24 STORMWATER INLETS

24.1 GENERAL

Stormwater runoff presents numerous safety hazards in urban areas. On-road ponding, reduced visibility and hydroplaning of vehicles are some of the hazards. In an urban setting these hazards are substantially magnified due to the increased traffic and pedestrian density.

Stormwater inlets, also known as gully inlets, are mainly provided to collect this stormwater from the paved surfaces, parks, landscaped and open space areas, and transfer it to underground pipe drains. Even where an open drain system is used, the inlets connect to the open drains by means of pipes. The provisions apply to both types of drainage system.

Inlets will not function properly if the downstream pipe or open drain system has insufficient capacity, causing backwater. The designer of these systems should refer to Chapters 25 and 26 respectively. As a guideline it is desirable to have at least 1.0 m height difference between the road level and the drain invert in order for the inlets to operate correctly.

Installing of inlets is encouraged in a more highly urbanised areas, for draining more runoff from streets, parking lots and airport facilities although more developed countries are now beginning to shift from hard engineering to soft engineering using roadside swale. This Chapter does not apply to roads where the runoff should discharge directly to a roadside swale (Chapter 26 and 31).

The materials used in this Chapter were adapted mainly from FHWA (1996) and QUDM (1992).

24.1.1 Pavement Inlets

The most common type of inlet is that from a road pavement. Inlets also provide access to pipes for maintenance. Standard sizes and shapes should be used to achieve economy in construction and maintenance. Adequate road drainage helps to protect the road subgrade from water-logging and damages. A typical arrangement of road drainage and stormwater inlets is shown in Figure 24.1.

The location of inlets on roads is governed by the safe flow limits in gutters. When selecting and locating inlets, consideration shall be given to hydraulic efficiency, vehicle, bicycle and pedestrian safety, debris collection potential, and maintenance problems. Care is needed to ensure that property access is not impeded. These principles are explained in greater detail in subsequent sections.

Three types of inlets may be utilised for pavement drainage:

- grate inlet (Figure 24.2a)
- kerb inlet (Figure 24.2b)
- combined inlet, grate and kerb (Figure 24.2c)

Kerb inlets are less affected by blockage. Extended kerb inlets, using lintel supports, can be used to increase capacity. The combined grate and kerb inlet (Figure 24.2c) is the most efficient, and it should be used on urban roads wherever possible. Details of the recommended standard kerb inlets are shown in Standard Drawing No. SD F-1.

Grates are effective in intercepting gutter flows, and they also provide an access opening for maintenance. In some situations they are prone to blockage. All grates on road should be an approved, bicycle-friendly design. FHWA (1978) have investigated several grates for inlets and developed bicycle-safe grate configurations. Typical schematic of bicycle-friendly grates are shown in Figure 24.3.

24.1.2 Other Inlets

Inlets are not normally required for drainage from private property, because in Malaysian practice this drainage is usually discharged into an open drain along the property boundary.

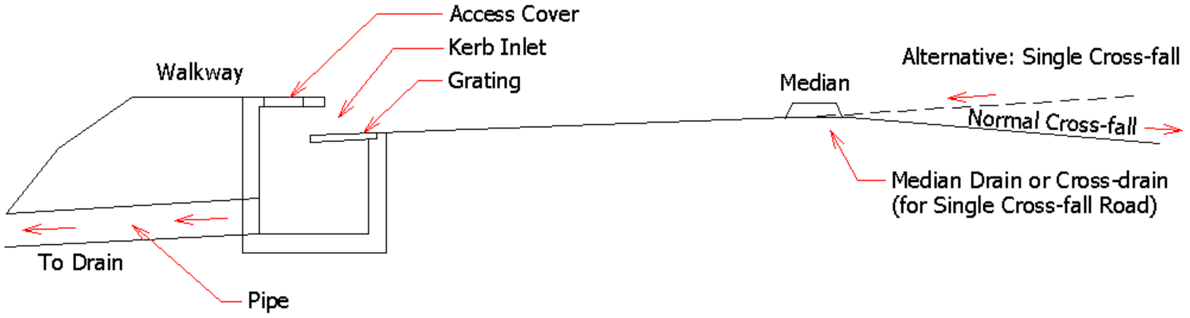


Figure 24.1 Road Drainage System and Stormwater Inlets

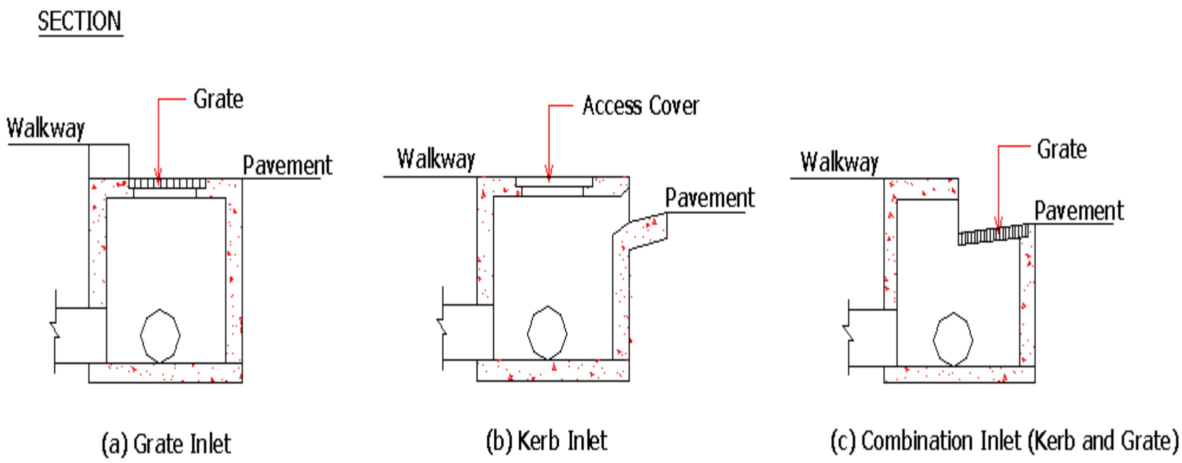


Figure 24.2 Pavement Inlets

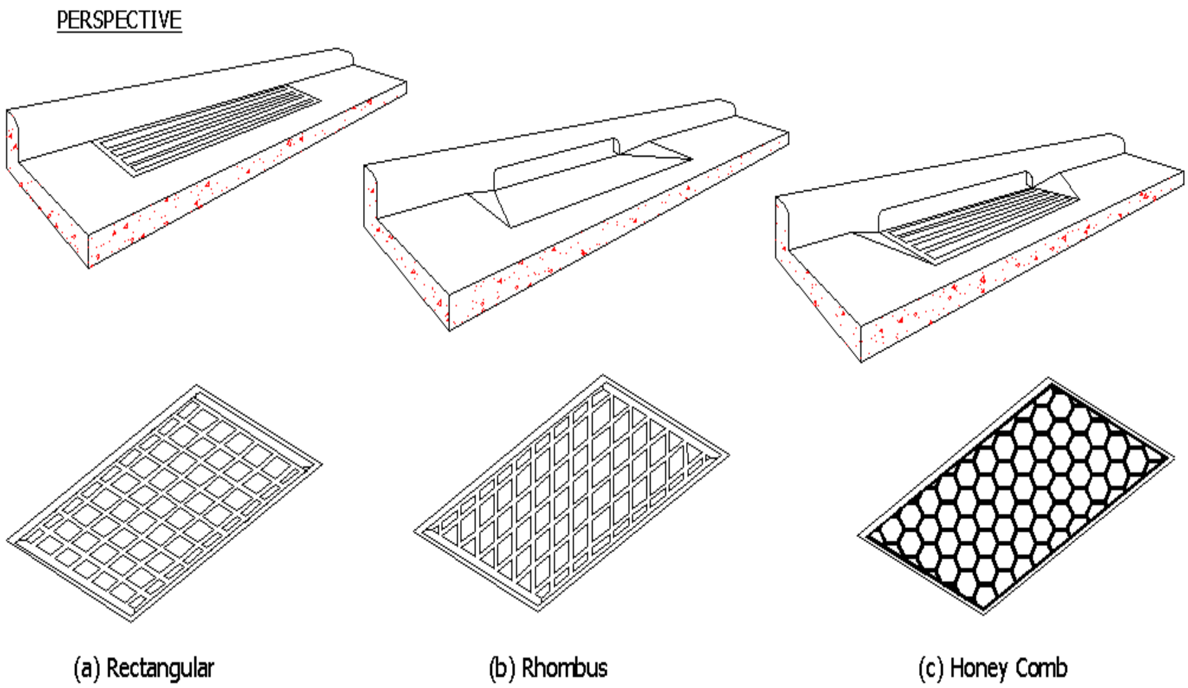


Figure 24.3 Bicycle-friendly Grates (based on Screen Opening)

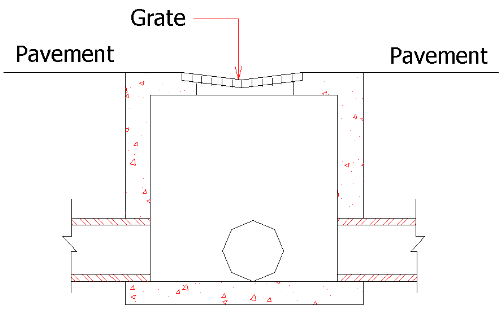
Other stormwater inlets are required to collect surface stormwater runoff in open space, reserves or swales where the flow is to be introduced to an underground pipe system. These grate inlets are known as 'field inlets'. A field inlet (Figure 24.4) is used in open space reserves, depressed medians and other locations away from pavement kerbs. Grated inlets can also be used in middle of the parking lots where kerbs are not required (Figure 24.5). A surcharge inlet is similar to a field inlet except that it is intentionally designed to permit surcharge for pressure relief in a pipe system.

Details of standard field inlets and surcharge inlets are shown in Standard Drawings SD F-2 and SD F-3, respectively.

However this Manual is not intended to preclude the adaption of other designs by a Local Authority. The Local Authority may determine which standard or other types of inlets are appropriate for its area. Standardisation of inlet designs within a local area is recommended in the interests of economic efficiency. If another design is adapted by a Local Authority, that Authority will need to obtain or derive inlet capacity Design Charts in place of those given in Appendix 24.A.

Figure 24.4 Grated Sump Field Inlet

(a) Perspective



(b) Section

Figure 24.5 Grated Parking Lot Inlet

24.2 PAVEMENT DRAINAGE

When rain falls on a sloped pavement surface, it forms a thin film of water that increases in thickness as it flows to the edge of the pavement. Factors which influence the depth of water on the pavement are the length of flow path, surface texture, surface slope, and rainfall intensity. A discussion of hydroplaning and design guidance for the following drainage elements are presented:

- Longitudinal pavement slopes
- Cross or transverse pavement slope
- Kerb and gutter design

Additional technical information on the mechanics of surface drainage can be found in Anderson et al (1995).

24.2.1 Hydroplaning

As the depth of water flowing over a roadway surface increases, the potential for hydroplaning increases. When a rolling tyre encounters a film of water on the roadway, the water is channelled through the tyre tread pattern and through the surface roughness of the pavement. Hydroplaning occurs when the drainage capacity of the tyre tread pattern and the pavement surface is exceeded and the water begins to build up in front of the tyre. As the water builds up, a water wedge is created and this wedge produces a hydrodynamic force which can lift the tyre off the pavement surface. This is considered as full dynamic hydroplaning and, since water offers little shear resistance, the tyre loses its tractive ability and the driver has a loss of control of the vehicle.

