CHAPTER 8

ROOF DRAINAGE

(from a paper by Schartz and Culligan, 1976)

INTRODUCTION

The number of reports of substantial damage to stock and installations in buildings as a result of the inadequate capacity of roof drainage facilities suggests that closer attention should be paid to the provision of waterproofing and storm-water control systems. The inconvenience of water flowing in and the resulting need to redecorate often detract from the prestige value of buildings and reduce the rate of return on the investment concerned.

Where high intensity rainfalls are experienced frequently it would appear acceptable to size eaves gutters so that they became periodically surcharged, provided of course that excess water can safely be discharged clear of the building. Internal or valley gutters or flat roofs should, however, be designed in such a manner that the consequences of functional failure are taken into account. The sizing of components can be rationally assessed only on the basis of a full consideration of the economic, hydrologic and hydraulic factors.

Steel-framed buildings with sloping roofs usually have gutters that are not integral with the roof so that a surcharge results in an overflow of the gutters. Concrete buildings, on the other hand, generally have horizontal or slightly sloping roofs and the problems that arise are due to the penetration of water through flaws in waterproofing membranes or inadequate flashing.

The high cost of ensuring lasting protection of flat roofs against moisture penetration is such that it is generally not wise to rely on a reduction of peak flows by roof-ponding (Fig. 8.1), a practice which in America has occasionally been enforced on property owners in order to reduce the surcharge on existing overloaded stormwater collection systems in the streets (Poertner, 1973).

In low and medium rainfall areas there seems little doubt that provision for storage is not warranted and most designers regard peak reduction by detention simply as an additional safety margin. Should it be decided to investigate the effect of storage then Fig. 8.2 after Pagan (1975) can be used to yield a preliminary estimate of the reduction likely to be achieved. Some suggestions for waterproofing regulations are contained in a paper by Lardieri (1975) on flood proofing.

