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THE DEPARTMENT OF THE ENVIRONMENT FOR NORTHERN IRELAND

Hydraulic Design of Road-edge Surface Water Channels

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VOLUME 4 GEOTECHNICS AND DRAINAGE
SECTION 2 DRAINAGE

PART 4

HA 37/97

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Contents

Chapter

1. INTRODUCTION

1.1 In the United Kingdom there are three main methods of dealing with surface run-off from rural trunk roads: filter drains; kerbs and gullies connecting to pipes below ground; and surface water channels along the pavement edge. This document is concerned with the hydraulic design of surface water channels.

1.2 The use of surface water channels allows larger distances between outlets than conventional gully systems, and the positions of the outlets can often be chosen to suit the local topography and the occurrence of natural watercourses. With this type of system, seepage flows in the pavement construction are usually drained by means of fin drains at the edge of the pavement. Geometric and performance requirements for surface water channels and fin drains are given in Volumes 1 to 3 of the Manual of Contract Documents for Highway Works (MCHW 1, 2 & 3) and in HA 39, Edge of Pavement Details (DMRB 4.2).

1.3 In order to design a surface water channel for a given frequency of flooding, it is necessary to take account of the time of travel of flow along the channel and the variation of rainfall intensity with storm duration. The design method described in this document is obtained by combining results from kinematic wave theory about time-varying flow conditions in channels [Refs 3 and 4] with a newly-developed description of UK rainfall characteristics. The validity of kinematic wave theory for flows in shallow drainage channels was checked using measurements from a trial on the M6 motorway [Ref 5].

1.4 Guidance on the hydraulic design of triangular and rectangular surface water channels was previously given in HA 37/88, Hydraulic Design of Road-edge Surface Water Channels (DMRB 4.2) in the form of a series of design curves. The present document uses the same hydraulic principles as before but with a new rainfall formula that is optimized for storm durations between two minutes and twenty minutes; this range covers the great majority of applications. The design method has been generalized so that it can additionally now be applied to channels of trapezoidal cross-section. The results are presented in the form of equations instead of graphs so as to allow a simpler calculation procedure that is also suitable for use in computer-based design packages.

2. SCOPE

2.1 This Advice Note describes a method of determining the length of road between outlets that can be drained by a given size of surface water channel constructed along the edge of the road. The design method can be used for channels of triangular, rectangular or trapezoidal cross-section (see Figure 1), and is based on application of Equation (13) in paragraph 5.3.

2.2 Safety considerations limit the depth and types of cross-sectional shape that may be used for surface water channels which are not segregated from traffic. Permitted dimensions and cross-falls given in MCHW and HA 39 (DMRB 4.2) are summarized in Chapter 3.

2.3 In the design method, the longitudinal gradient of a channel may either be constant or vary with distance. In the latter case, the gradient can be zero at the upstream or downstream end of the channel, but at all intermediate points there must be a positive fall towards the outlet. An approximate procedure is also given for checking the performance of channels when they are surcharged.

2.4 The design equations are based on assumptions that the cross-sectional properties of a channel do not vary with distance or depth of flow and that the width of road drained between two adjacent outlets is constant. If these assumptions are not satisfied, approximate results may be obtained using average values of road width or cross-sectional shape.

2.5 The design of outlets for use with surface water channels is described in HA 78, Design of Outfalls for Surface Water Channels (DMRB 4.2).