Hydroplaning is a function of the water depth, roadway geometries, vehicle speed, tread depth, tyre inflation pressures, and conditions of the pavement surface. It has been shown that hydroplaning can occur at speeds of 89 km/hr with a water depth of 2 mm. The hydroplaning potential of a roadway surface can be reduced by the following:

- Design the roadway geometries to reduce the drainage path lengths of the water flowing over the pavement. This will prevent flow build-up.
- Increase the pavement surface texture depth by such methods as grooving of cement concrete. An increase of pavement surface texture will increase the drainage capacity at the tyre pavement interface.
- The use of open graded asphaltic pavements has been shown to greatly reduce the hydroplaning potential of the roadway surface. This reduction is due to the ability of the water to be forced through the pavement under the tyre. This releases any hydrodynamic pressures that are created and reduces the potential for the tyre to hydroplane.
- The use of drainage structures along the roadway to capture the flow of water over the pavement will

reduce the thickness of the film of water and reduce the hydroplaning potential of the roadway surface.

The Design Acceptance Criteria for surface flow on roads (see Table 4.3 of Chapter 4) have been set to limit the potential for hydroplaning at high speeds, as well as the potential for vehicles to float or be washed off roads at lower speeds.

24.2.2 Longitudinal Slope

Experience has shown that the recommended minimum values of roadway longitudinal slope given in the AASHTO (1990) Policy on Geometric Design will provide safe, acceptable pavement drainage. In addition, the following general guidelines are presented.

- A minimum longitudinal gradient is more important for a kerbed pavement than for an unkerbed pavement since the water is constrained by the kerb. However, flat gradients on unkerbed pavements can lead to a spread problem if vegetation is allowed to build up along the pavement edge.
- Desirable gutter grades should not be less than 0.5 percent for kerbed pavements with an absolute minimum of 0.3 percent. Minimum grades can be maintained in very flat terrain by use of a rolling profile, or by warping the cross slope to achieve rolling gutter profiles.
- To provide adequate drainage in sag vertical curves, a minimum slope of 0.3 percent should be maintained within 15 metres of the low point of the curve.

24.2.3 Cross (Transverse) Slope

Table 24.1 indicates an acceptable range of cross slopes as specified in AASHTO's policy on geometric design of highways and streets. These cross slopes are a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slope for driver comfort and safety. These cross slopes represent standard practice. AASHTO (1990) should be consulted before deviating from these values.

Cross slopes of 2 percent have little effect on driver effort in steering or on friction demand for vehicle stability. Use of a cross slope steeper than 2 percent on pavement with a central crown line is not desirable. In areas of intense rainfall, a somewhat steeper cross slope (2.5 percent) may be used to facilitate drainage (Gallaway et al, 1979).

Where three (3) lanes or more are sloped in the same direction, it is desirable to counter the resulting increase in flow depth by increasing the cross slope of the outermost lanes. The two (2) lanes adjacent to the crown line should be pitched at the normal slope, and successive lane pairs, or portions thereof outward, should be increased by about 0.5 to 1 percent. The maximum pavement cross slope should be limited to 4 percent (refer to Table 24.1).

Additional guidelines related to cross slope are:

1. Although not widely encouraged, inside lanes can be sloped toward the median if conditions warrant.
2. Median areas should not be drained across travel lanes.
3. The number and length of flat pavement sections in cross slope transition areas should be minimised. Consideration should be given to increasing cross slope in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades.
4. Shoulders should be sloped to drain away from the pavement, except with raised, narrow medians and superelevations

Table 24.1 Normal Pavement Cross Slopes (FHWA, 1996)

Surface Type	Range in Rate of Surface Slope
High-Type Surface	
2 lanes	0.015 - 0.020
3 or more lanes, each direction	0.015 minimum; increase 0.005 to 0.010 per lane; 0.040 maximum
Intermediate Surface	0.015 - 0.030
Low-Type Surface	0.020 - 0.060
Shoulders	
Bituminous or Concrete	0.020 - 0.060
With Kerbs	$\geq$ 0.040

24.2.4 Kerb and Gutter

All roads in urban areas shall generally be provided with an integral kerb and gutter. The current practice of providing a kerb only on roads is generally not acceptable as there is no defined gutter to carry stormwater flows, and the road pavement will suffer damage from frequent inundation.

However, where the volume of gutter flow is negligible as in car parks and on the high side of single-crossfall roads, a kerb only is acceptable.

Kerbs are normally used at the outside edge of pavement for low-speed, and in some instances adjacent to shoulders on moderate to high-speed roads. They serve the following purposes:

- contain the surface runoff within the roadway and away from adjacent properties,
- prevent erosion on fill slopes,
- provide pavement delineation, and
- enable the orderly development of property adjacent to the roadway.

Gutters formed in combination with kerbs are available in 0.3 through 1.0 metre width. Gutter cross slopes may be same as that of the pavement or may be designed with a steeper cross slope, usually 80 mm per metre steeper than the shoulder or parking lane (if used). AASHTO geometric guidelines state that an 8% slope is a common maximum cross slope.

A kerb and gutter combination forms a triangular channel that can convey runoff equal to or less than the design flow without interruption of the traffic. When a design flow occurs, there is a spread or widening of the conveyed water surface. The water spreads to include not only the gutter width, but also parking lanes or shoulders, and portions of the travelled surface. Spread is what concerns the hydraulic engineer in kerb and gutter flow. The distance of the spread is measured perpendicular to the kerb face to the extent of the water on the roadway and is shown in Figure 24.6.

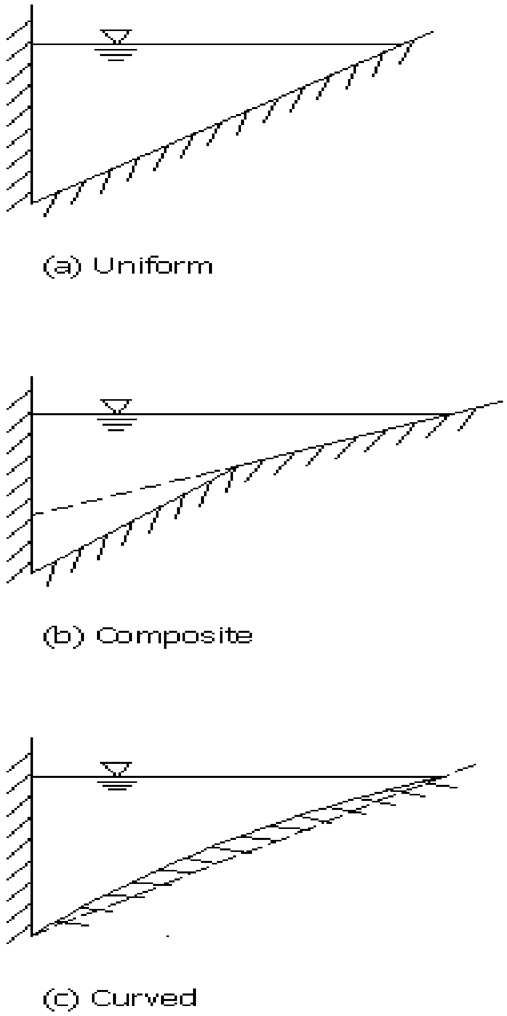


Figure 24.6 Gutter Sections

The kerb and gutter shall be a standard size to facilitate economical construction. Recommended standard details for road kerbs and gutters are shown in Standard Drawing No. SD F-4. The standard kerb height of 150 mm is based upon access considerations for pedestrians, vehicle safety including the opening of car doors, and drainage requirements.

If a local Authority decides to adapt a different standard, the design curves given in this Chapter will need to be adjusted accordingly.

24.2.5 Design Frequency and Spread

Two of the more significant variables considered in the design of pavement drainage are the frequency of the design event and the allowable spread of water on the pavement. A related consideration is the use of an event of lesser frequency to check the drainage design.

Spread and design frequency are not independent. The implications of the use of criteria for spread of one-half of a traffic lane is considerably different for one design frequency than for a lesser frequency. It also has different implications for a low-traffic, low-speed roads than for a higher classification roads. These subjects are central to the issue of pavement drainage and important to traffic safety.

(a) Selection of Design Frequency and Design Spread

The objective of pavement storm drainage design is to provide for safe passage of vehicles during the design storm event. The design of a drainage system for a kerbed pavement section is to collect runoff in the gutter and convey it to pavement inlets in a manner that provides reasonable safety for traffic and pedestrians at a reasonable cost. As spread from the kerb increase, the risks of traffic accidents and delays, and the nuisance and possible hazard to pedestrian traffic increase.

The process of selecting the ARI and spread for design involves decisions regarding acceptable risks of accidents and traffic delays and acceptable costs for the drainage system. Risks associated with water on traffic lanes are greater with high traffic volumes, high speeds, and higher road classifications.

A summary of the major considerations that enter into the selection of design frequency and design spread follows:

1. The classification of the road is a good point in the selection process since it defines the public's expectations regarding water on the pavement surface. Ponding on traffic lanes of high-speed, high-volume roadways is contrary to the public's expectations and thus the risks of accidents and the costs of traffic delays are high.

2. Design speed is important to the selection of design criteria. At speeds greater than 70 km/hr, it has been shown that water on the pavement can cause hydroplaning.
3. The intensity of rainfall events may significantly affect the selection of design frequency and spread. Risks associated with the spread of water on pavement is high in Malaysian conditions.

Other considerations include inconvenience, hazards and nuisances to pedestrian traffic. These considerations should not be minimised and in some locations such as in commercial areas, may assume major importance.

The relative elevation of the road and surrounding terrain is an additional consideration where water can be drained only through a storm drainage system, as in underpasses and depressed sections. The potential for ponding to hazardous depths should be considered in selecting the frequency and spread criteria and in checking the design against storm events of lesser frequency than the design event.

Spread on traffic lanes can be tolerated to greater widths where traffic volumes and speeds are low. Spreads of one-half of a traffic lane or more are usually considered a minimum type design for low-volume local roads.

The selection of design criteria for intermediate types of facilities may be the most difficult. For example, some arterials with relatively high traffic volumes and speeds may not have shoulders which will convey the design runoff without encroaching on the traffic lanes. In these instances, an assessment of the relative risks and costs of various design spreads may be helpful in selecting appropriate design criteria. Table 24.2 provides suggested minimum design frequencies and spread based on the types of road and traffic speed. Similar design criteria are also given in Chapter 4, Table 4.3.

The recommended design frequency for depressed sections and underpasses where ponded water can be removed only through the storm drainage system is a 50 year ARI. A 100 year ARI storm is used to assess hazards at critical locations where water can pond to appreciable depths.

(b) Selection of Major storm and Spread

A major storm should be used any time runoff could cause unacceptable flooding during less frequent events. Also, inlets should always be evaluated for a major storm when a series of inlets terminates at a sag vertical curve where ponding to hazardous depths could occur.

The frequency selected for the major storm should be based on the same considerations used to select the design storm, i.e., the consequences of spread exceeding that chosen for design and the potential for ponding. Where no significant ponding can occur, major storm are normally unnecessary.

Criteria for spread during the check event are :

1. one lane open to traffic during the major storm event
2. one lane free of water during the major storm event

These criteria differ substantively, but each sets a standard by which the design can be evaluated.

Table 24.2 Suggested Minimum Design Frequency and Spread (Adapted from FHWA, 1996)

Road Classification		Design Frequency	Design Spread
High Volume or Divided or Bi-directional	< 70 km/hr	10 year	1 m
	> 70 km/hr	10 year	No Spread
	Sag Point	50 year	1 m
Collector	< 70 km/hr	10 year	½ Lane
	> 70 km/hr	10 year	No Spread
	Sag Point	10 year	½ Lane
Local Streets	Low Traffic	5 year	½ Lane
	High Traffic	10 year	½ Lane
	Sag Point	10 year	½ Lane

24.3 LOCATING INLETS

24.3.1 General Requirements

The location and spacing of inlets on roads is governed in part by the need to provide safe, economical road drainage by limiting the amount of gutter flow. The design acceptance criteria for road flow is provided in Chapter 4, Table 4.3.

These criteria are based on pedestrian safety and vehicle stability. They assume that traffic will slow to a safe speed in the major flood when the road is flooded. They do not apply to expressways because ponding on expressways would cause a risk of vehicle aquaplaning. The design of expressway drainage is outside the scope of this Manual.