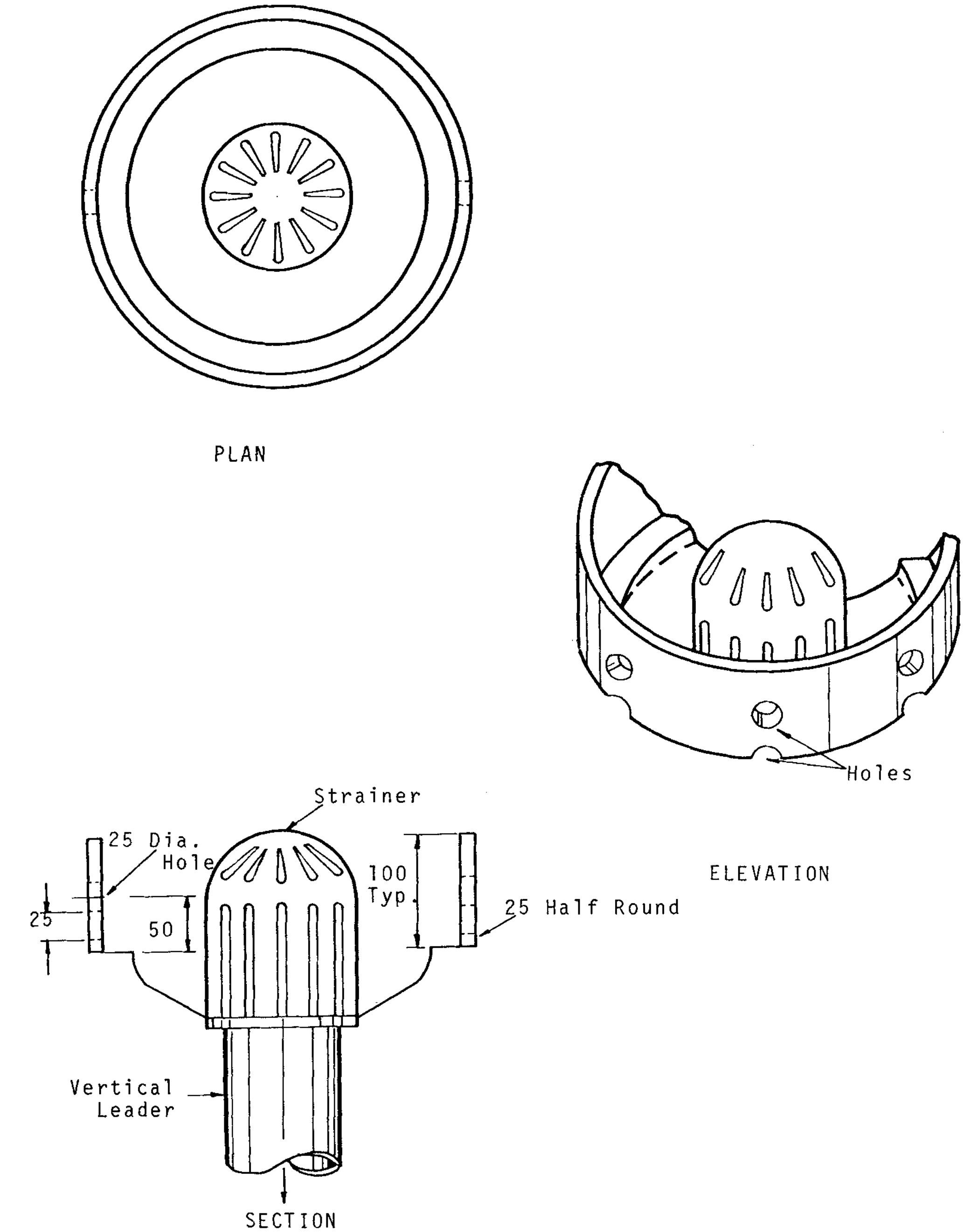

Fig. 8.1 Rainfall Detention Ponding Ring for Flat Roofs

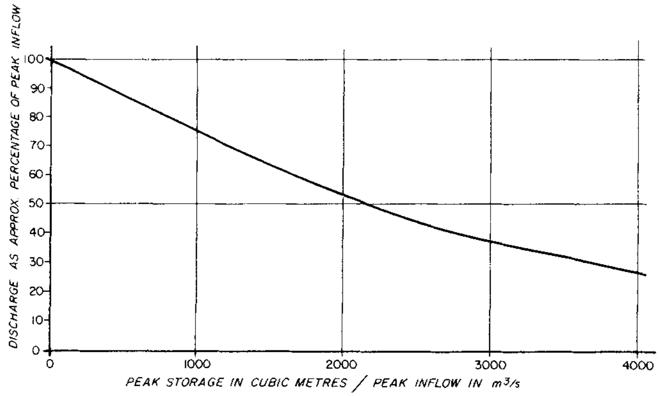

Fig. 8.2 Discharge attenuation due to storage (after Pagan, 1975)

It is practice in places to place downpipes in the centre of the columns of reinforced concrete buildings. This leads to certain difficulties in that the capacity of a down-pipe almost invariably depends on the design of its inlet. The concentration of beam or slab steel required at the top of a column often precludes the use of a hopper at that point. It may therefore be worth considering American practice of placing downpipes entirely clear of the columns.

In large buildings stormwater can be discharged internally into large conduits or culverts below ground level. An alternative approach sometimes adopted for large steel-framed industrial buildings is to place outlets at regular intervals in the floor of gutters and to collect the discharge from them in a suspended closed sloping launder or collector pipe which discharges at the perimeter of the building.

In 1973 the Division of Building Research of the CSIRO in Australia published a paper by Martin (1973) entitled 'Roof Drainage'. The paper presents a method of design which is essentially a modified version of a series of research digests published over a decade or more by the Building Research Station in England. The methods were adapted for Australian conditions where rainfall intensities are generally far higher than those of the United Kingdom. In addition, Martin investigated certain aspects such as the influence of slope on gutter capacity.

