CHAPTER 9

ROAD DRAINAGE

INTRODUCTION

Despite the fact that roads cover a relatively small proportion of the earth's surface, the drainage of roads is one of the more important of the drainage engineer's responsibilities. The popularity of road transport has resulted in a public awareness of road design. Poor geometric designs from the point of view of drainage, whether to take away precipitation falling on the road surface, or to divert stormwater approaching the road, will receive public condemnation. In fact, there is reason for this. Water on road surfaces poses a serious hazard to safety and interruptions of traffic can disrupt commerce.

ROAD SURFACES

The depth of water on a road surface has a direct bearing on the safety of vehicles. There is a water film depth beyond which tyres tend to skid or plane when braking. Steering and acceleration are also affected. The friction factor of wet surfaces is lower than that of dry surfaces, but this feature cannot be avoided if it rains. The engineer can however, control the depth of water on the road.

Splashing of water affects visibility and comfort of passengers and the noise can impair driving. Although there have been advances in tyre tread design to reduce skidding, these designs can only go so far without adding to drag resistance on dry roads.

The road surfacing affects the skid resistance in a number of ways. A good surface will be rough as well as quick-drying. One way of doing this is to provide a permeable surface so that water may seep through the upper layer. Another method is to provide a camber or cross-fall. The latter can be uncomfortable and dangerous especially near the edge where camber is the greatest.

Factors affecting the permissible depth of water on a road include:

Traffic speed
Tyre tread design
Weight of vehicles
Tyre compound
Road surfacing material
Road crossfall
Deposits such as oil and dirt on the road
Flow velocity of water

Water depths less than 1 mm are rarely conducive to hydroplaning, but between 1 and 2 mm the water film can significantly affect the grip. For greater depths other factors such as visibility usually limit driving speed anyway. For depths over 5 mm, driving can be dangerous.

Stopping distances at 70 km/h on wet roads vary from 60 m on rough asphalt to 120 m on smooth asphalt, in the case of new tyres. The distances can become 80 m on wet rough asphalt and 160 m on wet smooth asphalt in the case of smooth tyres. These distances are about double those for dry roads. In the case of inundated roads the stopping distances may be much greater. It should be noted that the coefficient of friction on wet roads drops with speed, from 0.6 at 20 km/h to 0.1 at 45 km/h, for smooth wet asphalt (Visser, 1976, Jackson and Rogan,1974).

The cross-sectional profile of a road may be calculated assuming a certain permissible depth of water. Apart from the flow in the gutter adjoining the kerb, the camber may be designed to result in uniform depth across the road. Consider the road depicted in Fig. 9.1. It is assumed the road drains laterally to either side, i.e. there is a hump in the centre. Precipitation rate is assumed uniform without any losses, (alternatively the excess rainfall rate is used) and the flow depth is assumed to have reached equilibrium and be the same at all points. Consider a strip 1 metre wide across the road. Then the discharge per unit width is

$$q = ix \tag{9.1}$$

where i is the precipitation rate and x is the distance from the crown. According to the Manning equation

$$q = \frac{K}{N} y^{5/3} S^{1/2} \tag{9.2}$$

where K is 1.0 in metre units and 1.486 in feet units, S is the cross-fall slope, N is the Manning roughness coefficient and y is the flow depth. According to Strickler $N = 0.13Kk^{1/6}/\sqrt{g}$ where k is the equivalent roughness. Solving the previous two equations for cross-slope in terms of permissible depth of flow y,

$$\frac{dZ}{dx} = S = 0.0169 \frac{i^2 x^2 k^{1/3}}{g y^{10/3}} \tag{9.3}$$

Integrating this with respect to x we get an expression for cross-fall from the crown:

$$Z = 0.0056 \frac{i^2 x^3 k^{1/3}}{g y^{10/3}} \tag{9.4}$$

Thus if roughness k = 10 mm, g = 9.8 m/s², permissible water depth y = 1 mm, and design precipitation rate i = 100 mm/h, then

$$Z = 0.95\ x^3/10^3 \tag{9.5}$$

Thus for a road width of 6 m, $x = 3$ m and the crown rise must be $Z = 25$ mm.

GUTTER FLOW

Water flowing laterally off the road surface may either discharge into the countryside or into a lateral ditch, or be collected in shoulder drains. The latter may be trapezoidal formed ditches running beside the road, or may be formed between the cross sloping road surface and a near-vertical kerb. The water will flow longitudinally until diverted by an inlet to an underground drain. The longitudinal slope of the road as well as the lateral slope therefore influence the gutter flow.

The discharge rate in a trapezoidal channel may be related to depth of flow according to an equation such as the Manning equation:

$$Q = \frac{KA}{N} R^{2/3} S_o^{1/2} \tag{9.6}$$

where S_o is the longitudinal slope.

$$R = A/P \tag{9.7}$$

$$A = \frac{y^2}{2}\left(\frac{1}{S_1} + \frac{1}{S_2}\right) \tag{9.8}$$

$$P = y\sqrt{1 + 1/S_1^2} + y\sqrt{1 + 1/S_2^2} \tag{9.9}$$

and for turbulent flow

$$N \doteqdot 0.13\ Kk^{1/6}/\sqrt{g} \tag{9.10}$$

y is the water depth in the triangular shaped channel with side slopes S_1 and S_2 respectively and k is the equivalent roughness.

