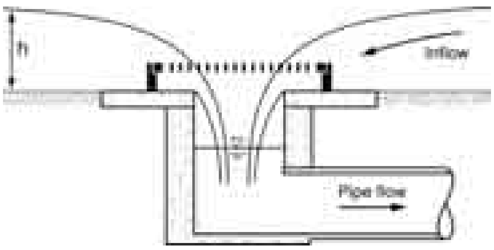
g = acceleration due to gravity (9.79 m/s^2)

0.60 = constant = $(1/K_g)^{1/2}$ = $(1/2.75)^{1/2}$

K_g = pressure change coefficient for the grate

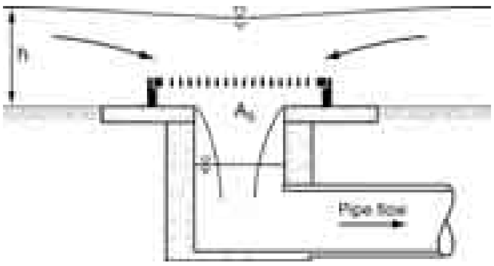
The pressure change coefficient (K_g) can vary significantly for unusual grate designs. The coefficient used in Equation 7.05 is based on a typical open mesh grate. It is noted that the pressure change coefficient for the old cast iron "City Grate" has been adopted as 2.23. Designers of unusual hydraulic structures should seek expert advice or review reference documents on orifice flow.

If the field inlet is fully drowned (i.e. no air gap exists below the grate and thus the hydraulic pressure below the grate is not atmospheric) then an estimate must be made of the head loss through the structures as per a normal Hydraulic Grade Line (HGL) analysis. Such calculations require considerable experience and hydraulic judgement. Guidance on head losses through screens is provided in Sections 7.16.14(c) and 12.04.6.



Field inlet operating under weir flow

Figure 7.05.4



Field inlet operating under free orifice flow

Figure 7.05.5

(b) Freeboard considerations

Freeboard provisions should be made at field inlets as follows:

(i) Where the inlet is contained within a pond formed by earth mounds or similar, freeboard should be 20% of the depth of the pond with a minimum of 50mm under minor storm conditions. However where overflow must be avoided the design storm shall be the major storm event.

(ii) Where flooding of buildings is possible freeboard provision should be in accordance with Section 7.03 for the major storm event.

(c) Minimum width of scour protection lip

The concrete lip formed around a field inlet should have sufficient width to:

(i) minimise the risk of grass growing over the grate, or causing blockage of the grate;

(ii) prevent scour of an adjoining surface.

Unless otherwise supported by site specific hydraulic calculations, the minimum recommended "lip" width (*Z*) required to minimise the risk of scour within the adjoining grass may be determined from Equation 7.06.

$$Z = 2.3\,A_g/L \qquad (7.06)$$

where:

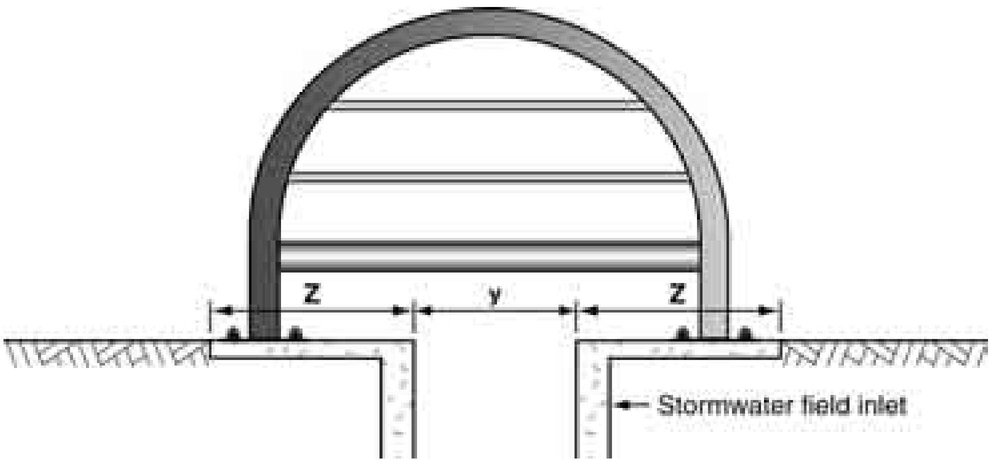
Z = minimum lip width for scour protection (m)

A_g = effective "clear" opening area of drop inlet (m^2)

L = total internal circumference of drop inlet (m)

Thus, for square inlets ($A_g = y^2$ & $L = 4y$) the minimum lip width: $Z = 0.57y$
where:

y = internal side dimension of square drop inlet (m)



Minimum lip width required for scour protection
(dome inlet screen shown as example only)

Figure 7.05.6

(d) Safety issues

Safety risks should be reviewed in circumstances where a field inlet is located within areas accessible to the public. Safety considerations include the following:

(i) Safety risks associated with people tripping over the screen (i.e. if not set flush with the ground).