In April 1974 the British Standards Institution (BSI) issued a comprehensive code of practice which deals with the drainage of roofs and also of paved areas. Design procedures are given together with helpful notes on the practical considerations of the choice and disposition of elements of a drainage system. Special mention is made of the effects of mining subsidence. The publication contains diagrams giving roof areas served by rainwater pipes and gutters for design intensities of 75 mm/h. The diagrams may, however, be modified for other intensities

GUTTER CAPACITY

Optimum proportions of rectangular gutters

The depth of a valley gutter is generally limited by structural considerations such as the size of purlins or by other space limitations, but it is considered instructive to ascertain the optimum proportions of a level box gutter discharging freely at one end.

By application of the momentum principle it can readily be shown that if friction effects are ignored the maximum depth y at the upstream end of a level box gutter is $\sqrt{3}$ times the critical depth h_c (that is $y = 1.73h_c$). This theoretical relationship holds regardless of the length of gutter.

When frictional losses are included then an analysis similar to that developed by Hinds (1926) for side-channel spillways indicates that the maximum depth for the normal range of gutter lengths varies from about 1.8 to 2.1 times the critical depth.

In CP 308 (BSI, 1974) a value of twice the critical depth is advocated for design purposes. If the ratio of maximum depth to critical depth can be accepted as being constant then it can readily be shown that when a flat metal sheet of width W is to be bent into a rectangular horizontal gutter of any length then if an allowance is made for freeboard and lips, the remainder of the sheet should be bent in such a way that the maximum depth of flow y is three quarters of the gutter width b. Any other proportion would imply that the capacity of the gutter is less than the optimum for the material employed and the constraints specified. If the width of a rectangular gutter is chosen to be not less than 300mm in order to facilitate maintenance it follows that for flows less than 0.035 m^3/s it is not feasible to maintain optimum proportions.

If a strip of metal is bent into a rectangular gutter in such a way that the maximum depth of flow is one half of the width then for spatially varied flow the maximum discharge will be about five percent less than that of a gutter with optimum proportions.

Fig. 8.3 shows the width of gutter needed for a maximum depth to width ratio of both 0.75 and 0.5 and allows the designer to select a suitably sized gutter for various rainfall intensities. It must be borne in mind that the diagram is valid only if the water at the outlet discharges freely, say into a rainwater head.