For a channel with one side a vertical kerb and the other side the road camber S_c, the flow equation becomes:

$$Q = 0.32\ y^{8/3} S_o^{1/2}/S_c N \text{ (SI units)} \tag{9.11}$$

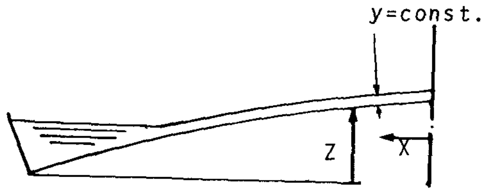
On the other hand if the gutter is treated as a lot of strips each with $A/P = y$, then integrating over the width results in

$$Q = 0.375\ y^{8/3} S_o^{1/2}/S_c N \text{ (SI units)} \tag{9.12}$$

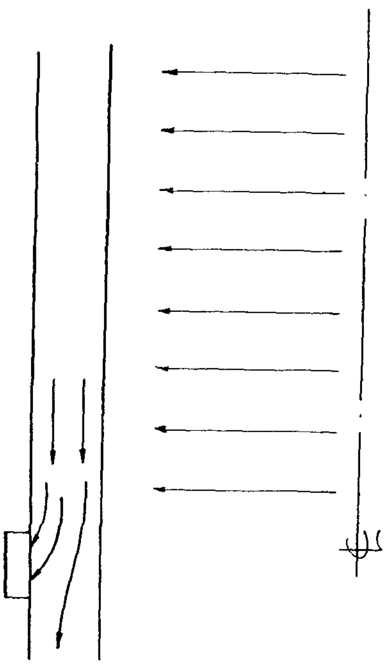
Here the longitudinal momentum of the water off the road is neglected and the discharge rate at any point is the rate of runoff from the area draining to that point. Again, owing to the limited areas usually involved, equilibrium conditions are assumed and the design flow is that corresponding to the maximum rainfall intensity for the selected recurrence interval.

RURAL ROADS

In the country it is rare to have gutters or kerbs. Runoff may run directly into adjacent lands. Alternatively it may be collected in



Exaggerated cross section through road



Plan of Road

Fig. 9.1 Road Drainage

mitre channels excavated alongside the road. Lateral channels will lead the water into fields in a herringbone pattern.

LATERAL INFLOW

The analysis of the flow profile along a channel with inflow along the length must be made using momentum principles, (Henderson, 1966). There is a loss of energy due to the inflow mixing with the water in the channel. The incoming flow is assumed to have no momentum in the longitudinal direction so one can write

$$M = \frac{Q^2}{gA} + A\bar{y} = \text{constant} \tag{9.13}$$

If there is a bed slope and bed resistance then one has

$$\frac{dM}{dx} = A(S_o - S_f) \tag{9.14}$$

This equation must be solved numerically, starting at a known control point. A problem may arise if the channel is steep and flow is supercritical at some point. In that case the critical flow section must be located.

INLET CONFIGURATION

Stormwater off the road will flow down the edges confined by a kerb to a channel with a triangular cross section formed by the camber on the road on one side and the kerb on the other side. The water may be intercepted at intervals by stormwater inlets leading to buried stormwater drains. The spacing and size of the inlets will depend on the design runoff rate. Details of the design of the inlet vary according to standard practice in different towns. A number of practical considerations should affect the selection of inlet type. Some of the configurations adopted are shown in Fig. 9.2. Vertical inlets into the kerb, termed kerb inlets, offer practical advantages to traffic, but are less efficient hydraulically unless special attempts are made to divert the flow laterally. Horizontal screens, termed gutter inlets, set in the road are liable to damage by heavy vehicles. Longitudinal slots in gutter inlets are more efficient than perforations, but pose a danger to bicycle traffic. Small perforations are also liable to blockage by litter or grass cuttings from verges.

In general, the cross-fall towards the inlet should be as steep as practical with depressions at the inlet adding to the efficiency. A small amount of carry-over to the next inlet is acceptable as the inlet capacity improves the deeper the flow.