(ii) Inlet screens located in vehicular or pedestrian areas shall comply with the requirements of AS 3996.

(iii) If there is the risk of a child being swept by stormwater towards a horizontal inlet screen, then the maximum clear spacing of the bars shall be 90mm.

(iv) If there is the risk of a child being swept by stormwater towards a vertical or inclined inlet screen, then the maximum clear spacing of the bars shall be 125mm.

(v) Maximum clear bar spacing of 89mm if located within a park or playground (AS4685.1 *Playgrounds and Playground Equipment*), otherwise a maximum spacing of 125mm.

(vi) Flow velocities through the screen/grate sufficiently low to prevent a child from being held against the screen/grate by hydraulic pressure. It is recommended that the maximum flow velocity through the grate/screen should be 1m/s.

Raised, horizontal screens are generally not acceptable adjacent footpaths, bikeways or public areas where significant numbers of people gather as these inlets may represent an unacceptable safety risk. In such circumstances, flush screens should be used, or possibly large dome screens if such screens are likely to be clearly visible and not represent a safety risk. Alternatively, marker posts or fencing may be used.

7.06 Access chambers

7.06.1 General

Access chambers should be provided on drainlines:

- to provide access for maintenance;
- at changes of direction, grade or level;
- at junctions.

Consideration should be given to the placement of an access chamber at an obstruction or penetration by a conduit or service, to facilitate the removal of debris.

The maximum recommended spacing is given in Table 7.06.1.

Table 7.06.1 Recommended maximum spacing of access chambers

Condition	Pipe Size (mm)	Spacing (m)
Generally	Less than 1200	100
	1200 and above	150
Immediately upstream of outlet to tidal waterway	All	100
Roadways	All	200

The local authority may direct that standard access chamber should be used and may make available standard drawings for these installations. However for multiple pipes, large diameter pipes, or odd configurations of pipes, it may be necessary to design a special chamber. Special chambers should be designed to accept the loadings detailed in Section 7.09 of this Manual.

Benching of the floors of access chambers leads to a general reduction in losses and promotes improved hydraulic efficiency (Johnston et al. 1990).

Technical Note 7.06.1:
Benching does not necessarily help to align incoming flows with the outlet pipe. Instead, benching works by reducing the effects of flow expansion adjacent the base of the access chamber. The higher the benching and the more it removes effective "deadwater" zones around the base of the chamber, the more effective the reduction in losses.

Hydraulic improvements are difficult to quantify and the construction of benching can be costly. Benching is therefore recommended only when it is important to minimise losses. Further information is provided in Section 7.16.8 (b).

Some local authorities exclude or limit the use of precast access chambers and designers should check that they are acceptable. In cases where precast chambers are used, the connecting stormwater pipes should not protrude into the chamber and should be sealed and finished in accordance with an approved construction detail.

The geometry of pipes at access chambers is critical in respect of hydraulic head loss. This matter is discussed further throughout Section 7.16. The main principles to be followed to minimise head loss are:

(a) Minimise changes in flow velocity through the chamber.

(b) Minimise changes in flow direction.

(c) Avoid "opposed lateral" inflows, i.e. all incoming pipes should ideally be contained within a 90^{o} arc, but certainly less than 180 degrees.

(d) Limit the deflection from inflow to outflow for pipes smaller than 600mm diameter to 90 degrees, or 67.5 degrees for pipes 600mm and greater in diameter.

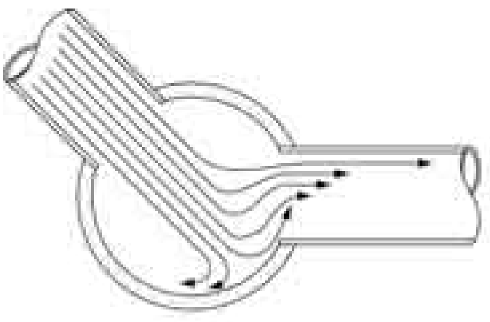
(e) Avoid vertical misalignment i.e. "drop pits", unless deliberately intending to induce high head loss.

(f) Where practical, direct inlet pipes wholly into the barrel of the outlet pipe (Figure 7.06.2). It is noted that for various reasons, inflow pipes often need to be directed towards the centre of the pit (Figure 7.06.1), however, this will increase losses.

(g) Rounding the entrance to the outlet pipe at a radius of one-twelfth of the outlet diameter will help to reduce losses (Figure 7.06.3).

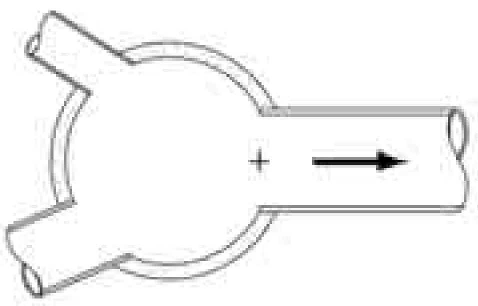
(h) Where practical, the change of direction of flow should occur at or near the downstream face of the chamber.