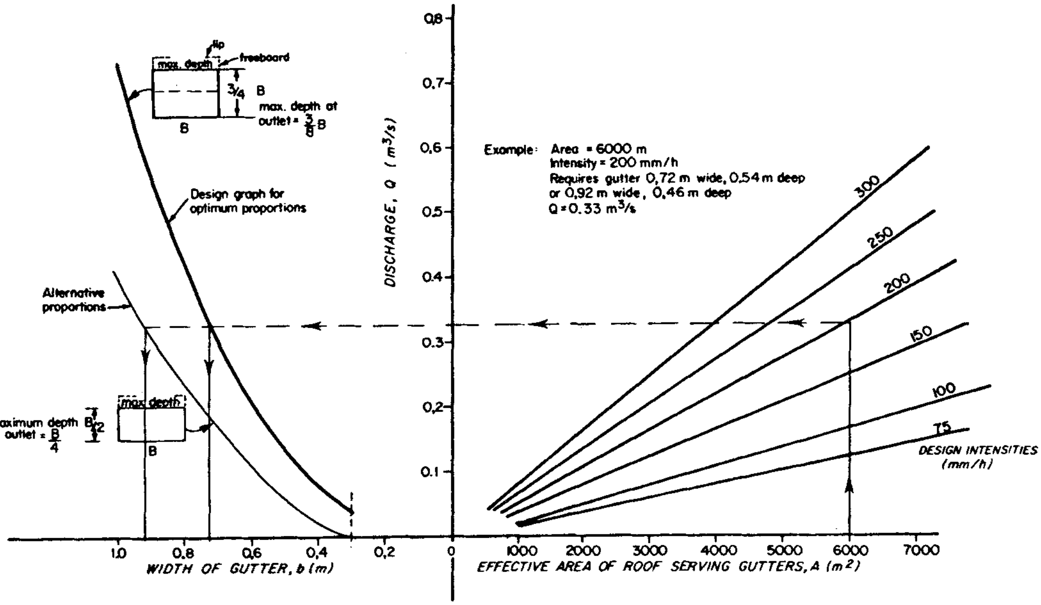

Fig. 8.3 Graph for design of level gutters

Sloping gutters

Sloping gutters carry more than horizontal gutters but unless the gutter is steep the additional discharge can generally be regarded only as an extra safety margin. Martin (1973) published a graph showing that when the slope of a drainage channel is about 2° the discharge capacity is doubled. If the flow should become supercritical then special care would have to be taken because water flowing supercritically would not readily negotiate bends.

A comprehensive computer analysis of flow in variously shaped horizontal gutters from 3 to 25 m long, established the effect of gutter length on maximum flow depth. Using the Hinds momentum equation it was found that the following empirical relationships accurately predict discharge capacity:

For rectangular gutters:

$$QL^{0.05} = 1.429\ (y\ b^{0.67})^{1.614} \qquad (8.1)$$

where Q is the discharge in m³/s, y, b and L are maximum flow depth, gutter width and gutter length respectively (all in meters).

For trapezoidal gutters:

$$Q = 0.697\ \frac{(A\ \Theta^{0.25})^{1.338}}{(b^{0.09})} \qquad (8.2)$$

where Q is the discharge in m^3/s, A is the cross-sectional area of the gutter in m^2, b is the bottom width in m, and Θ is the side slope in radians measured from the horizontal. For trapezoidal gutters the maximum depth including an allowance for freeboard can be taken to be approximately 2.3 times the critical depth.

For half-round gutters:

To take account of length effects the Building Research equation should be modified to read as follows:

$$Q = 2.26 \frac{0.8433A^{1.25}}{L^{0.47}} \qquad (8.3)$$

where Q is the discharge in m^3/s, A is the cross-sectional area of the gutter in m^2 and L the gutter length in metres.

Box receivers :

Where possible gutters should discharge freely into a box-receiver, the depth of which can be selected so as to match the use of a downpipe of convenient size. The receiver should be at least as wide as the maximum gutter width and should according to CP 308 be long enough to prevent the flow from overshooting the box. The horizontal distance m travelled by a particle leaving a horizontal gutter is given by the equation $m = 2\sqrt{ny}$ where y is the depth of flow at the outlet and n the vertical drop of the particle.

If one assumes that the jet is not to strike the far wall of the receiving box then the box could turn out to be unduly long and when loaded have a total mass of several hundred kilograms or more. It is therefore suggested that for large buildings the box be limited in size by the introduction of baffles even if the impact force has to be catered for in the structural design. The importance of placing the downpipe asymmetrically to prevent swirl which decreases effectiveness is worthy of note. External boxes should be provided with overflow weirs.

FLAT ROOFS

Flat roofs should have a slightly sloping upper surface to shed water to drains or outlets and it is recommended that ponding be minimized to restrict the ingress of water through waterproofing membranes that might for some reason have suffered damage. The depth of water on the roof will depend mainly on critical depth at overflow and thus a gutter or large depressed outlet is desirable. Fig. 8.4 gives for a series of representative rainfall intensities the area of flat roof served per metre of free overfall for selected depths of flow approaching the brink. For a limiting depth of 20 mm the critical depth would be 13.3 mm. The design of the downpipe and its inlet would have to be taken into account in establishing the depth of water likely to occur on the roof and this aspect is dealt with separately.