24.3.2 Gutter Flow

Many pavement drainage problems occur in Malaysia because of a failure to give due attention to gutter flow and inlets. In many cases gutters are poorly formed or absent, inlets are too widely spaced, and the design of the inlets is inadequate to capture gutter flow and convey it to the drainage system.

In particular, the common practice of forming a round or half round inlet at the entrance of a pipe is unacceptable because:

- the available inlet area is too small to be effective,
- the design is hydraulically inefficient,
- water must pond on the road to produce sufficient head available to force gutter flow into the inlet,
- it is prone to blockage, and
- when used on grades, gutter flow simply bypasses the inlet altogether.

It is vital that proper hydraulic design principles be applied to the design of stormwater inlets. These principles are discussed in this Chapter.

Parameters required to calculate gutter flow from the pavements are shown in Figure 24.7. Knowing those parameters, gutter flow capacity may be calculated by Izzard's equation given below (Izzard, 1946):

$$Q = \frac{3\ F_f}{8}\left[\left(\frac{Z_g}{n_g}\right)(d_g^{\ 8/3} - d_p^{\ 8/3}) + \left(\frac{Z_p}{n_p}\right)(d_p^{\ 8/3} - d_c^{\ 8/3})\right] S^{1/2} \quad (24.1)$$

where, subscripts *g* , *p* and *c* refer to the gutter, pavement and road crown, respectively. F_f is a flow correction factor, Z is the cross slope, S is the longitudinal slope and d is the runoff depth over the pavement. A Design Chart for gutter flow calculation is given in Appendix 24.A. Recommended values of Manning's roughness coefficient n and the Flow Correction Factor F_f for gutter flow are given in Table 24.3.

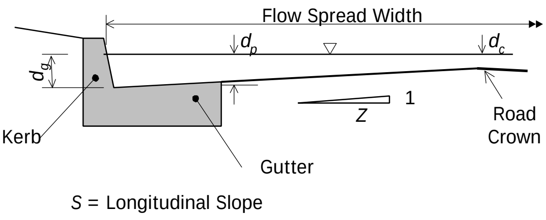


Figure 24.7 Kerb and Gutter, Showing Half Road Flow

Table 24.3 Manning's 'n' and Flow Correction Factor, F_f for Gutter Flow (QUDM, 1992)

Surface Type	n
Concrete	0.013
Hot mix asphaltic concrete	0.015
Sprayed seal	0.018
Kerb and Gutter type	F_f
Semi-mountable type	0.9
Barrier type	0.9

This form of the equation allows for the pavement and channel to have different roughnesses and/or different crossfalls. For the definition of terms in the equation refer to Figure 24.7. The face of the kerb is approximated as being vertical.

Using either Equation 24.1 or the Design Chart in Appendix 24.A, suitable limits for gutter flow can be determined. The average inlet spacing is then determined to ensure that this limit is not exceeded. A worked example of this calculation is provided in Appendix 24.B (based on AR&R, 1987).

Note that the inlet capacity of an inlet increases with increasing gutter flow. Therefore, provided the flow width limits are satisfied, it is an advantage to allow some bypass gutter flow on sloping roads to maximise the use of the inlet capacity (Sutherland, 1992).

24.3.3 Selection of Inlet Type

Kerb inlets on grade shall normally be type 'S' with a 2.4m long lintel as shown on Standard Drawing SD F-1. The capacity of these inlets is shown in Design Chart 24.2. Type 'M' or 'L' lintels may be used at sag points to provide additional capacity if space and kerb geometry permits.

A Type 'S' inlet may also be used:

- at changes in direction where entry of water is not essential (i.e. side entry may be sealed)

- in tight radius kerb returns where the length of a type M or L inlet is inappropriate
- as a field inlet

24.3.4 Inlet Spacing Calculation

Inlet spacing calculation uses the Rational Method to estimate discharge in the design storm. For simplicity each inlet subcatchment is assumed to be approximately rectangular as shown in Figure 24.8. If the subcatchments are not rectangular they should be replaced by equivalent rectangular subcatchments.

The average inlet spacing on grade is calculated so that allowable gutter flow is not exceeded, using the procedure shown in Figure 24.9.

A worked example for the calculation of inlet spacing on grade is given in Appendix 24.B3.

24.3.5 Location of Inlets

Illustrations showing the typical location of inlets for roads are given in Figure 24.10.

(a) General

Kerb inlets for all roadways shall be spaced such that gutter flow widths do not exceed the previously discussed limits. Inlets should also be located such that the quantity of gutter flow entering an intersection kerb return is minimised.

Inlets shall be provided:

- in the low points of all sags;
- on grades, with average spacing calculated in accordance with Section 24.3;
- at the tangent point of intersection kerb returns such that the width of gutter flow around the kerb return in the Minor Design Storm does not exceed 1.0m;
- immediately upstream of pedestrian crossings, access ramps, taxi or bus stops;
- immediately upstream of any reverse crossfall road pavement, where flow would be directed across the pavement;
- along the high side of islands or medians so as to meet the gutter flow width limitations in Section 24.3, and at the downstream end of the island or median to prevent gutter flow continuing onto the road pavement.

Inlets shall not be located on the curve at an intersection because of the risk they present to vehicles. Also, the structural design of a side inlet on a curve is much more complex.

Kerb inlets within an island or median strip should, where possible, be a normal inlet. However if the space available within a median strip is insufficient, a median drain design similar to Figure 3.10 of JKR "Guide to Drainage Design of Roads" can be used. Because this alternative is less hydraulically efficient, appropriate modifications shall be made to the inlet spacing. If the depth and velocity of gutter flow are within acceptable limits, a median opening may alternatively be used to allow runoff to flow to the downhill kerb drain.

Where sufficient width is available, grated inlets can be recessed into the kerb or island so that the grate does not project onto the road pavement. However this also reduces their effectiveness.