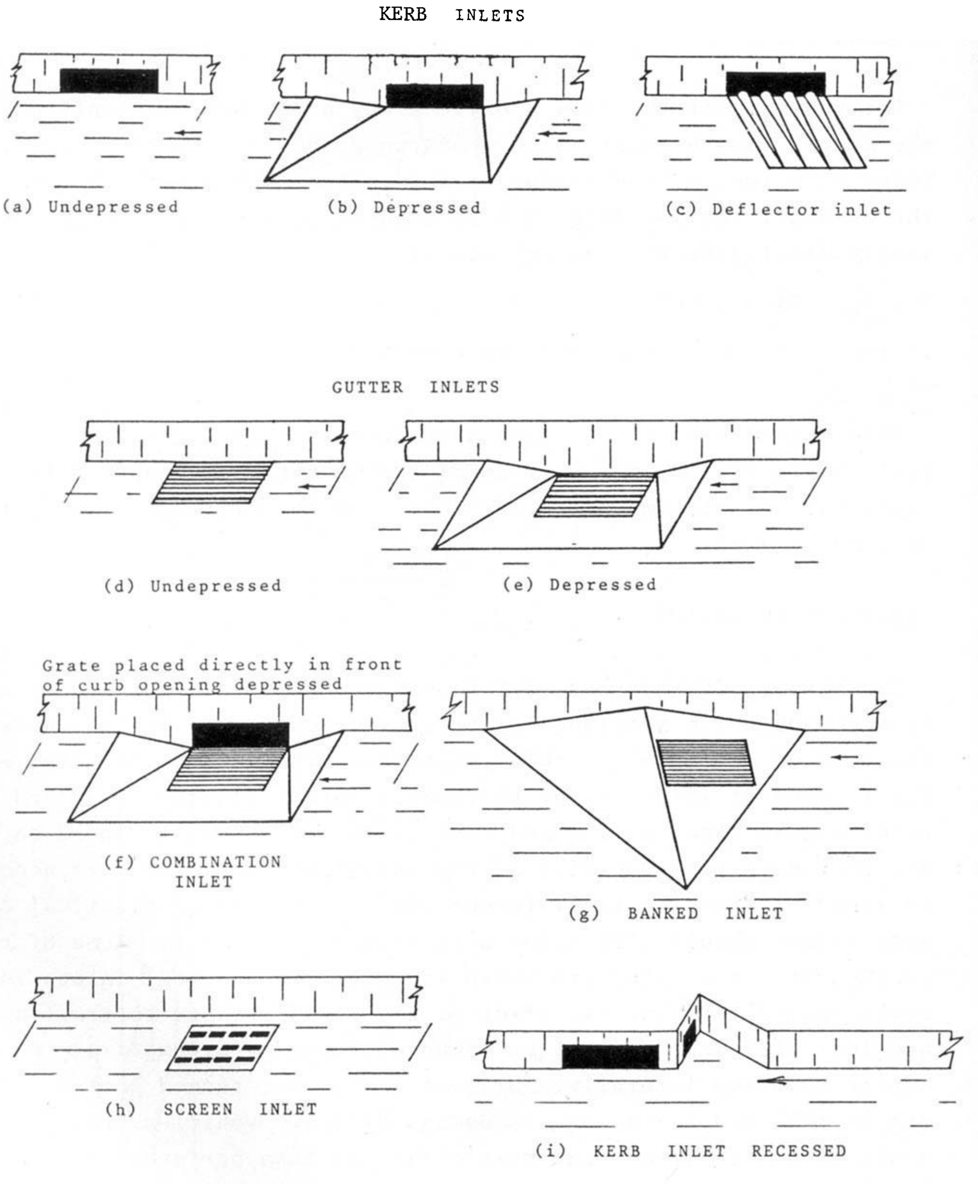


Fig. 9.2 Kerb and gutter inlets

SIDE WEIRS

In the case of lateral flow out of a channel, momentum is lost along the length, and energy principles must be employed for analysis. The specific energy or energy per unit mass of the water in the channel is assumed to remain intact, whence one may write

$$\frac{dE}{dx} = S_o - S_f \tag{9.15}$$

$$\text{but } E = y + \frac{Q^2}{2gA^2} + Z_1 \tag{9.16}$$

Hence it may be shown

$$\frac{dy}{dx} = \frac{S_o - S_f - \frac{2Q}{gA^2}\frac{dQ}{dx}}{1 - F^2} \tag{9.17}$$

where $F^2 = Q^2B/gA^3$, B is the channel width, Q is the flow rate, A is the cross sectional area of flow, y is depth, x is the longitudinal direction, S_o is bed slope, and S_f is friction gradient. dQ/dx is given approximately by

$$-\frac{dQ}{dx} = C_1\sqrt{2g}\,(y - H)^{3/2} \tag{9.18}$$

where H is the weir height and C_1 is about 0.51 if y, the depth at the crest is used on the right hand side, not E (see Ackers, 1970). The discharge in the main channel is, where b is the width,

$$Q = by\sqrt{2g}\,(E-y) \tag{9.19}$$

The last three equations may be solved numerically for outflow rate dQ/dx, discharge Q and depth y at any point x along a side weir. An analytical solution is possible for $S_o = S_f = 0$:

$$\frac{xC_1}{b} = \frac{2E-3H}{E-H}\sqrt{\frac{E-y}{y-H}} - 3\sin^{-1}\sqrt{\frac{E-y}{y-H}} + \text{constant} \tag{9.20}$$

The variations in possible water surface profiles are shown in Fig. 9.3.

KERB INLETS

The kerb inlet, i.e. a slot into the side of the kerb, remains preferable to the bottom or gutter inlet in many towns and centres despite its low hydraulic efficiency. A substantially larger hole is required than for a gutter inlet in most cases. Nevertheless as holes do not cost much more than curb, they are less susceptible to traffic damage, and as they are less of an obstacle to traffic than gutter grates, they remain popular in high density traffic zones.

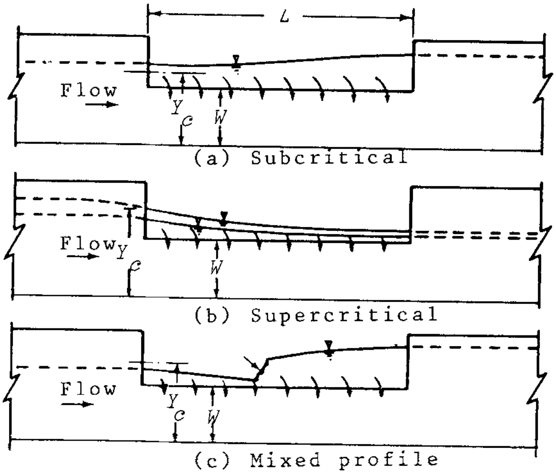


Fig. 9.3 Typical Flow Profiles at Side-Discharge Weirs.

The efficiency of kerb inlets is measured in terms of the amount of water diverted. Associated with the inlet or hole there may be means of improving the lateral diversion of the flow. This may include a steeper cross-fall than the general road cross-fall, a depression, or diagonal diverter ribs on the road surface.

Hydraulic analysis of the flow into a side inlet is difficult as explained previously. The research by the John Hopkins University (1956), also summarized by Li et al (1951-4) employed a semi empirical approach. They grouped the basic variables into significant dimensionless parameters, as follows: For a plain rectangular kerb opening without a gutter depression:

$$\frac{Q}{Ly\sqrt{yg}} = f\left(\frac{v}{\sqrt{gy}},\ \frac{q}{Q},\ \Theta\right) \tag{9.21}$$

From tests it was established that

$$\frac{Q}{Ly_o\sqrt{y_o g}} = K(\Theta) \tag{9.22}$$

where K = 0.23 for $\tan\Theta = 12$, and K = 0.20 for $\tan\Theta = 24$ and 48.

Here Q is the abstraction through the inlet, L is the inlet length to catch discharge rate Q, y is the depth, v the flow velocity in the longitudinal direction, and q the carry-over flow bypassing the inlet. Θ is the road cross-slope angle from the vertical (see Fig. 9.4 for undepressed inlet and Fig. 9.5 for depressed inlet arrangement).

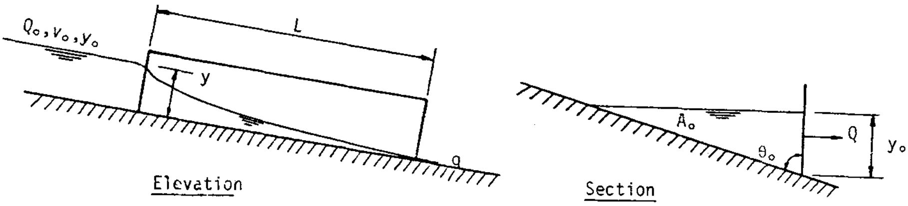


Fig. 9.4. Undepressed Kerb Inlet Notation.