(i) Head losses resulting from surface inflows (Figure 7.06.4) are reduced if the design water level in the chamber is well above the outlet pipe obvert.



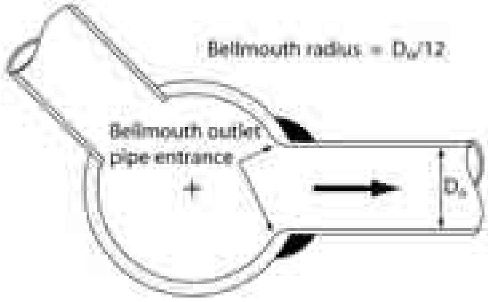
Flow lines resulting from inflow pipe directed at pit centre

Figure 7.06.1



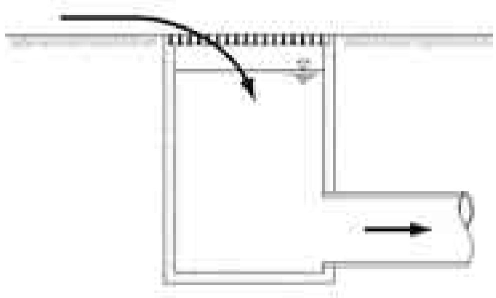
Inflow pipe directed at centre of outflow pipe

Figure 7.06.2



Bellmouth entrance to outlet pipe

figure 7.06.3



Inlet chamber showing water level well above outlet obvert

Figure 7.06.4

7.06.2 Access chamber tops

Access chambers in a carriageway or paved surface should be finished with their tops flush with the finished surface. Where an access chamber is located within a carriageway, the chamber top, or access point, should be positioned to avoid wheel paths.

Elsewhere, access chambers should be finished 25mm above natural surface with the topsoil or grassed surface around the chamber graded gently away. On playing fields they may be finished 200mm below the finished level, but only when located in a straight line between two permanently accessible chambers.

7.06.3 Deflection of pipe joints, splayed joints etc.

Changes of direction for drainlines of 1200mm diameter or greater may be achieved by deflection of pipe joints, the use of splayed joints or fabricated bends.

The recommended radius of curvature for pipes with deflected joints or splayed units should be as agreed with the relevant local authority in consultation with the pipe manufacturer.

Plans showing curved stormwater lines should show the radius of curvature, the total deflection angle, the maximum deflection per pipe length, the length of pipes and the joint type.

7.06.4 Reduction in pipe size

For single drainlines, a downstream pipe of smaller diameter than the upstream pipe may be permitted as long as the system works hydraulically and as long as the change in diameter is no greater than the following:

Table 7.06.2 Recommended maximum reduction in pipe size – single pipes

Upstream Pipe Diameter (mm)	Allowable Change in Diameter
Less than 600	No change
675 to 1200	ONE pipe size
Greater than 1200	TWO pipe sizes

The above recommendations are based upon the nominal sizes of pipes as manufactured in accordance with AS 4058.

At the location where the reduction in size occurs, pipes should be graded invert to invert to prevent the accumulation of sediment etc.

7.06.5 Surcharge chambers

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

(i) The potential for a person (that has been swept into the upstream drainage system) being trapped inside the surcharge chamber unable to exit the chamber or the outlet pipe.

(ii) Potential surcharge of the upstream system and flooding problems caused by debris blockage of the outlet screen.

(iii) Structural integrity of the chamber, outlet screen, top slab and concrete coping, and its ability to withstand high outflow velocities and high "pressure" forces caused by debris blockages. There is a need in many cases to ensure the surcharge screen is securely anchored to the top slab, and the slab to the chamber walls, to avoid displacement of the chamber lid/screen.

(iv) Safe maintenance access to allow removal of debris trapped within the surcharge chamber.

The hydraulic analysis of surcharge chambers is presented in Section 7.16.14

7.07 Pipeline location

Minor pipes connecting one kerb inlet to another is acceptable at the top of the street drainage system. These pipes may be located under the kerb and channel.

For pipelines greater than 600mm it is recommended that the location for drainlines in the road pavement—other than a kerb inlet to kerb inlet connection—be 2.0 metres measured towards the road centreline from the invert of the kerb and channel. The required location should be verified with the local government. Access chamber tops or access points should be located to avoid wheel paths.

Where sufficient verge width is available stormwater pipes may be located in the verge to suit the services allocations of the relevant local government.

In divided roads, drainage pipelines may be located within the median, normally offset 1.5 metres from the centreline (as street lighting poles are normally on the centreline).

If reasonable alternative locations are available drainage pipelines should not be located within allotments. In many cases overland flow requirements will require the provision of a pathway, drainage reserve or park in which the pipelines may be located.