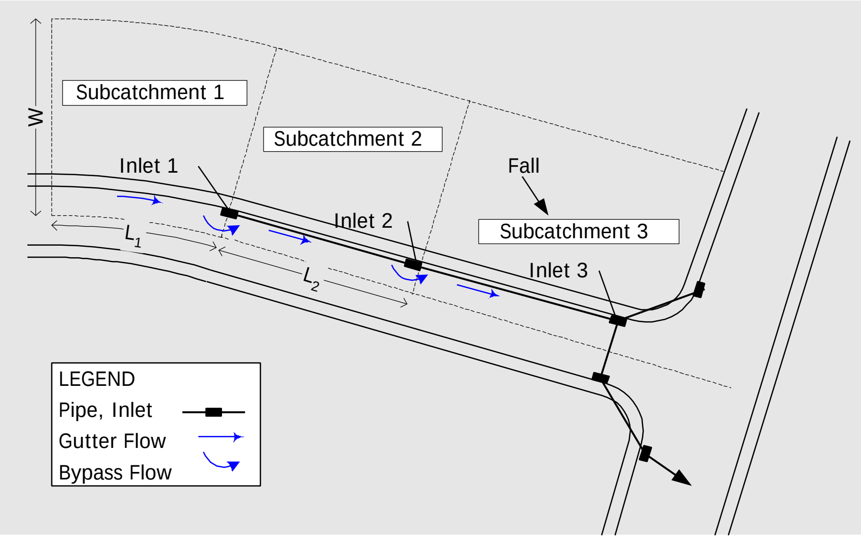


Figure 24.8 Calculation of Gutter Flow and Inlet Spacing on Grade

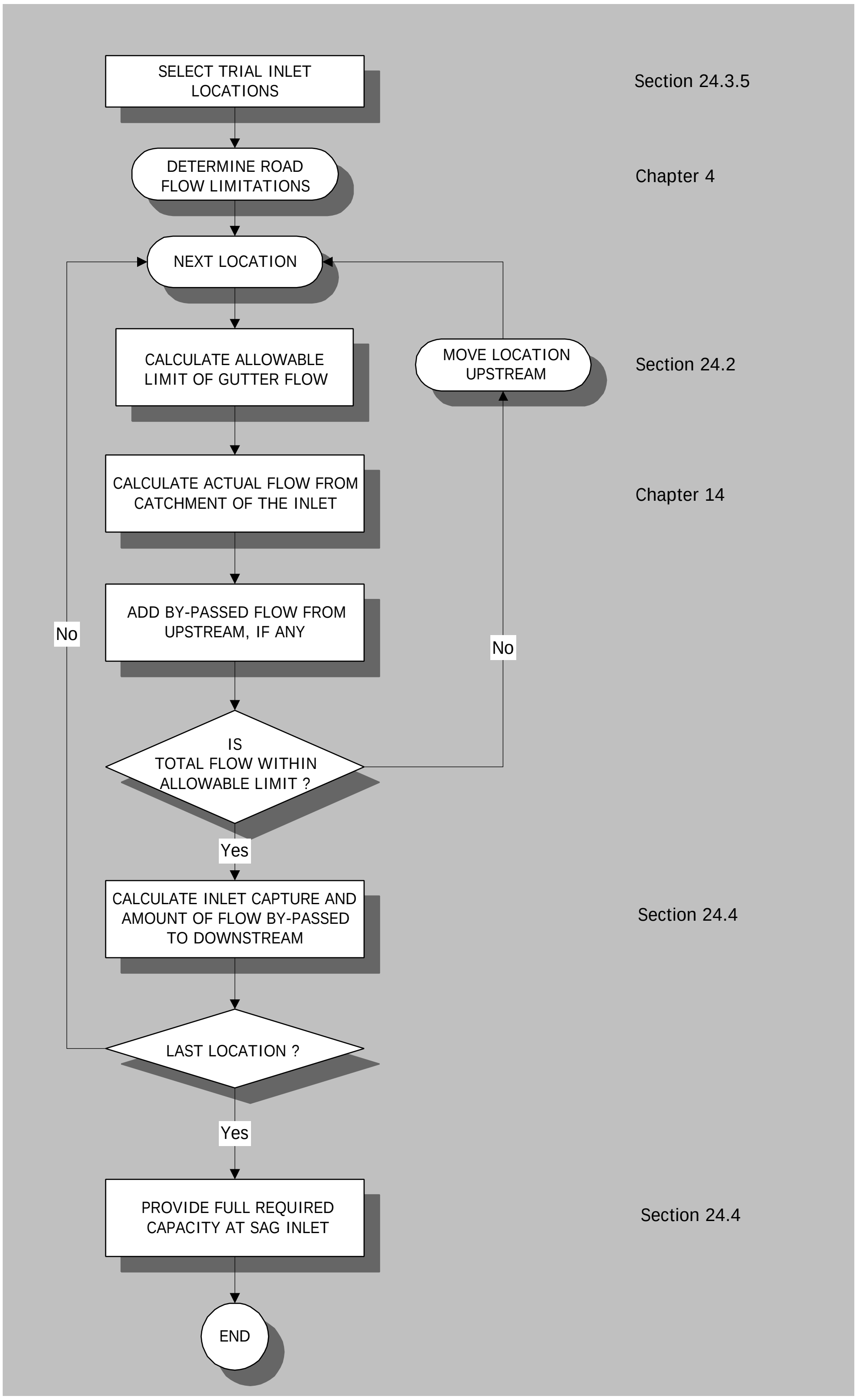


Figure 24.9 Flowchart for Calculation of Inlet Spacing

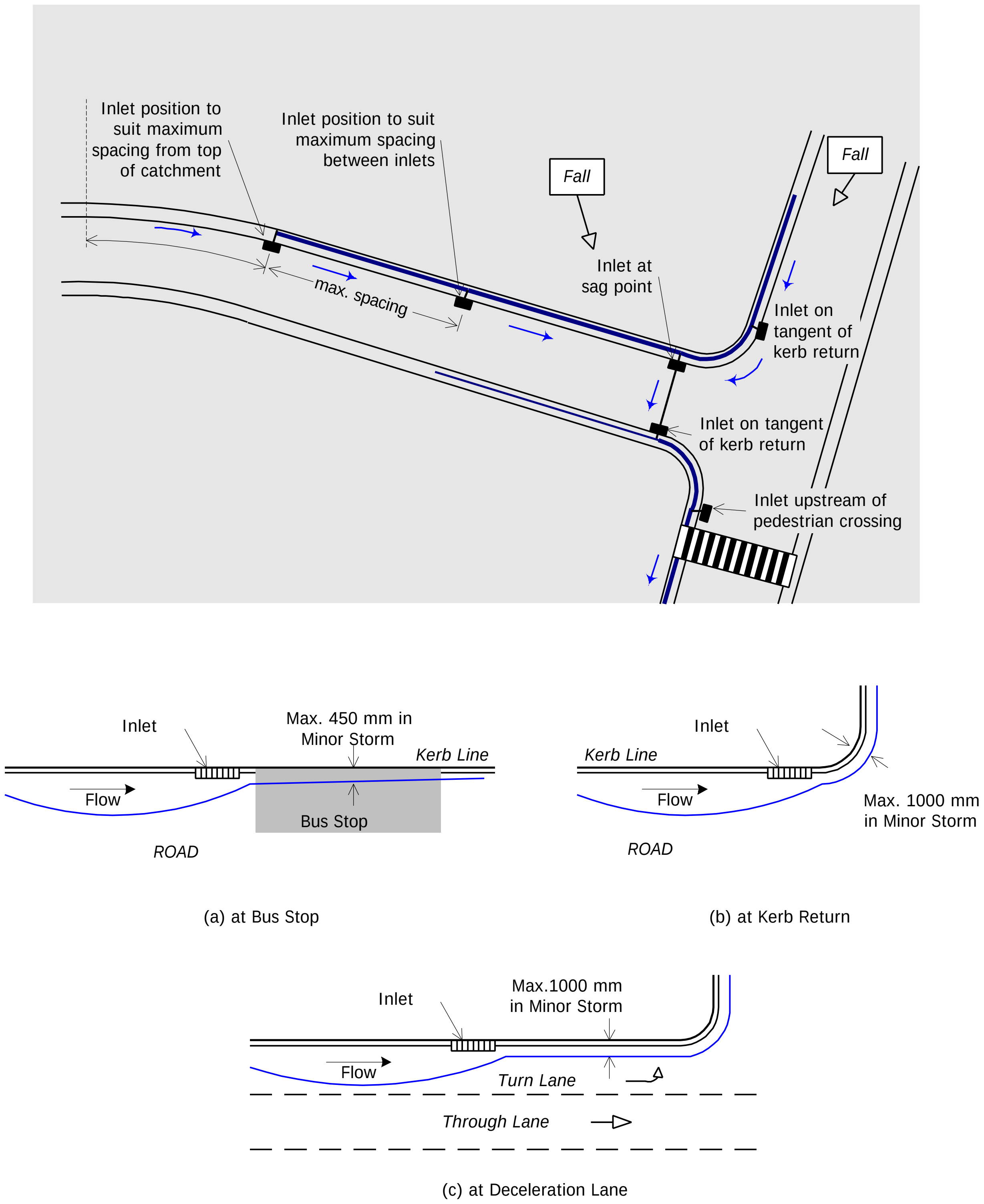


Figure 24.10 Typical Location of Inlets for Roads

Locations are also influenced by (AR&R, 1998):

- the positions of other utility services;
- the positions of driveways;
- superelevations and other changes to road cross-sections, which cause flow to cross roads;
- maintenance requirements, such as clear access; and
- the need to limit flow depths on the low side of roads below crest levels of driveways serving properties below road level.

(b) Inlets on Grade

Designers should be aware that the inlet capacity of pits on grade is controlled by the longitudinal grade and the road crossfall. Inlet Capacity charts for standard inlets are given in Appendix 24.A of this Chapter.

Bypass gutter flow from an upstream inlet must be accounted for in the design of the downstream inlet which receives the flow. A design procedure which satisfies this requirement is given in Chapter 16. There is no limit to the amount of gutter flow that may be bypassed, provided that the gutter flow restrictions in Section 24.3 are adhered to.

If the longitudinal grades of the kerbs approaching an intersection are steep, it may be necessary to check for the effect of flow super-elevation on the gutter flow spread around the kerb return.

(c) Inlets in Sags

Inlets in sags must have sufficient capacity to accept the total gutter flow reaching the inlet, including all bypass flows from upstream. Ponding of water at sags must be limited to the limits set in Section 24.3, particularly at intersections where turning traffic is likely to encounter ponded water.

(d) Inlets for Parking Lot

Parking lot inlets should be located outside of heavily traveled pedestrian areas (e.g. crosswalk, kerb ramps, and lead walks to the building and between parked vehicles). Inlets should be placed in areas where people can access their vehicles without stepping around the inlet. Figure 24.11 shows recommended placement of inlets in parking areas.

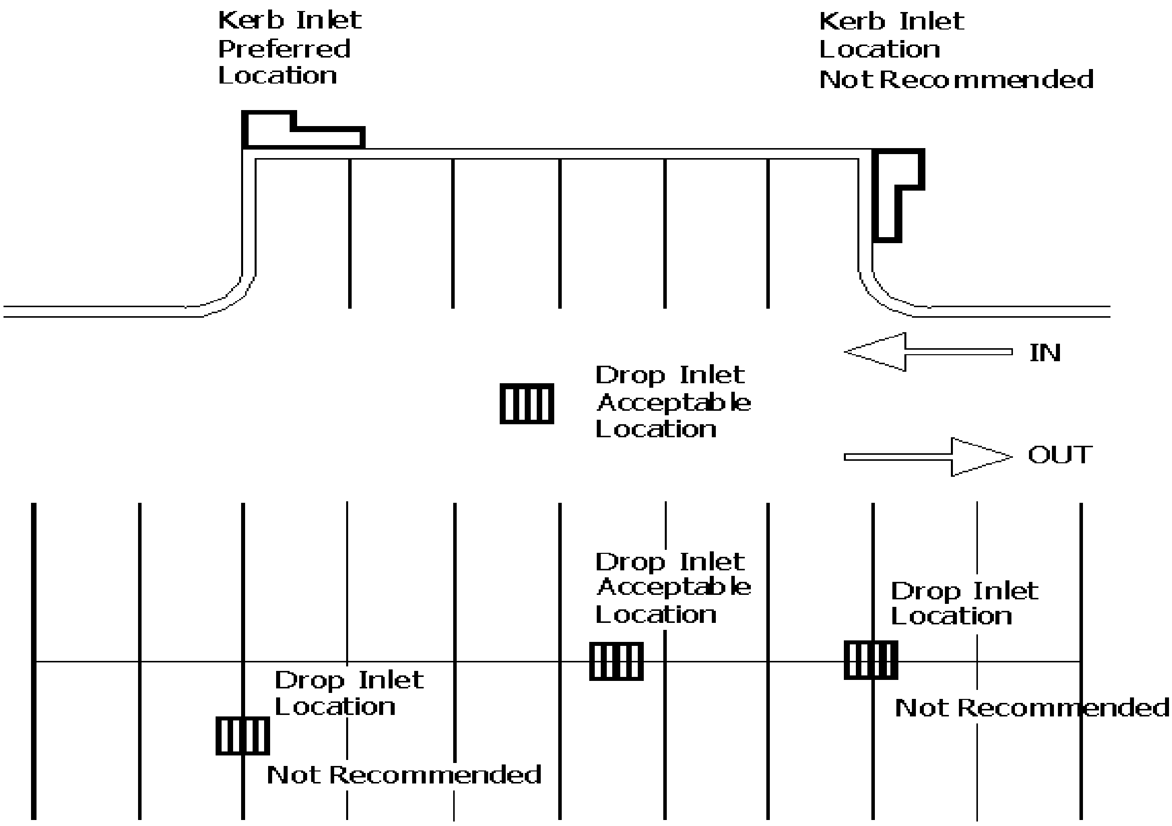


Figure 24.11 Typical Location of Inlets for Parking Lots

24.4 INLET CAPACITY CALCULATION

24.4.1 Allowance for Blockage

Inlet interception capacity has been investigated by several agencies and manufacturer of grates. Hydraulic tests on grate inlets and slotted inlets were conducted by Bureau of Reclamation for the U.S. Federal Highway Administration. Normally the longitudinal bars are vertical and the transverse bars (vanes) are fixed in different angle and orientation to get maximum hydraulic efficiency with minimum blockage from litters. Few typical arrangement of vanes in the grates are shown in Figure 24.12.

The design blockage allowance shall normally be 30% for an inlet on grade and 50% for a sag inlet or field inlet, unless otherwise directed.

The gutter flow required to be handled by the inlets shall be determined from network design calculations as described in Chapter 16.

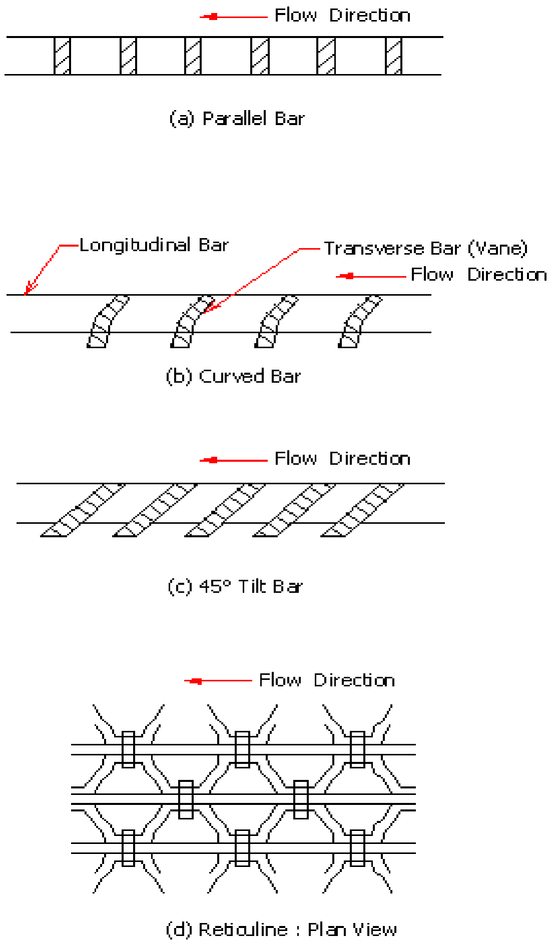


Figure 24.12 Typical Arrangement of Vanes for Grates

24.4.2 Combination Kerb Inlet

Combination kerb inlets can have 2.4 m, 3.6 m or 4.8 m long lintels (refer Standard Drawing SD F-1 types 'S', 'M' and 'L', respectively).

The inlet capacity of combination kerb inlets can be taken to be approximately equal to the sum of the kerb opening and grate capacities.

The *kerb opening capacity* depends on the inlet throat geometry (see Figure 24.13). The inlet throat acts as an orifice and the orifice flow equation applies (FHWA, 1984).

$$Q_t = 0.67hL\sqrt{2gd_o} \qquad (24.2)$$

where,

Q_t = flow through the inlet throat,

L = length of kerb opening,

d_o = effective head at centre of the orifice throat, and

h = orifice throat width

For inlets on grade, this theoretical capacity is reduced because of the tendency of fast-flowing water to bypass the inlet opening. The efficiency E of a kerb opening on grade is given as:

$$E = \frac{Q_i}{Q} \qquad (24.3)$$

where,

Q = total incoming flow through the gutter side

Q_i = flow captured by the inlet

The efficiency of an inlet on grade depends on the length of the opening, longitudinal slope, cross-fall, and whether there are any deflector bars to divert flow into the grate.