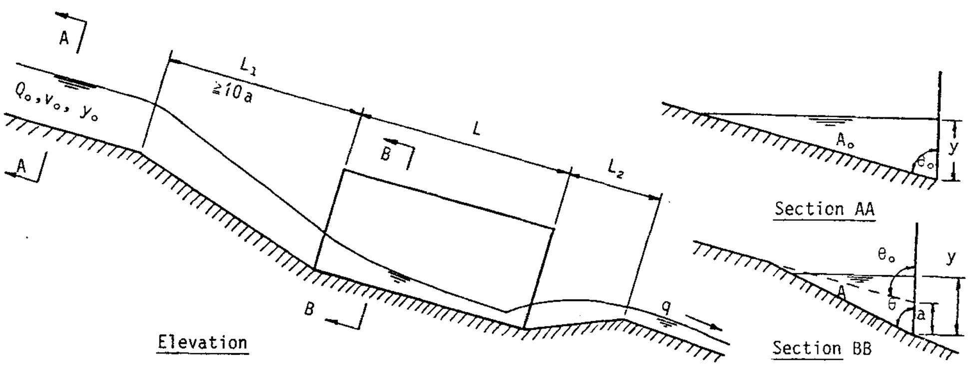


Fig. 9.5 Depressed Kerb Inlet Notation.

The depth of flow y in the depression and the corresponding velocity should be established from an energy balance using the Bernoulli equation

$$\frac{v_o^2}{2g} + y_o + Z_o = \frac{v^2}{2g} + y + Z + S_f L_1 \qquad (9.23)$$

assuming normal flow depth y_o upstream of the depression. Z is elevation and S_f is the friction loss gradient over length L_1. It was established that the inflow could be predicted by the equation

$$\frac{Q}{Ly\sqrt{gy}} = K + C \qquad (9.24)$$

where K is as for (9.22) and

$$C = \frac{0.45}{1.12^{(v^2L/gya\ \tan\Theta)}} \qquad (9.25)$$

a is the projected gutter depression (see Fig.9.5)

Li et al (1955) provide curves for quick estimation of y, C or Θ for any given design flow. ASCE (1969) present charts some of which are here translated to metric units. Fig. 9.6 and 9.7 are based on a two-dimensional depiction of the streamlines assuming uniform flow in the gutter.

Zwamborn (1966) used model and prototype studies to establish design equations for gutter flow, inlet capacity and optimum gutter depression. He employed the Chezy equation to express flow rate as a function of slope etc. as he was thus able to scale down roughness for model tests. He presented charts (in feet units) for gutter flow as a function of cross-fall and longitudinal slope of the road. From tests he determined that the water depth decreased linearly along the length of an undepressed gutter opening, although the depth y (see Fig. 9.4) is a fraction of the normal flow depth, as determined by experiment. Starting from the free-fall equation $Q/L = Kg^{1/2}y^{3/2}$ where L is the weir length, and integrating over the length L where depth y_o decreased linearly to zero, one obtains an equation similar to

$$Q/L = 0.33y^{1.25} \text{ (metre units)} \qquad (9.26)$$

where the coefficients were determined experimentally. Zwamborn also recommended that side inlets should be dropped 60 mm to give an increase in inlet capacity. For partly intercepted flow Zwamborn derived the equation

$$\frac{Q'}{Q} = 1 - \left(1-\frac{L'}{L}\right)^{5/2} \qquad (9.27)$$

where L' is the actual inlet length to catch Q' and L is the length required to catch the full flow Q.

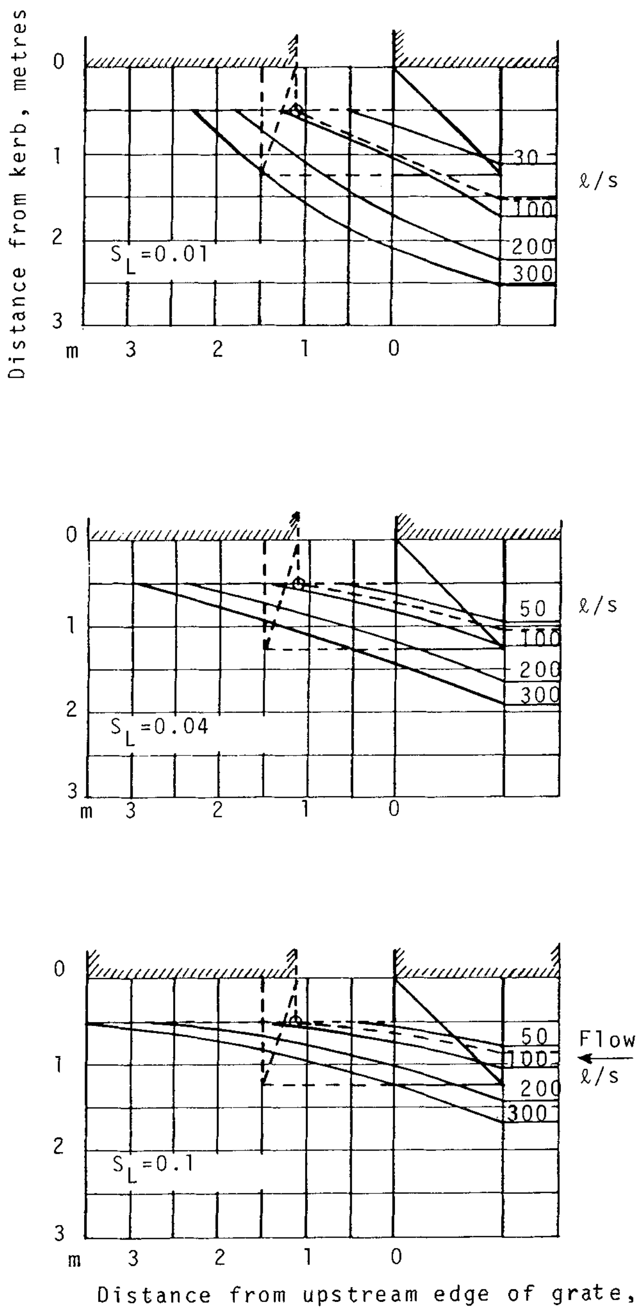
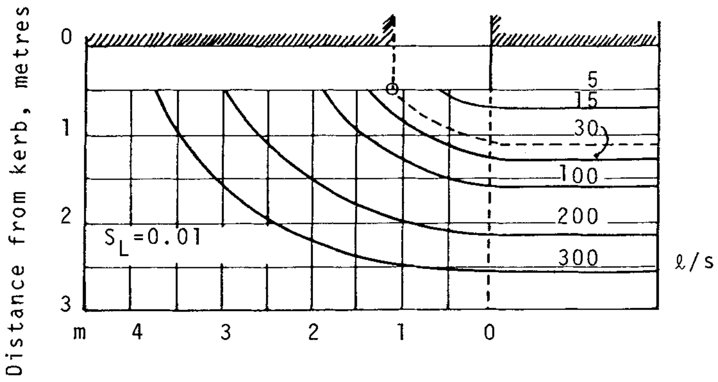
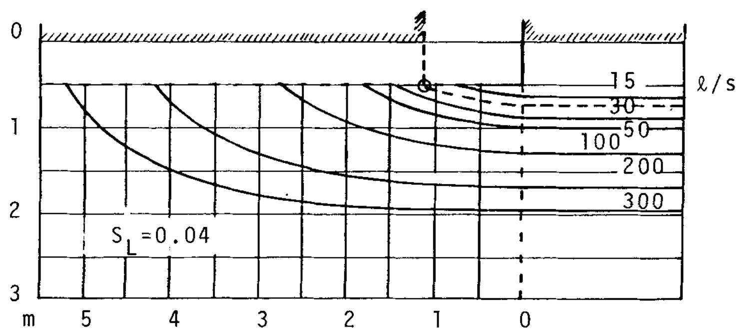