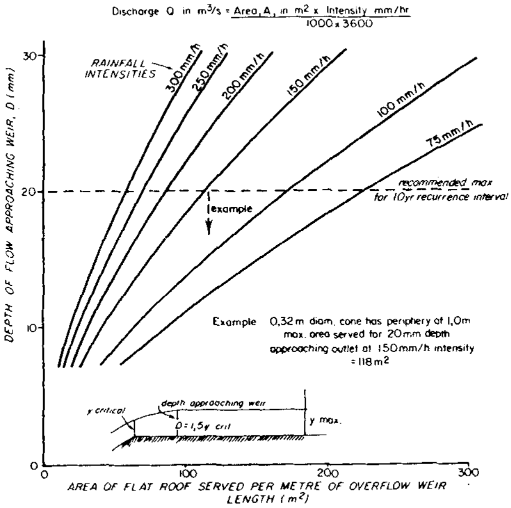

Fig. 8.4 Area of roof for unit length of free overfall available

DOWNPIPES

Martin (1973) found in Australia that the optimum size of downpipes to serve a gutter is given by the rule that the cross-sectional area of the downpipe should be half the cross-sectional area of the gutter. The rule is advocated by him as it has been found satisfactory in practice. Application of the rule presupposes, however, that a rain-water head of sufficient depth is available to avoid surcharge at the upper end of the gutter. Care must therefore be exercised in applying the rule. The British Code quite rightly lays stress on the design of the inlet and indicates that the size of the downpipe may be reduced once the water has entered it effectively.

For downpipes fed by flat areas and not gutters Martin limits the effective velocity to 1.78 m/s and produces a diagram (Fig. 8.5) which gives an indication of the roof area served by downpipes for various intensities of rainfall. It is important, however, to note that the capacity of a downpipe is normally controlled by inlet conditions and designers should avoid making the error of selecting a down-pipe size from the chart without ensuring that the water build-up at the entrance to the pipe necessary to feed the pipe at the design rate of flow can safely be accommodated. It should perhaps be emphasized that downpipes seldom run full. In fact, when the water reaches its maximum velocity in a vertical stack the pipe usually runs only about one-quarter full. Thus, down-pipes could be reduced in size but not without causing considerable noise and vibration due to pneumatic effects.

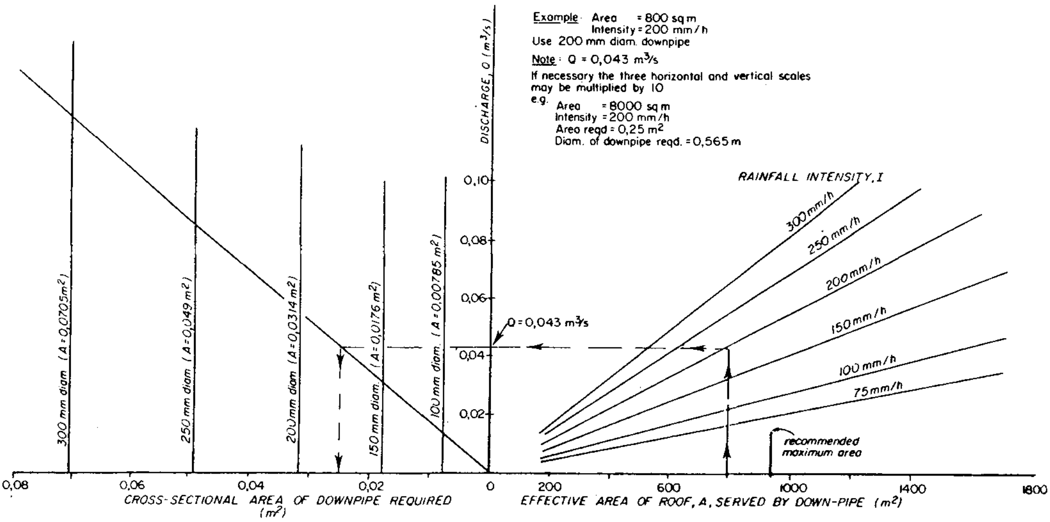

Fig. 8.5 Preliminary downpipe selection graph (inlet conditions to be checked)