The *grate capacity* depends on pavement geometry, the direction and depth of flow and the grate configuration including the spacing and size of bars. For shallow depths, up to approximately 200 mm, the weir equation can be applied.

$$Q_G = F_B \times 1.66 \times L_e h^{3/2} \qquad (24.4)$$

where,

L_e = effective length of grate opening in the direction of flow,

F_B = blockage factor,

Q_G = grate capacity

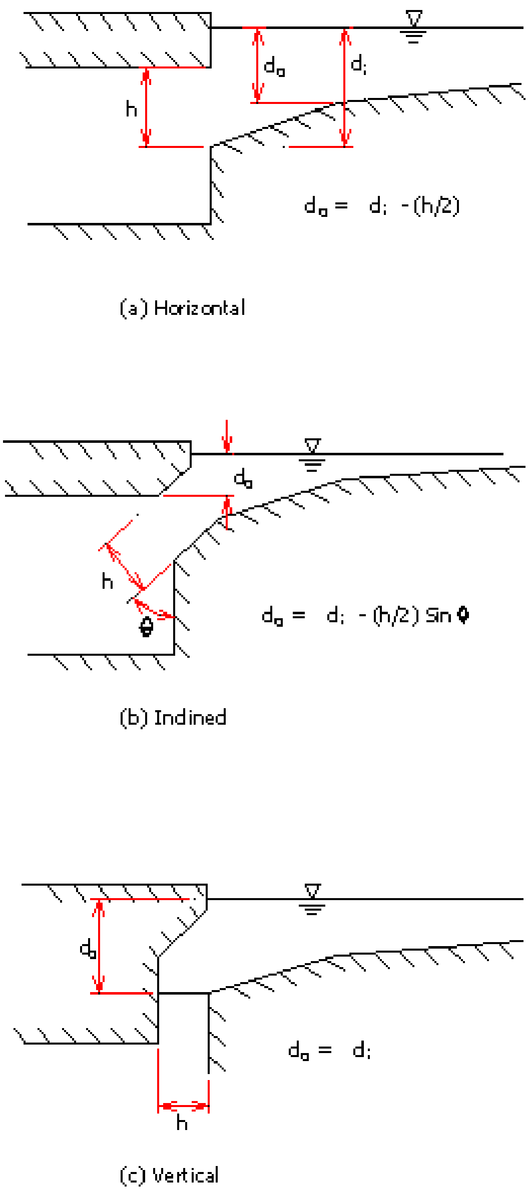


Figure 24.13 Throat Configuration of Kerb Opening Inlets (FHWA, 1996)

The effective length of the grate opening will depend on its width, the width of the grate bars and on the approach direction of flow. On grade, L_e = (W-W_b) and in sags, L_e = 2(W-W_b), where, W is the overall width of the grate and W_b is the total width of the longitudinal bars.

At depths greater than 200 mm, grate inlets in sags can function under orifice flow conditions as discussed in the following section.

In practice, it is not worthwhile or practical to perform these calculations for each inlet. Instead the capacity is estimated from empirical curves which should be based on prototype testing. Empirical inlet capacity design curves for combination kerb inlets are shown in Appendix 24.A. These curves based on QUDM (1992), show the combined capacity of the kerb and grate inlet. Allowance must be made for blockage as described in Section 24.4.1.

24.4.3 Field Inlet

The inflow capacity of a field inlet depends on the depth of water over the inlet. For shallow depths, up to approximately 200 mm, the flow will behave as a sharp-crested weir. For greater depths the inlet will become submerged and will behave as an orifice. The discharge/head characteristics of the two flow types are different (see Equations 24.5 and 24.6).

The capacity of the inlet should be checked using both formulae and the lesser inlet capacity adapted.

(i) under weir flow conditions

$$Q_G = F_B \times 1.66 \times L h^{3/2} \qquad (24.5)$$

(ii) under orifice flow conditions

$$Q_G = F_B \times 0.60 A_G \times \sqrt{(2gh)} \qquad (24.6)$$

where

A_G is the area of the grate opening.

24.4.4 Surcharge Inlets

Surcharge inlet structures shall be provided:

- where branch pipelines connect to low flow pipelines in floodways
- where there are shallow points in the system to form an emergency overflow relief path in times of acute hydraulic overload or blockage of the pipe system

The need for a surcharge inlet on pipelines shall be determined by Hydraulic Grade Line Analysis, as described in Chapter 25. If the HGL analysis indicates the likelihood of surcharge but the location does not permit surcharge water to flow away safely, a sealed manhole lid with a lock-down cover shall be provided.

To minimise the risk that the surcharge opening will become partially or fully blocked by debris and litter in the surcharged flow, the surcharge capacity of the inlet structure should be twice the total design inflow from all pipes connected to the structure. Details of the recommended standard surcharge inlet are shown in Standard Drawing SD F-3.

24.5 HYDRAULIC CONSIDERATIONS

The calculations given in this Chapter assume that there is no downstream constraint to inlet flows. This means that the capacity, level and grade of the pipe drain or downstream open channel is sufficient to convey the flow from the inlet(s).

In order to achieve this condition, the downstream system must be properly designed and have sufficient freeboard above the HGL. A number of older existing drainage systems do not meet this criterion. The designer of these systems is referred to Chapters 25 and 26, respectively. As a guideline it will be necessary to have at least 1.0 m height difference between the road level and the drain invert in order for the inlets to operate correctly.

In practice, the stormwater inlets and pipe drains must be designed together because the two systems interact:

- if there is insufficient inlet capacity the pipes will not flow full, and
- backwater effects from the pipe drainage system may reduce the effectiveness of the inlets, or cause them to surcharge instead of acting as inlets.

The complexity of these interactions is such that in all but the simplest situations, the design task is best handled by computer models. Some suitable computer models are described in Chapter 17.

24.6 CONSTRUCTION

24.6.1 Structural Adequacy

Stormwater inlets shall be constructed so that they are structurally sound and do not permit ingress of water through the walls or joints. Materials shall be resistant to erosion and corrosion. Where necessary, corrosion resistant cement shall be utilised.

24.6.2 Materials

Stormwater inlets may be constructed from:

- in-situ concrete,
- precast concrete,
- cement rendered brickwork, or
- mortared blockwork

The lintels for type S, M and L pits shall be precast, to comply with appropriate Malaysian or British standards.

24.6.3 Access Covers

The type of inlet cover shall be selected according to the following criteria:

- sealed solid top for structures in engineered waterways and other locations subject to hydraulic loads, for
  - inlet structures, or
  - surcharge structures (bolt-down locking shall be provided with stainless steel bolts to secure the cover and the seating ring to the structure)
- grated cover, for
  - inlets subject to traffic loadings, or
  - inlets in paved pedestrian areas

(a) Concrete cover

An ungrated inlet not subject to traffic loads or hydraulic surcharge shall be provided with a standard reinforced concrete seating ring and lid in accordance with Standard Drawing SD F-7.

The minimum size opening for access is 600x600 mm.

(b) Metal Grates

An inlet grate which will be subjected to vehicle loadings shall be designed to support those loads in accordance with the relevant Malaysian or British Standard.

Ductile iron covers shall be 'GATIC', or other proprietary design as approved in writing by the Local Authority.

24.6.4 Cover Levels

Stormwater inlet grates and access covers (if used) shall be set at the finished cover levels given in Table 24.4.

Table 24.4 Grating Cover Levels

Location	Grate/ Cover Level
Roads, other paved areas	Flush with finished surface
Footpaths and street verges	Flush with finished surface
Landscaped areas, parks	Flush with finished surface
Elsewhere	100 mm above surface to allow for topsoiling and grassing (see Note)

Note: Stormwater inlet tops shall be protected by placing fill against the top. The fill shall be graded down to natural surface at a maximum slope of 1 in 10.

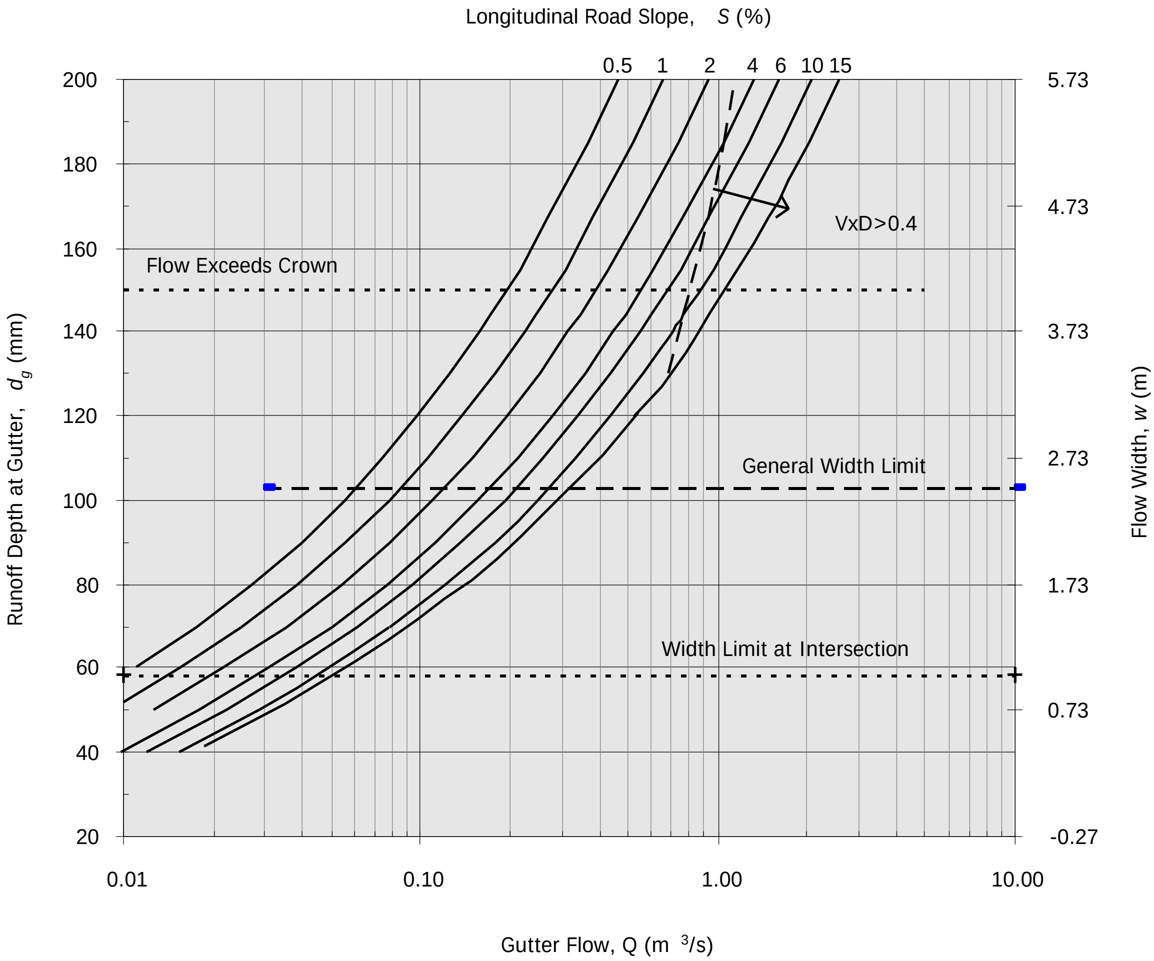
Where finished surfaces are steeper than 1(V):10(H), the access cover shall be level. An adjacent flat area shall be provided with sufficient space on which to place a removed cover.

24.7 MAINTENANCE

Inlets shall be checked and cleaned regularly, to prevent an accumulation of litter and debris, which may cause blockage. Sag locations are particularly susceptible to blockage.

Chapter 25 provides more detail maintenance required for maintenance required for drainage system, which involves inlets.