Fig. 9.6 Flow pattern for simplified design of combination inlets. Crown slope 1/18, N=0.013, Depression 65mmx1.2m





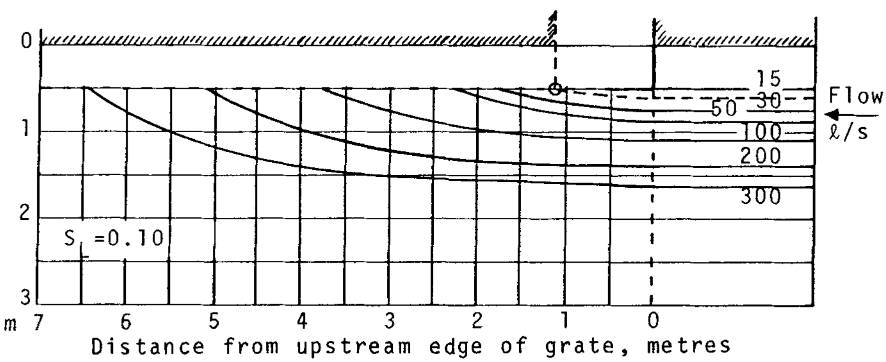


Fig. 9.7 Flow pattern for simplified design of combination inlets. Crown slope 1/18, N=0.013, Undepressed.

For ponded flow, however, it is recommended to use the equation

$$Q/L = 1.66\ y^{3/2} \text{ (metre units)} \tag{9.28}$$

Forbes (1976) later recommended a standard drop of 75 mm and drop width of 300 mm (Fig. 9.8 and 9.9).

Zwamborn mentions that grooves 100 mm wide, 50 mm deep and 50 mm apart at 45^0 to the kerb over 500 mm wide, increase gutter capacity up to 20%, but they may get damaged and become clogged.

Forbes noted that Zwamborn's results were based on tests on 0.9 m long inlets only, and that they were of limited use for steep gradients. Using approximations, he analyzed the flow pattern in the vicinity of the inlet. He considered successive cross sections, solving simultaneously the Manning equation in the direction of flow, the weir equations in the lateral direction and the continuity equation in steps using a desktop calculator. It was necessary to apply a correction factor of 0.48 to the computed results to conform to other published data.

Fig. 9.10 was prepared by Forbes to indicate the capacities of inlets for various road gradients, cross falls and inlet lengths. The charts also indicate the required upstream gutter length and flooded road width.

BOTTOM OPENINGS

In the case of flow over a longitudinal bar screen (Fig. 9.11) the outflow velocity head is equal to the specific energy. The discharge coefficient has been found to vary between 0.44 and 0.50 for bed slopes between 0.2 and 0.

If there are openings in a perforated screen, the outflow velocity head is equal to the overlying water depth. There is a change in energy due to the change in direction. The corresponding discharge coefficient varies from 0.75 to 0.80 for bottom slopes from 0.2 to 0.

GUTTER INLETS

Although horizontal inlets on the road surface are attractive in that they do not require kerbs, they suffer a number of disadvantages (ref. also U.S. Dept. Transport, 1969). Bars or perforated screens are required to prevent traffic falling in the hole. The inlet capacity is reduced by the screens or bars. In particular longitudinal bars which are most efficient hydraulic-wise, are a danger to bicycles unless narrowed down below 25 mm. The bars or screens are prone to blockage.