Dawson and Kalinske (1939) showed that for ordinary plumbing stacks the maximum velocity is attained in about 3 to 6 m of fall. It follows that for multi-storied buildings the water velocity at ground-level would be no greater than that for a two-storey building. The maximum velocity measured in experimental stacks was of the order of 7 m/s and therefore Martin's rule of sizing downpipes by assuming that the nominal velocity based on the full cross-sectional area is 1.78 m/s appears reasonable.

The size of a downpipe fed by a gutter should also be selected from Fig. 8.5 and the inlet designed to provide sufficient head to ensure that the water enters without causing distress elsewhere.

Inlet conditions for downpipes

For a pipe flush with a flat roof or gutter floor the weir formula

$$Q = k_1 h^{3/2} \tag{8.4}$$

is applicable for low flows (i.e. when the head h on the weir overflow is less than about one-third of the diameter). For greater heads the orifice relationship

$$Q = k_2 h^{1/2} \tag{8.5}$$

is applicable. In these formulae Q is the discharge in m^3/s, h is the head in metres and k_1 and k_2 are appropriate constants.

If conical outlets are used then the origin of the orifice equation for the pipe entrance is below the roof or gutter level and as the discharge increases the control may shift to a lower level. If a protective grill is used due allowance should be made for its presence. Fig. 8.6 illustrates the concepts involved and it is immediately apparent from the figure that the design of an inlet is by no means a straightforward matter.

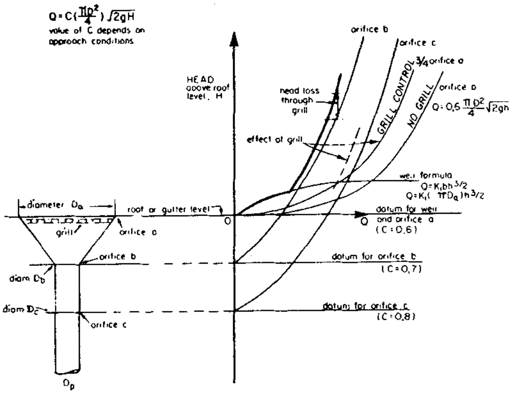

Fig. 8.6 Diagram showing method of determination of inlet control

If the downpipe is fed by a level rectangular gutter then (if surcharge is to be avoided) the depth of flow in the gutter at the outlet should not exceed 80 percent of the depth at the upstream end. For design purposes the more conservative rule that the water depth at the outlet should not be more than 50 percent of the effective gutter depth is recommended. If, on the other hand, the downpipe is fed directly from a flat roof then the approach head should be limited to about 25 mm and this severely restricts the discharge.

Calculations, supplemented by some laboratory tests for selected sizes, indicate that the chart given as Fig. 8.7 can be used for inlet design. Similar reasoning may be applied to ascertain the depth of water required in receiving boxes.