APPENDIX 24.A DESIGN CHARTS

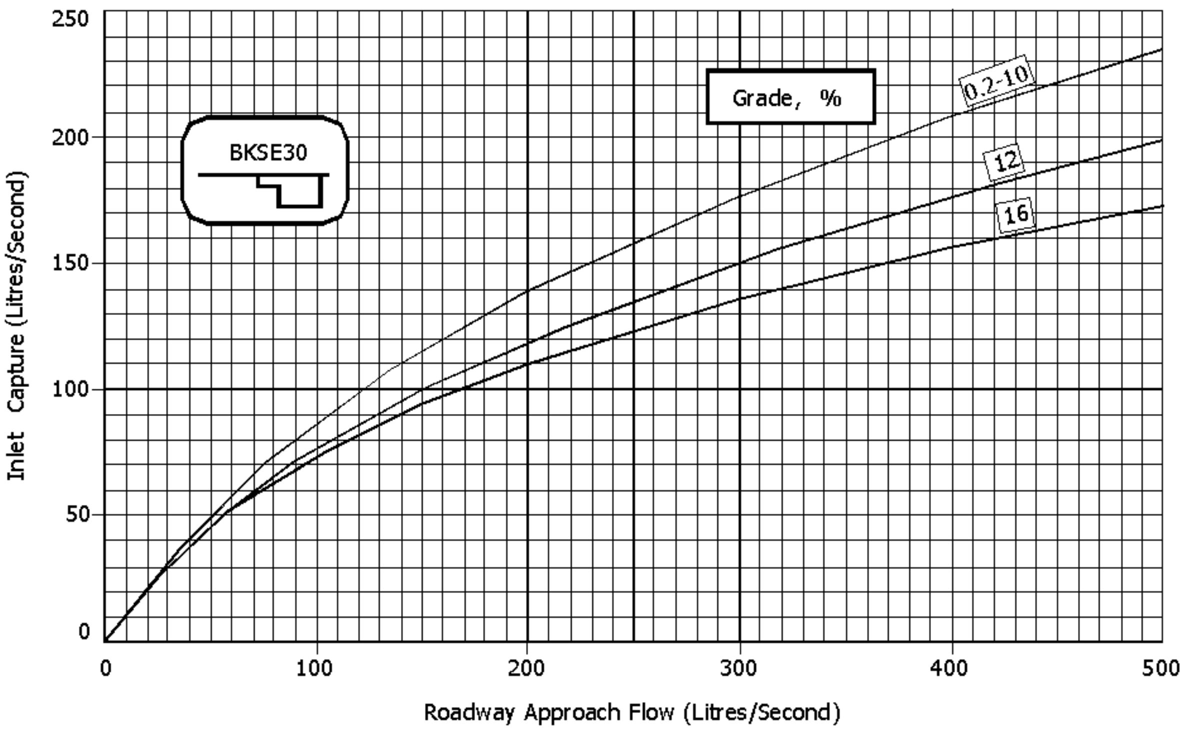


Design Chart 24.1 Gutter Flow using Izzard's Equation (QUDM, 1992)

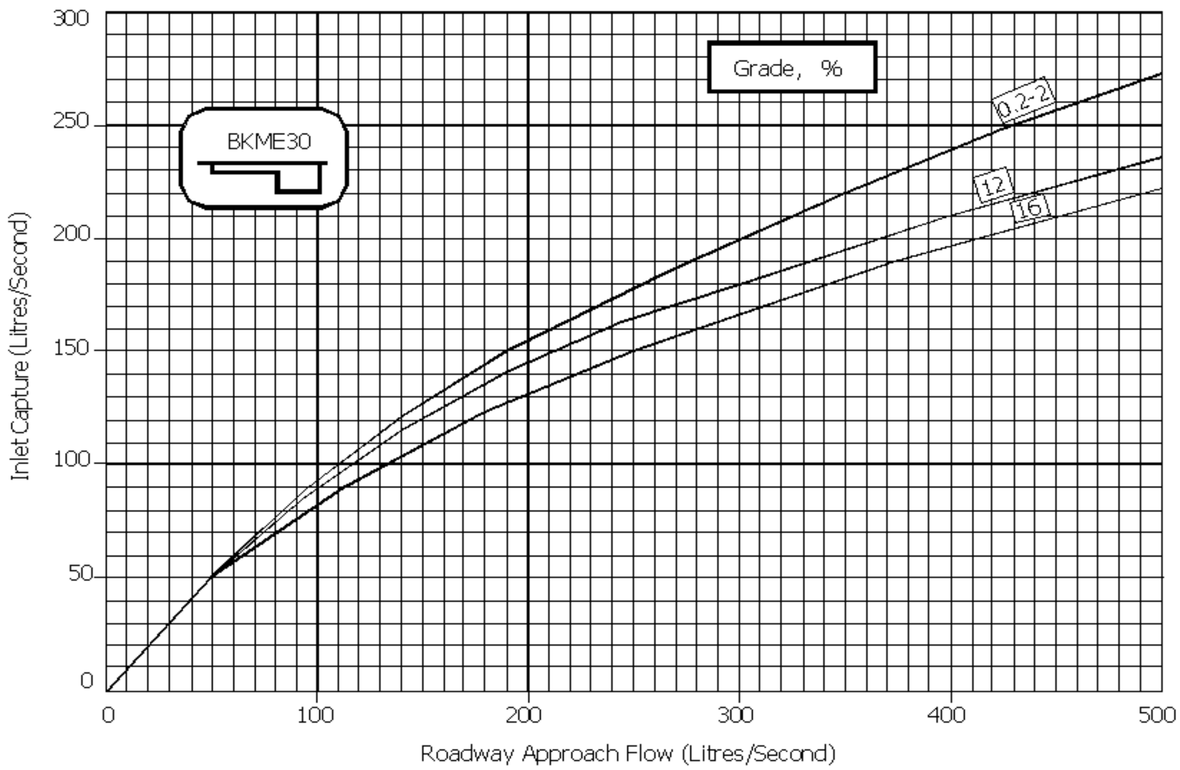
Based on Z_p and Z_g = 3% (Road Crossfall),
d_c = 0,
Barrier kerb type B1 (450mm),
n_p = 0.015,
n_g = 0.013

Note:

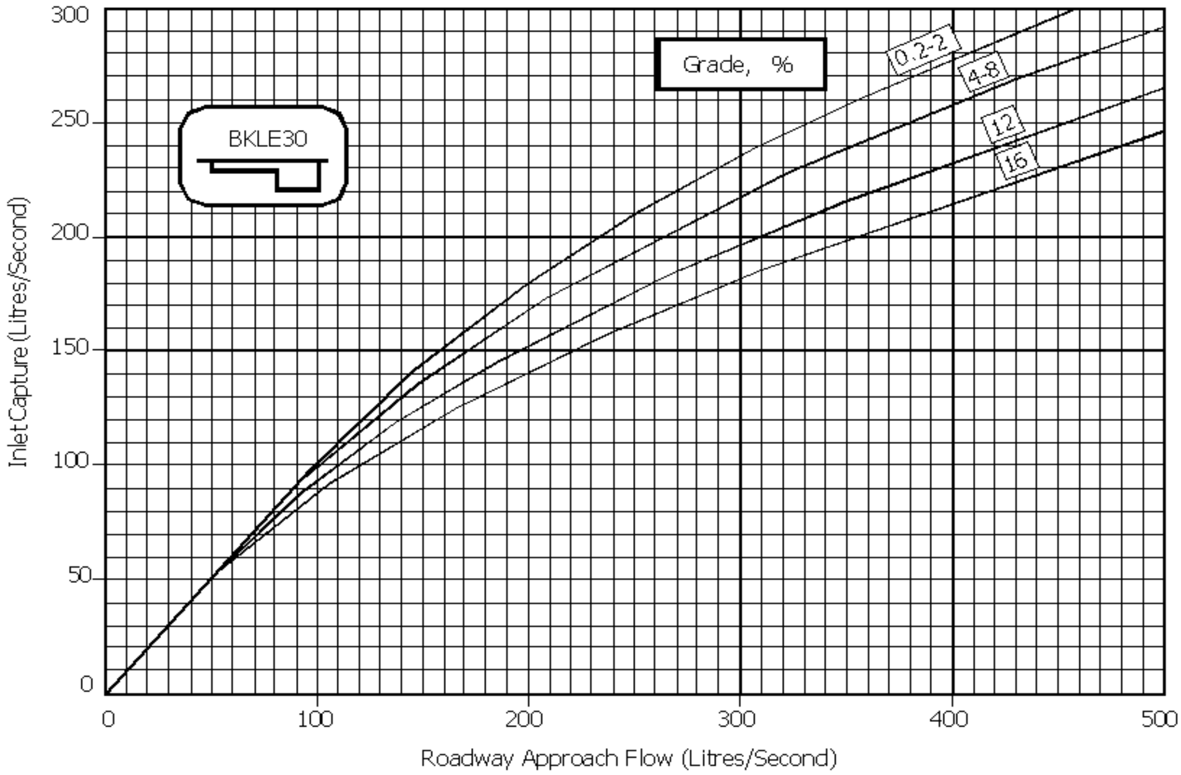
A number of similar set of curves can be prepared using different combination of variables in Izzard's Equation.



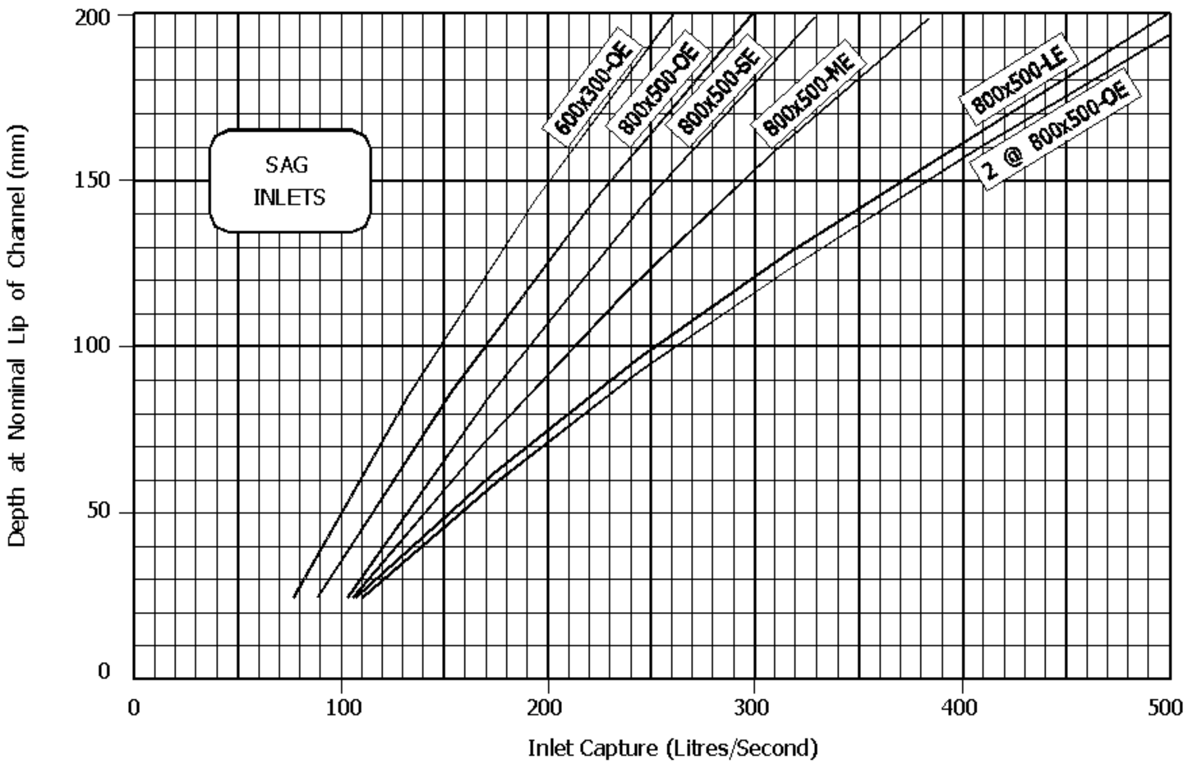
Design Chart 24.2 Combination Kerb Inlet Capacity – Type S (QUDM, 1992)



Design Chart 24.3 Combination Kerb Inlet Capacity – Type M (QUDM, 1992)



Design Chart 24.4 Combination Kerb Inlet Capacity – Type L (QUDM, 1992)



Inlet Capture vs Width of Ponding

1:30 Crossfall

Inlet Capture Litres/Second						
Flow Width m	600 x300 0E	800 x300 0E	800 x500 SE	800 x500 ME	800 x500 LE	2 @ 800 X500
2.0	103	118	135	144	159	165
2.5	118	136	155	168	191	198
3.0	135	155	175	193	226	234
3.5	152	174	196	219	262	271
4.0	169	194	217	245	300	310
4.5	187	215	240	273	339	351
5.0	206	237	263	302	380	394
5.5	226	259	287	332	423	438
6.0	246	282	312	363	467	483
6.5	266	306	337	395	512	530
7.0	287	330	363	427	559	579
7.5	309	355	389	460	607	629
8.0	331	381	416	495	657	680

1:40 Crossfall

Inlet Capture Litres/Second						
Flow Width m	600 x300 0E	800 x300 0E	800 x500 SE	800 x500 ME	800 x500 LE	2 @ 800 X500
2.0	91	104	120	126	134	139
2.5	102	117	134	143	157	163
3.0	113	130	148	160	181	188
3.5	125	144	163	179	206	214
4.0	138	158	179	197	232	241
4.5	150	173	194	217	259	269
5.0	164	188	210	237	288	298
5.5	177	203	227	258	317	328
6.0	191	219	244	279	347	359
6.5	205	236	262	300	377	391
7.0	220	252	279	323	409	424
7.5	234	269	298	345	442	457
8.0	250	287	316	369	475	492

Inlet Captures Shown Above are Independant of Capture by the Grated Area. The 2 @ 800X500 Configuration Requires Two Standard Inlets Connected by 2.4 m Length of Pipe.