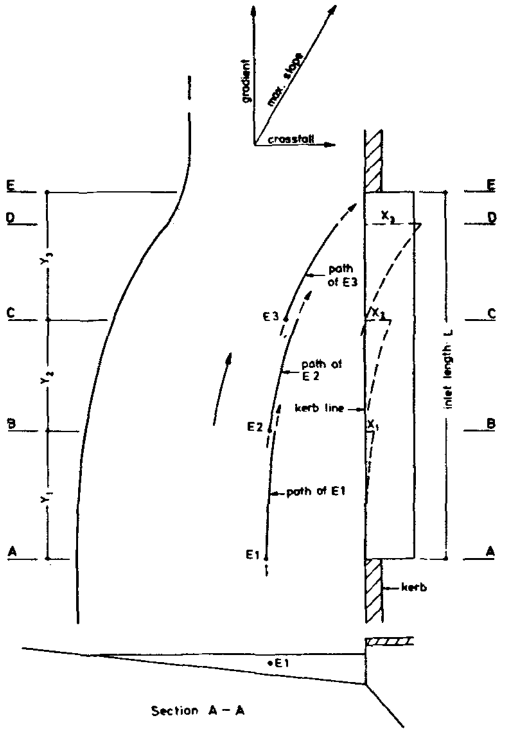


Fig. 9.8 Road flow, undepressed gutter for kerb inlet

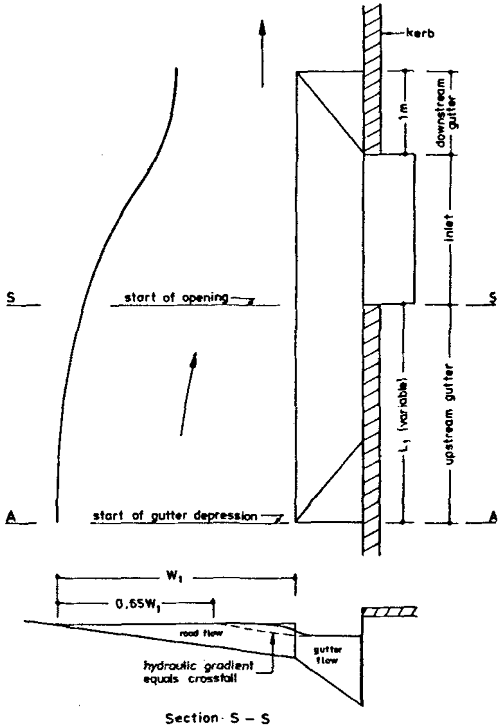


Fig. 9.9 Road flow, depressed gutter for kerb inlet

The screens or bars are liable to break under heavy loads. Gutter inlets, like vertical kerb inlets, are susceptible to overshoot unless angled into the flow path (see Fig. 9.2c). The most efficient system appears to be longitudinal bars unrestricted by laterals.

The length of grate required to fully capture the gutter flow may be estimated using free fall theory. If the approach depth is y_o and discharge per unit width of gutter q_o, then the locus of the water surface beyond the upstream edge of the gutter is given by the equations

$$y = gt^2/2 \therefore t = \sqrt{2y/g} \tag{9.29}$$

$$x = vt = qt/y = \sqrt{2q^2/gy} \tag{9.30}$$

i.e. the length of gutter grate to catch a flow q per unit width is $L_o = (2q^2/gy_o)^{1/2}$ where y_o is the depth of gutter flow.

This equation was found by the John Hopkins University (1956) to apply irrespective of whether there are bars obstructing the flow provided a/b > 1 (Fig. 9.12) where a is the gap width and b the bar width.

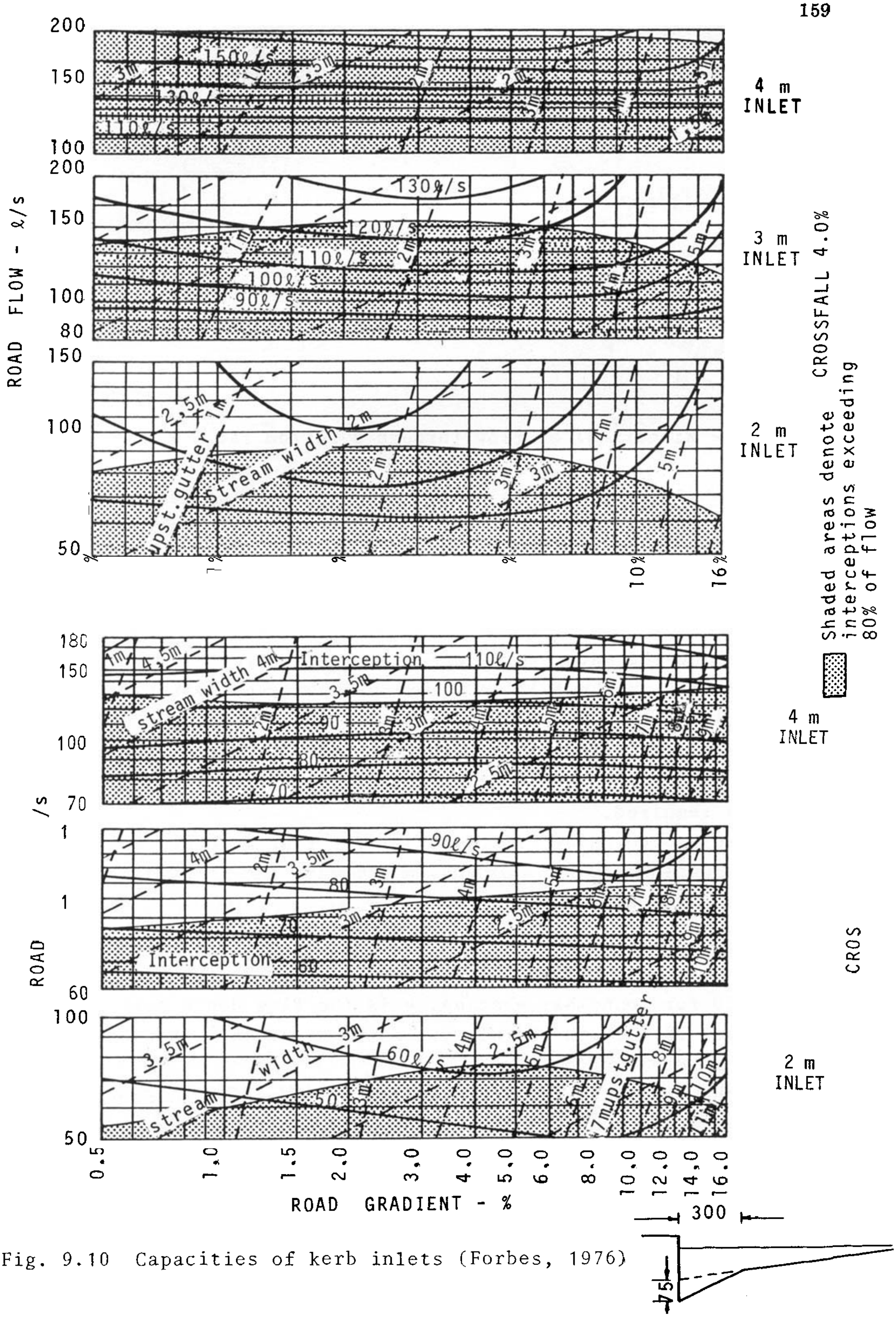


Fig. 9.10 Capacities of kerb inlets (Forbes, 1976)