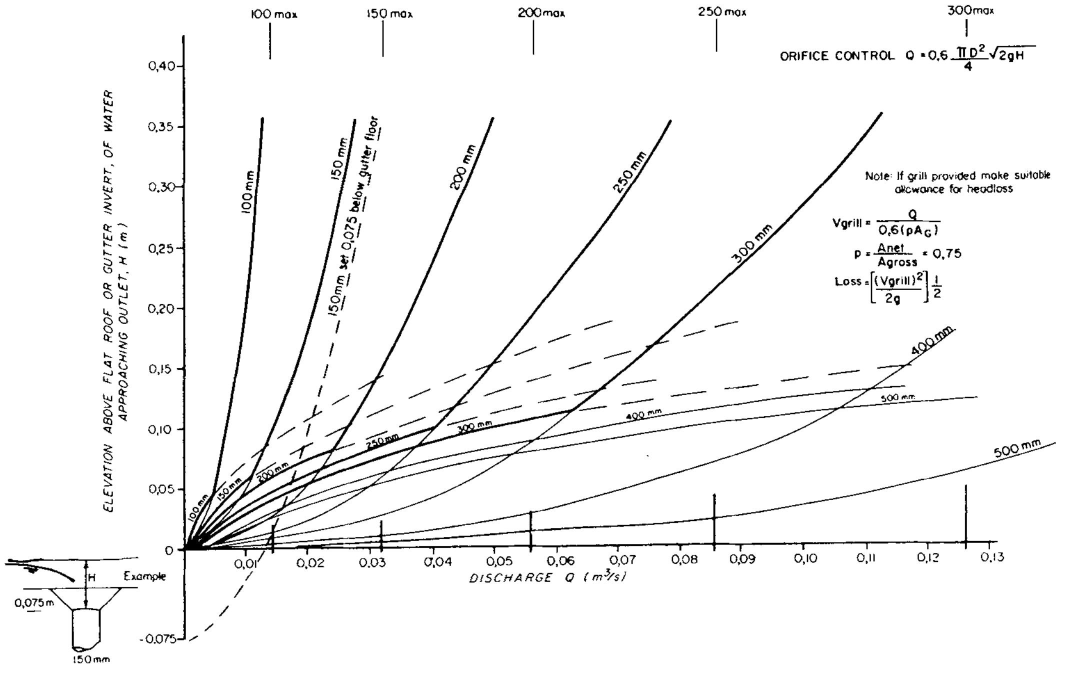

Fig. 8.7 Inlet design diagram

A useful technique for the design of conical outlets and tapers is to plot orifice relationships on a transparent overlay and then to slide the overlay vertically over a diagram such as Fig. 8.7 to establish optimum conditions.

A grill in a conical outlet acts as a control for low discharges and as an obstruction for larger discharges. Generally the open area of the grill is about 75 percent of the gross area A, and the head loss can be approximated by taking half the velocity head at each vena-contracta. The total effective area is then about 60 percent of the 75 percent mentioned above. Thus, the head loss, h, across the grill, may be approximated by the expression

$$h = 0.5 \left(\frac{Q}{0.6 \times 0.75 \times A_1}\right)^2 \frac{1}{2g} \qquad (8.6)$$

where Q is the discharge in m^3/s and A_1 the gross grill area in m^2.

Fig. 8.8 may be used for the final selection of downpipe size depending on the value of available head. Where receivers are used the depth of receiver required for a selected downpipe diameter may be determined from the diagram.

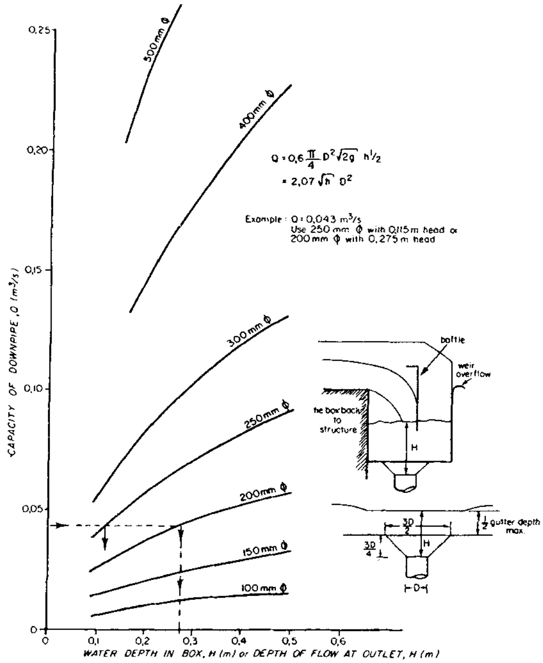

Fig. 8.8 Downpipe selection for different available heads

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