Design Chart 24.5 Sag Inlet Capacity

APPENDIX 24.B WORKED EXAMPLE

24.B.1 Spacing of Inlets (Half Road Width)

Problem:To determine inlet spacing to cater runoff from half road catchment in Ipoh, Perak. Following data are given:

t_c	=	5 minutes
Rainfall intensity, $^5I_5$	=	300 mm/hr
Half road width	=	9 m
Longitudinal slope	=	0.5 %
Cross slope	=	3%

The minor system design = 5 year ARI

The outer lane is a through lane, W < 1.5m (Table 4.3)

Solution:

1) From Design Chart 14.3, C = 0.91 [Category (1)],

From Equation 14.7;

Q_{Road} = (C x $^5I_5$ x A)/360

= 0.91 x 300 x (9 x L_1 x 10^{-4})/360

= 0.000683 L_1

where L_1 is the length of gutter flow in the upstream subcatchment.

2) Calculate the allowable limit of gutter flow.

Using the Design Chart 24.1 and W = 1.5 m;

Q = 0.018 m^3/s

= 18 L/s and $V \times D$ is less than 0.4 m/s.

Therefore, spacing for the first inlet is,

L_1 = 0.018 / 0.000683

= 26.3 m ≈ 26 m

3) Use a Type 'S' inlet as recommended in Section 24.4.3. Refer to Design Chart 24.2 for a Type 'S' inlet (BKSE30). With a gutter approach flow of 18 L/s, the inlet capture is 18 L/s is giving a capture efficiency of 100 %.

Therefore, bypass gutter flow is zero and the inlet spacing to be adapted is 26 m.

24.B.2 Spacing of Inlets (Combined Catchment and Road)

Problem: Figure 24.B1 shows an idealised catchment and minor road system in Ipoh. In this case the surface catchment drains to a gutter with a uniform longitudinal slope of 2%. Determine the maximum permissible inlet spacing from residential/road catchment combined width of 45 m (half road width is 9 m). Time of concentration is 15 minutes and lumped runoff coefficient for the combined catchment is 0.85. Manning n for pavement, n_p = 0.015 (hot-mix asphalt pavement), and for gutter, n_g = 0.013 (concrete kerb and gutter). Road cross slope is 3%.

Solution: The minor storm is taken to be 5 year ARI (Table 4.1). Each subcatchment is approximately rectangular so area, $A = W \times L$. Time of concentration is assumed as 15 minutes.

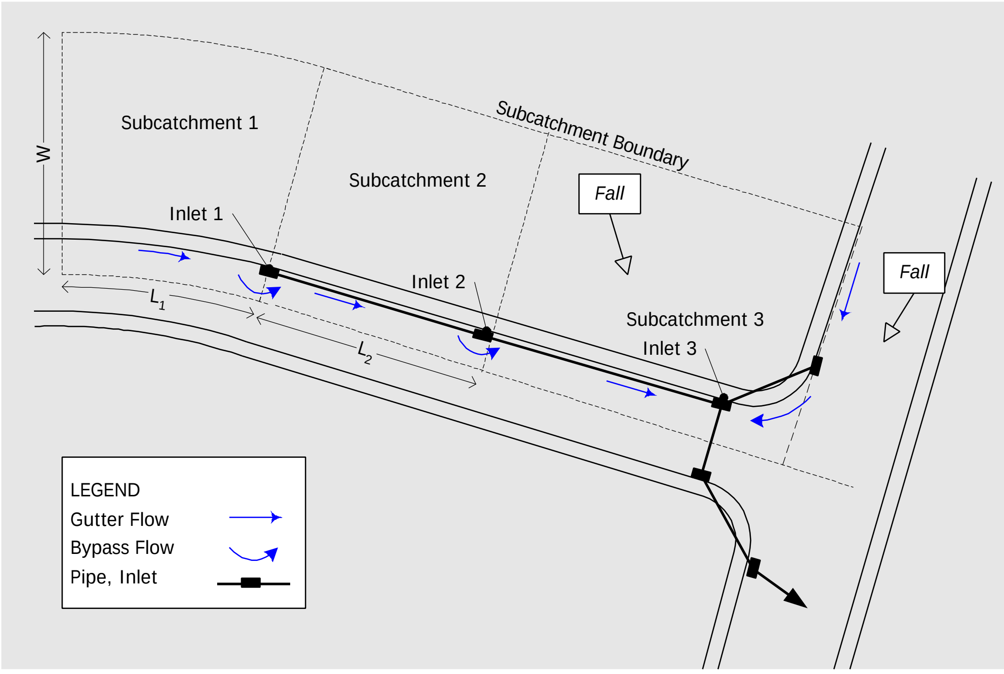


Figure 24.B1 Example for Catchment and Road Drainage

1) Adapt I = 175 mm/hr from Ipoh IDF data for 5 year ARI, 15 minute duration storm,

$Q_{combined}$ = $C.I.A.$ / 360

= 0.85 x 175 x (45 x L_1 x 10^{-4})/360

= 0.001859 L_1 where L_1 is the length of gutter flow in the first upstream subcatchment

2) Calculate the allowable limit of gutter flow.

For a minor road the allowable flow width is 2.5 m (Table 4.3). Note that the cross-fall is 3% and the runoff depth at gutter, d_g is given by 0.03 x 2.5 = 0.075 m. So, the flow will not overtop the kerb. Using Design Chart 24.1 in Appendix 24.A for S = 2%, n_p = 0.015 (hot-mix asphalt pavement), spread of 2.5 m width, and n_g = 0.013 (concrete kerb and gutter). The limiting gutter (half-road) flow based on flow not exceeding the road crown is:

Q = 170 litres per second

= 0.17 m^3/sec with $V \times D$ is within the allowable limit of 0.4 m/sec.

or 0.001859 L_1 = 0.17

Therefore, L_1 = 0.17 / 0.001859

= 91 m

As in the previous example, adapt type 'S' inlet. Determine the capture efficiency on a 2% slope.

3) Capture efficiency of a Type S inlet.

Use the Design Chart 24.2. With a gutter approach flow of 170 L/s, the inlet capture is 125 L/s giving a capture efficiency of about 73%.

Therefore, bypass gutter flow = 170 – 125

= 45 L/s

= 0.045 m^3/s.

4) This bypass gutter flow reduces the capacity of the next and subsequent inlets to accept inflow from their own subcatchments. The spacing required between subsequent inlets is given by:

$$L_1 = (0.170 - 0.045) / 0.001859$$
$$= 67 \text{ m}$$

5) For design purposes, adapt a maximum inlet spacing of 70 m. The adapted design is shown in figure below.

Note: This example ignores the fact that roof drainage would normally be connected directly to the piped drainage system, therefore the result is likely to be conservative.

24.B.3 Inlet Capacity Calculation

Problem: Determine the inlet capacity and analyse the hydraulics of Line 3B, part of an open drainage system for a similar idealised catchment in Ipoh (Figure 24.B2). Inlet numbers 3A/5P, 3A/10P,3B/1P and 3B/2P are Type-S. Inlets 3A/7P and 3A/8P are Type-M or L (depends on capacity required). The road has a uniform longitudinal slope of 2%.

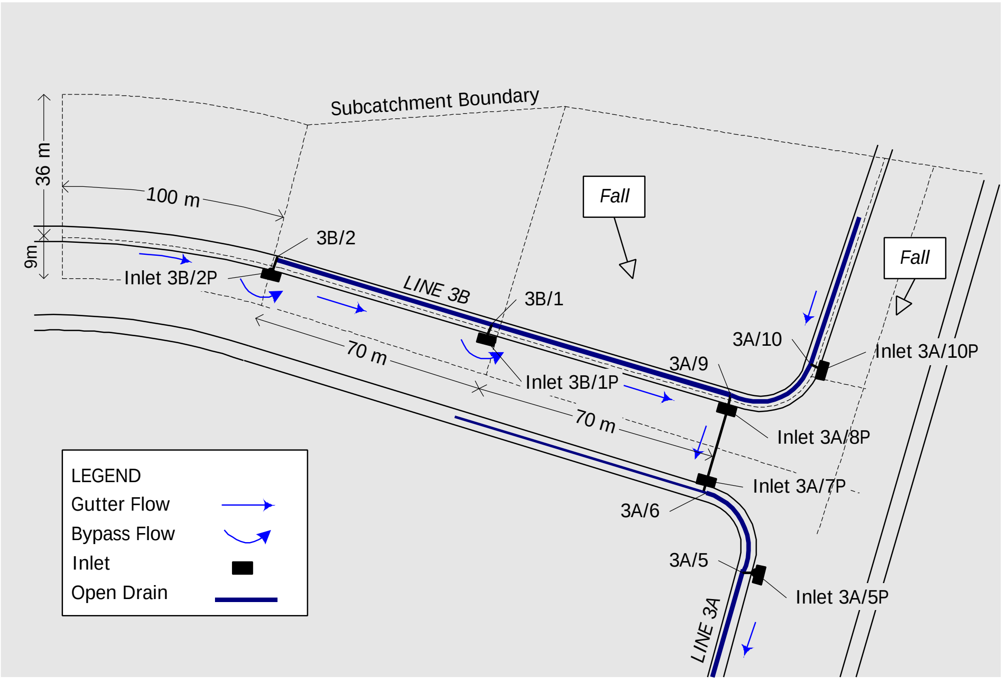


Figure 24.B2 Example for Road and Catchment Drainage to an Open Drain

Solution:

1) The required calculations are tedious to perform by hand. As such, RatHGL software is used for the analysis. The RatHGL network layout is shown in Figure 24.B3:

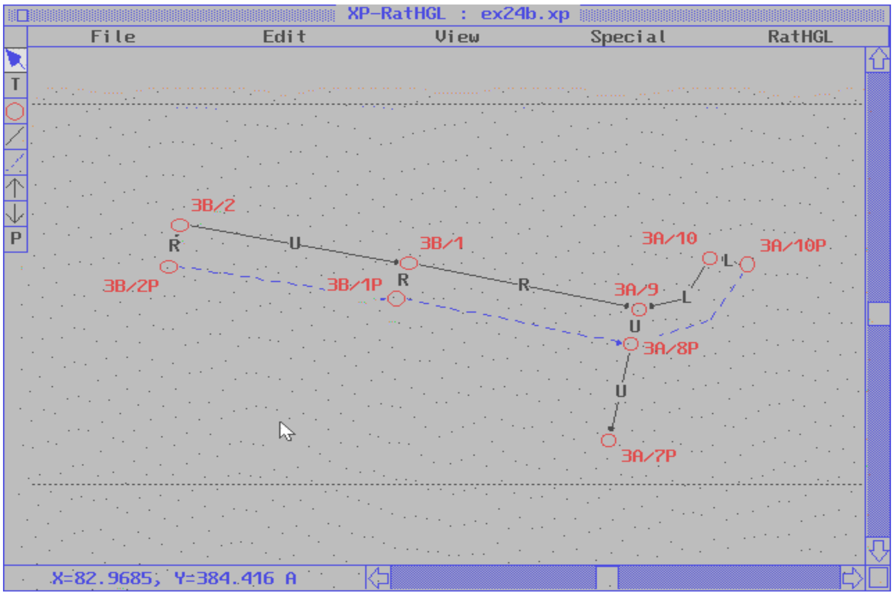


Figure 24.B3   Example for Road and Catchment Drainage to an Open Drain

2) The preliminary design shown above was prepared based on calculations similar to Appendix 24.B.2, taking care to account for the catchment areas draining to each section of drain. The analysis is performed with Ipoh rainfall IDF data.