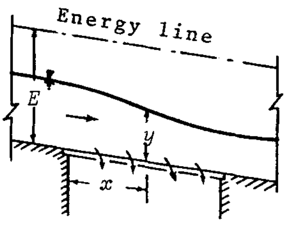


Fig. 9.11 Withdrawal of flow through a bottom rack

If the gutter flow is subcritical on account of a flat longitudinal slope then the flow depth at the grate crest may be assumed to be critical and

$$y_o = \sqrt[3]{q^2/g} \tag{9.31}$$

hence $L_o = \sqrt{2}\ y_c$ (9.32)

By plotting the water surface (which is an inverted parabola) from (9.30) where y is the fall in water surface level over a distance x, one may also estimate the length of inlet required for sloping grates. If the grate is tilted up slightly (at 5 to 10 degrees to the horizontal) the capacity increases considerably, or conversely a shorter grate length is required.

If there is overshoot, i.e. not all the flow is captured, then the orifice equation may be employed to estimate the inlet capacity. The inflow per unit width is

$$q = C_c L \frac{a}{a+b} \sqrt{2gy} \tag{9.33}$$

where C_c is a contraction coefficient, about 0.6 for square edges and nearly 1.0 for round bar grating. y is the flow depth here, and if this varies over the length of inlet then the equation must be integrated to obtain the total inflow.

The number of possible water surface profiles is, however, large. Downstream water depth is in turn dependent on discharge, so that it is difficult to solve the equation. The John Hopkins University (1956) therefore resorted to empirical tests.

Transverse bars increase the length of inlet required considerably. Thus three bars (at 1/4 points) will double the required length.

An alternative to transverse bars which has not been tested is diagonal bars.

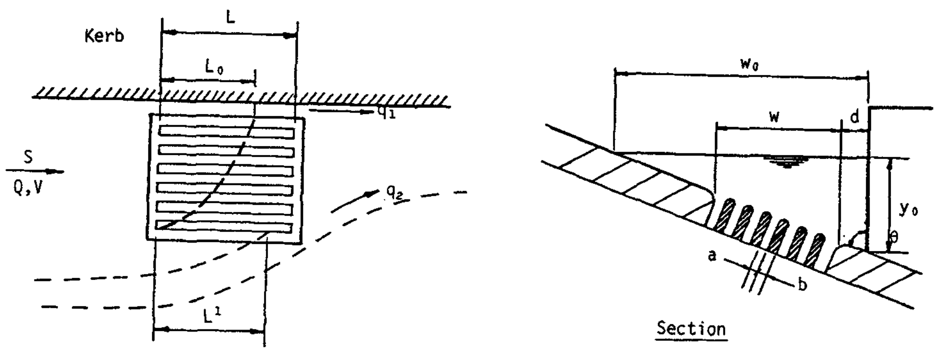


Fig. 9.12 Gutter Inlet Notation

For very wide road flow (W_o>>W where W is the width of grate and W_o the width of gutter), then for complete capture of the outer portion of the flow,

$$\frac{L'}{V_o}\sqrt{\frac{g}{y_o - W/\tan\Theta}} = 1.2 \tan \Theta \qquad (9.34)$$

and if the inlet length is less than this (i.e. L < L'), the carry-over is

$$q_2 = \frac{1}{4}(L'-L)\sqrt{g}\,(y_o - W/\tan\Theta)^{3/2} \qquad (9.35)$$

In the case of bar or perforated inlet screens, the coefficient of discharge is fairly low due to a vena-contracta of about 0.6 times the hole opening width (or each edge length in the case of perforations).

DROPS

Water entering an inlet from the road falls into a drain pipe. Apart from sizing the drop structure for ease of access, it should also take the design flow with minimum impedence. The drop inlet is normally free-fall so that the length of lip is the flow-limiting criterion. In fact, the vertical drop could be constricted without restricting the capacity.

The falling water tends to draw in air and this aerates the water in the drain beneath. This action can reduce the capacity of the drain. There is also a head loss in the drain due to the incoming flow