   - Runoff coefficients 0.90 (impervious), 0.60 (pervious)
   - inlet rating curve used for Type S, M and L inlets on grade as mentioned in the problem. For nodes on the open drain, the capacity is set to a large value (5 m^3/s) so that there is no constraint on inflow.

3) Hydrology input data and results for the network in the 5 year and 100 year ARI storms are shown in Table 24.B1.

4) Hydraulic input data and hydraulic grade line results for the network in the 5 year and 100 year ARI storms are shown in Table 24.B2.

Table 24.B1 Hydrologic Input and Results of Network Analysis for 5 and 100 year ARI Storms

NODE NAME	NODE DESCRIPTION	CATCHMENT NUMBER	RETURN PERIOD	TIME OF CONCENTRATION – IMPERVIOUS – METHOD	IMPERVIOUS – LENGTH (m)	IMPERVIOUS – SLOPE (%)	IMPERVIOUS – ROUGHNESS	IMPERVIOUS – Tc (min)	PERVIOUS – METHOD	PERVIOUS – LENGTH (m)	PERVIOUS – SLOPE (%)	PERVIOUS – ROUGHNESS	PERVIOUS – Tc (min)	AREA – TOTAL (ha)	AREA – IMPERVIOUS (ha)	FLOWS – TOTAL FLOW (cms)	PIPE FLOW (cms)	SURFACE FLOWS – CATCHMENT CONTRIBUTION (cms)	TOTAL GUTTER FLOW (cms)	CAPTURED FLOW (cms)	PRIMARY PATH – DESTINATION	PRIMARY PATH – FLOW (cms)	SECONDARY PATH – DESTINATION	SECONDARY PATH – FLOW (cms)	AL AREA – CRITICAL AREA Tc (min)
3B/2P	-	1	5	Kinematic	100.000	2.00	0.015	3.21	Constant	-	-	-	5.00	0.090	0.090	0.053	0.037	0.053	0.053	0.037	3B/1P	0.016	3B/2	0.000	3.211
-	-	-	100	-	-	-	-	2.76	-	-	-	-	5.00	-	-	0.077	0.050	0.078	0.078	0.051	3B/1P	0.027	3B/2	0.000	2.763
3B/2	-	1	5	Constant	-	-	-	2.00	Kinematic	150.000	2.000	0.025	5.69	0.360	0.180	0.198	0.198	0.164	0.164	0.164	3B/1	0.000	-	0.000	6.689
-	-	-	100	-	-	-	-	2.00	-	-	-	-	4.89	-	-	0.287	0.287	0.241	0.241	0.241	3B/1	0.000	-	0.000	5.888
3B/1P	-	1	5	Kinematic	70.000	2.00	0.015	2.58	Constant	-	-	-	5.00	0.063	0.063	0.054	0.037	0.038	0.055	0.038	3A/8P	0.017	3B/1	0.000	2.578
-	-	-	100	-	-	-	-	2.22	-	-	-	-	5.00	-	-	0.082	0.053	0.055	0.083	0.053	3A/8P	0.029	3B/1	0.000	2.219
3B/1	-	1	5	Constant	-	-	-	2.00	Kinematic	100.000	2.000	0.025	4.41	0.315	0.105	0.356	0.356	0.130	0.130	0.130	3A/9	0.000	-	0.000	7.856
-	-	-	100	-	-	-	-	2.00	-	-	-	-	3.79	-	-	0.517	0.517	0.191	0.191	0.191	3A/9	0.000	-	0.000	7.055
3A/10P	-	1	5	Kinematic	50.000	2.00	0.015	2.10	Constant	-	-	-	5.00	0.050	0.050	0.030	0.021	0.030	0.030	0.021	3A/8P	0.009	3A/10	0.000	2.098
-	-	-	100	-	-	-	-	1.81	-	-	-	-	5.00	-	-	0.044	0.031	0.044	0.044	0.031	3A/8P	0.013	3A/10	0.000	1.807
3A/10	-	1	5	Constant	-	-	-	2.00	Kinematic	50.000	2.000	0.025	2.87	0.050	0.025	0.044	0.045	0.024	0.024	0.024	3A/9	0.000	-	0.000	3.870
-	-	-	100	-	-	-	-	2.00	-	-	-	-	2.47	-	-	0.065	0.065	0.035	0.035	0.035	3A/9	0.000	-	0.000	3.470
3A/9	-	1	5	Constant	-	-	-	2.00	Kinematic	120.000	1.500	0.025	5.41	0.560	0.140	0.616	0.616	0.231	0.231	0.231	3A/8P	0.000	-	0.000	6.411
-	-	-	100	-	-	-	-	2.00	-	-	-	-	4.65	-	-	0.891	0.892	0.339	0.339	0.339	3A/8P	0.000	-	0.000	5.650
3A/8P	-	1	5	Kinematic	70.000	2.00	0.015	2.58	Constant	-	-	-	5.00	0.063	0.063	0.673	0.674	0.035	0.059	0.059	3A/7P	0.000	-	0.000	6.457
-	-	-	100	-	-	-	-	2.22	-	-	-	-	5.00	-	-	0.980	0.981	0.051	0.090	0.090	3A/7P	0.000	-	0.000	5.696
3A/7P	-	-	5	-	-	-	-	-	-	-	-	-	-	-	-	0.672	-	0.000	0.000	0.300	-	0.000	-	0.000	6.562
-	-	-	100	-	-	-	-	-	-	-	-	-	-	-	-	0.978	-	0.000	0.000	0.300	-	0.000	-	0.000	5.803

INTENSITY (mm/hr)	EQ. IMPERVIOUS AREA (ha)
237.442	0.081
345.905	0.081
218.934	0.351
320.798	0.351
240.812	0.057
350.271	0.057
212.724	0.628
311.425	0.628
243.368	0.045
353.586	0.045
233.937	0.082
340.225	0.082
220.412	1.045
322.712	1.039
220.169	1.102
322.342	1.095
219.608	1.102
321.485	1.095

HYDRAULIC DATA & RESULTS					CONDUIT DATA															DATA AT UPSTREAM MANHOLE						
																					MANHOLE LOSSES					
UPSTREAM PIT NAME	DOWNSTREAM PIT NAME	CONDUIT SHAPE	NUMBER OF CONDUIT	RETURN PERIOD	TOTAL LENGTH	ROUGHNESS k	DIAMETER OR HEIGHT	HYDRAULIC AREA	WETTED PERIMETER	PIPE CLASS	UPSTREAM INVERT	DOWNSTREET INVERT	CONDUIT GRADE	UPSTREAM HGL	DOWNSTREAM HGL	INTER NODAL LOSS	FRICTION GRADE	FLOW	VELOCITY	PIT TYPE	K_u	K_w	INLET TYPE	INLET INFLOW	PONDING LEVEL LIMIT	MANHOLE COVER LEVEL
					m	mm	m	m²	m		m	m	%	m	m			cms	m/s					cms	m	m
3B/2P	3B/2	Circular	1	5	3.000	0.60	0.300	0.071.	0.942	1	18.800	18.750	1.670	19.141	19.136	0.00	0.115	0.037	0.524	Type 2	7.194	7.194	BKSE30	0.037	20.200	19.800
-	-	-	-	100	-	-	-	-	-	-	-	-	-	19.262	19.254	-	0.211	0.050	0.714	-	6.379	6.379	-	0.051	-	-
3B/2	3B/1	Other	1	5	70.000	0.60	1.000	0.500	2.400	1	18.600	17.200	2.000	18.909	17.960	0.00	0.211	0.198	1.507	Type 6	1.964	2.657	-	0.164	20.100	19.800
-	-	-	-	100	-	-	-	-	-	-	-	-	-	18.977	18.136	-	0.218	0.287	1.665	-	1.963	2.657	-	0.241	-	-
3B/1P	3B/1	Circular	1	5	5.000	0.60	0.300	0.071	0.942	1	17.400	17.350	1.000	17.966	17.960	0.00	0.118	0.037	0.530	Type 2	5.817	5.817	BKSE30	0.038	18.800	18.400
-	-	-	-	100	-	-	-	-	-	-	-	-	-	18.148	18.136	-	0.236	0.053	0.756	-	4.575	4.575	-	0.053	-	-
3B/1	3A/9	Other	1	5	70.000	0.60	1.000	0.500	2.400	1	17.200	15.800	2.000	17.620	16.517	0.00	0.248	0.356	1.785	Type 4	2.092	2.499	-	0.130	18.700	18.400
-	-	-	-	100	-	-	-	-	-	-	-	-	-	17.712	16.771	-	0.280	0.517	2.010	-	2.063	2.460	-	0.191	-	-
3A/10P	3A/10	Circular	1	5	5.000	0.60	0.300	0.071	0.942	1	15.940	15.900	0.800	16.050	16.097	0.00	0.258	0.021	0.909	Type 2	4.536	4.536	BKSE30	0.021	17.500	17.100
-	-	-	-	100	-	-	-	-	-	-	-	-	-	16.073	16.149	-	0.278	0.031	1.024	-	4.464	4.464	-	0.031	-	-
3A/10	3A/9	Other	1	5	24.000	0.60	1.000	0.500	2.400	1	15.850	15.800	0.210	16.020	16.572	0.00	0.110	0.045	0.797	Type14	2.381	2.743	-	0.024	17.450	17.100
-	-	-	-	100	-	-	-	-	-	-	-	-	-	16.055	16.843	-	0.108	0.065	0.879	-	2.381	2.743	-	0.035	-	-
3A/9	3A/8P	Circular	1	5	5.000	0.60	0.750	0.442	2.356	1	15.650	15.600	1.000	16.134	16.162	0.00	0.380	0.616	2.045	Type 10	1.928	2.507	-	0.231	17.400	17.050
-	-	-	-	100	-	-	-	-	-	-	-	-	-	16.233	16.306	-	0.526	0.892	2.422	-	1.920	2.496	-	0.339	-	-
3A/8P	3A/7P	Circular	1	5	13.000	0.60	0.750	0.442	2.356	1	15.570	15.450	0.920	16.065	16.000	0.00	0.409	0.645	2.087	Type 3	0.439	0.439	BKLE30	0.029	17.350	17.000
	-	-	-	100	-	-	-	-	-	-	-	-	-	16.169	16.200	-	0.594	0.936	2.474	-	0.440	0.440	-	0.045	-	-

Table 24.B2 Hydraulic input data and hydraulic grade line results for the network in the 5 year and 100 year ARI storms

16.306	16.162	16.979	16.668	16.163	16.108	16.311	16.241	18.218	18.026	18.281	18.049	19.352	19.217	19.428	19.241	m	WATER SURFACE LEVEL		
0.694	0.838	0.071	0.382	0.937	0.992	0.789	0.859	0.182	0.374	0.119	0.351	0.448	0.583	0.372	0.559	m	FREEBOARD		
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	m	LIMIT CODE		