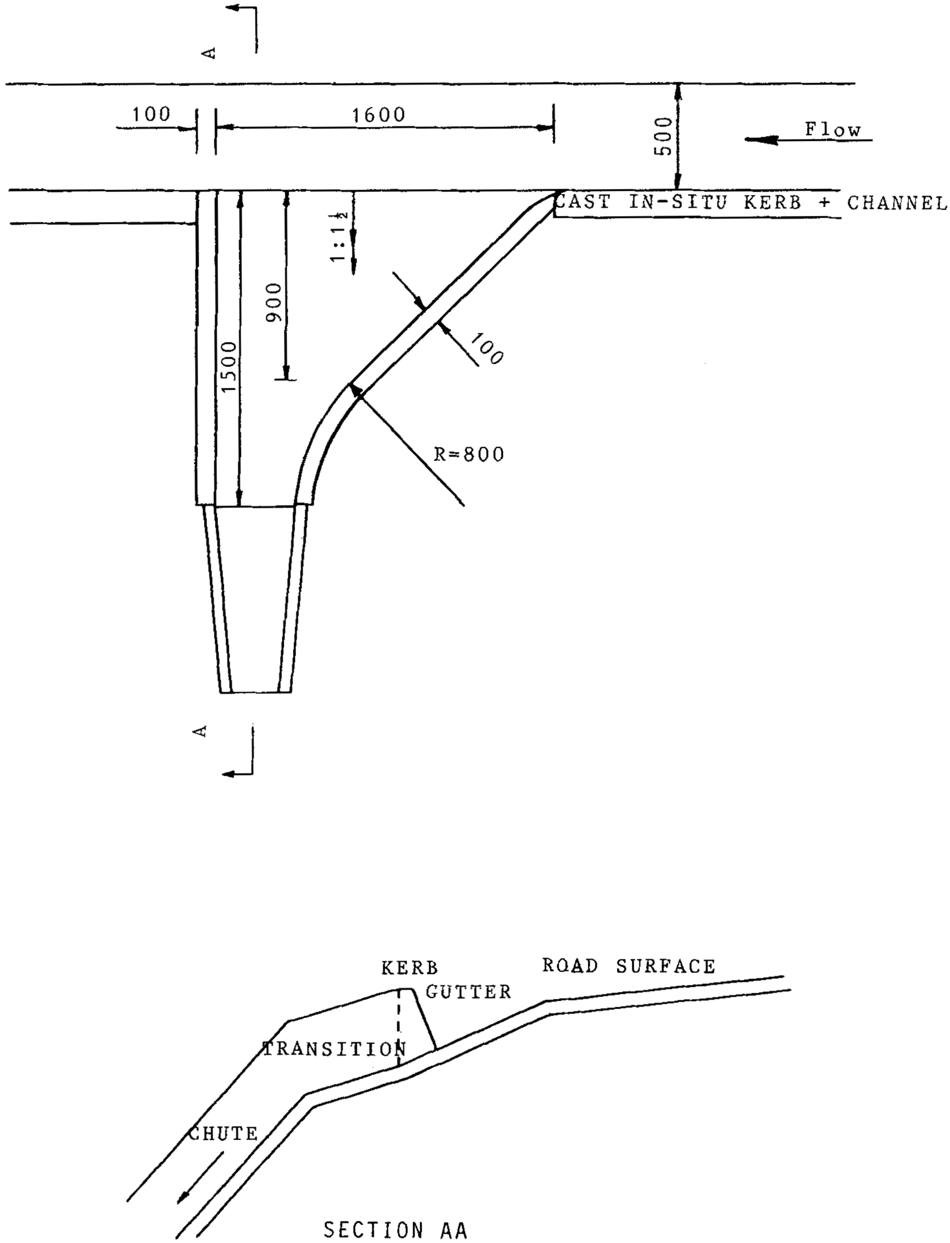


Fig. 9.13 Typical highway gutter transition

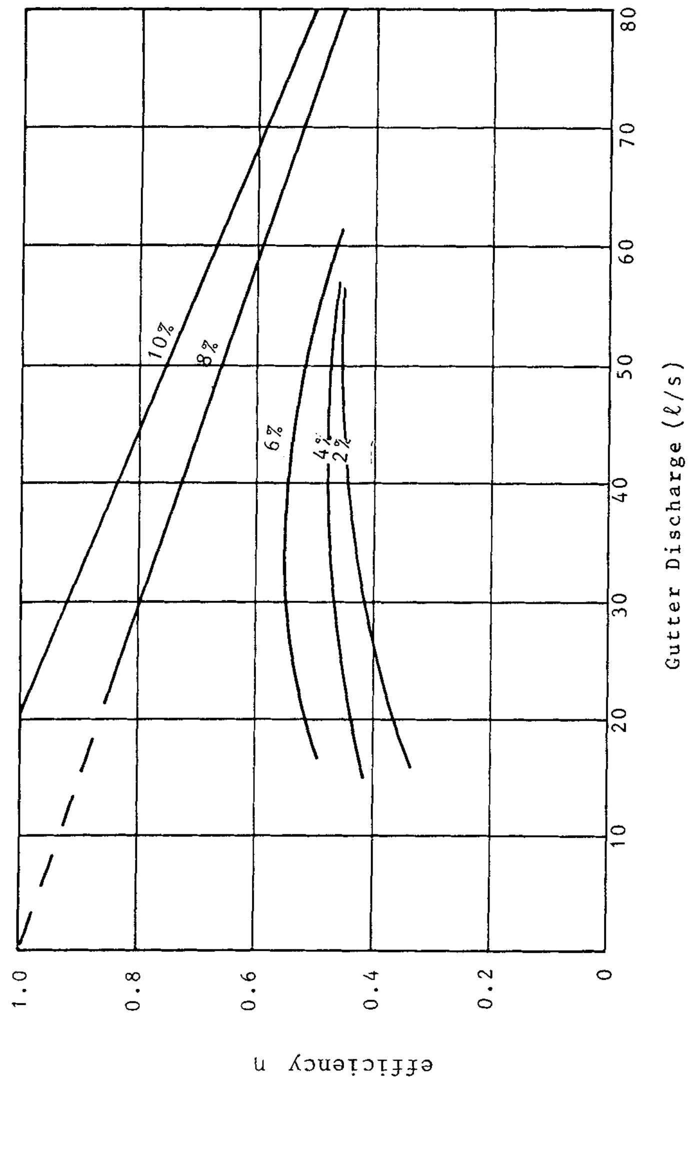


Fig. 9.14 Chute take-off efficiency curves for constant 1% longitudinal slope and variable cross slope

(Townsend and Prins, 1978). The losses may be assessed using momentum principles.

Devices for minimizing head losses and air entrainment have not been used extensively probably due to the complexity of construction. Spiral drops, (e.g. Ackers and Crump, 1960), chutes and tapered drops are some of the possibilities.

HIGHWAY CHUTE TAKE-OFFS

Where the road is on an embankment, stormwater will naturally run off laterally. Unless constrained, the runoff may erode the embankment, washing away vegetation and soil. It is common to provide shoulder gutters to collect the flow, and discharge down the embankment at suitable intervals. The design of chutes to take the flow down the embankment is discussed here.

The means of diverting the flow laterally is normally the restriction on the capacity of the system. Transitions of the type shown in Fig. 9.13 are used.

The chute itself can be made of precast concrete units. The base should receive particular attention as flow down the chute is supercritical and liable to erode unless some form of energy dissipation works is provided. Fig. 9.14 indicates that cross-fall plays a significant role in the catch-efficiency of take-offs.

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