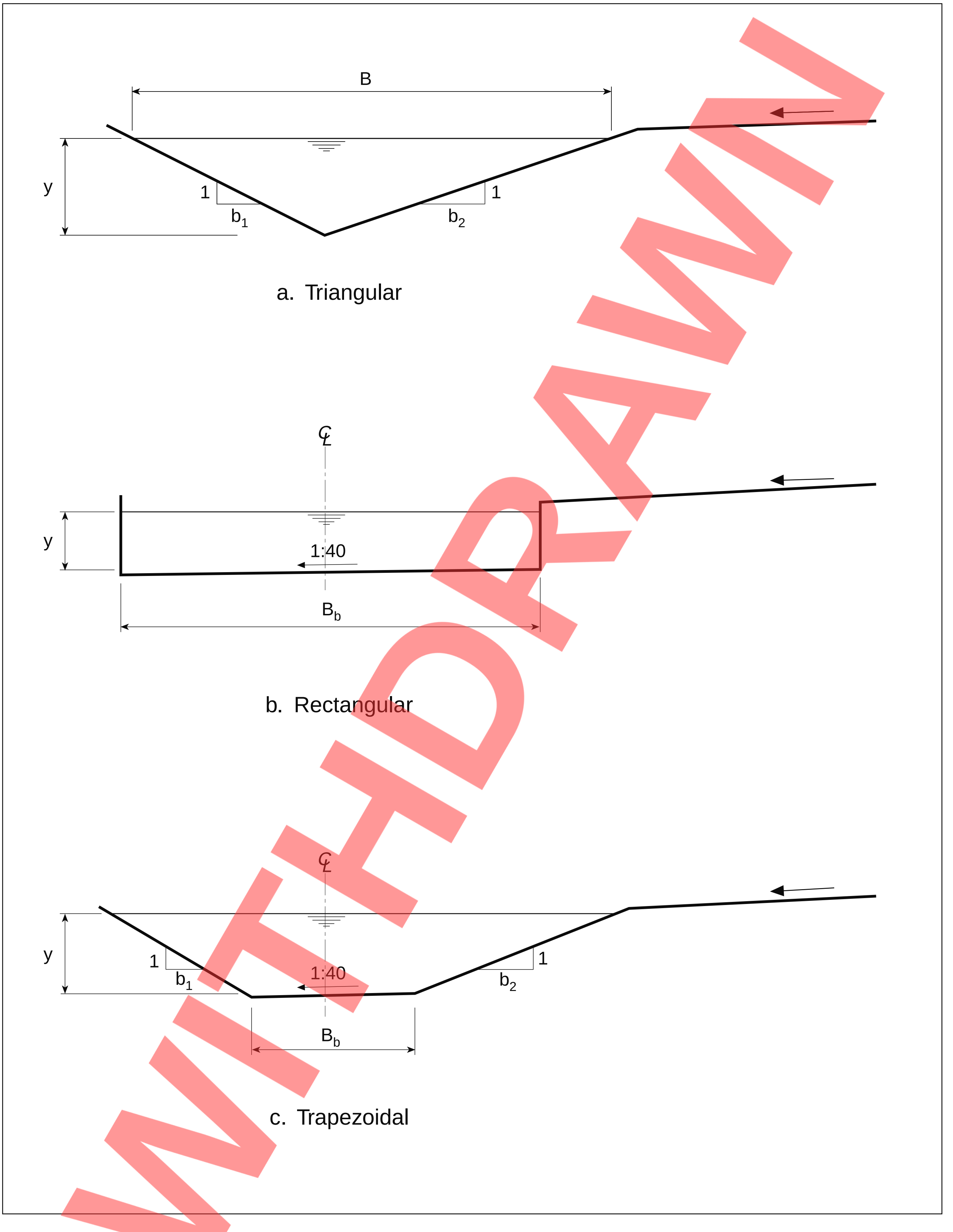


Figure 1 Cross-sectional shapes of channels

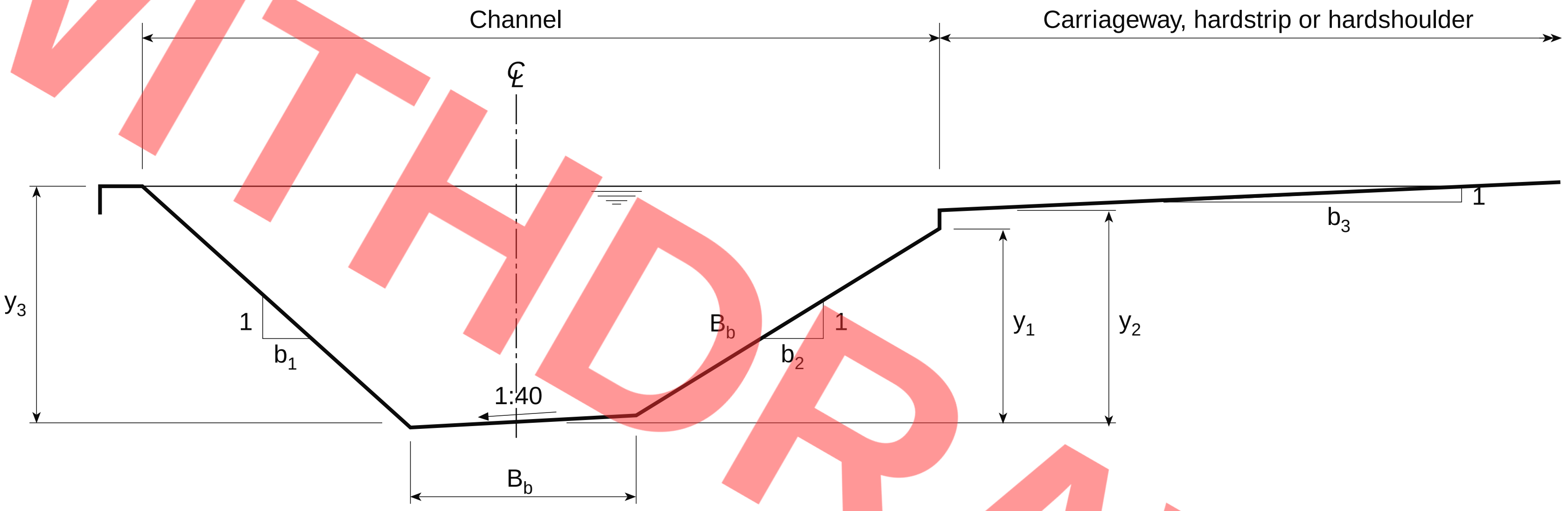
3. SAFETY ASPECTS

3.1 When considering the use of surface drainage channels, in particular those of rectangular cross-section, safety aspects relating to their location should be taken into account. Triangular or trapezoidal channels will usually be sited adjacent to the hardstrip or hardshoulder or at the edge of the carriageway and in front of the safety fence, where one is provided; layout details are given in the 'A' and 'B' Series of the Highway Construction Details (HCD) (MCHW 3). In these locations the maximum design depth of flow (dimension y_1 in Figure 2) should be limited to 150mm. In both verges and central reserves the channel slopes ($1:b_1$ and $1:b_2$ in Figure 2) should not normally be steeper than 1:5 for triangular channels and 1:4.5 for trapezoidal channels; in very exceptional cases slopes of 1:4 are allowable for both types of channel.

3.2 Rectangular channels, or triangular channels of depth greater than 150mm, should be used only when safety fencing is provided between the channel and the carriageway; such channels can, therefore, normally only be justified when safety fencing is warranted by other considerations. In addition, these channels should not be located in the zone behind the safety fence into which the fence might reasonably be expected to deflect on vehicle impact. Shallower channels as described in paragraph 3.1 may lie in this deflection zone, or be crossed by the safety fence (usually at a narrow angle) provided that the combined layout complies with the requirement of TD 19, Safety Fences and Barriers (DMRB 2.2), TD 32, Wire Rope Safety Fence (DMRB 2.2.3) and the HCD (MCHW 3). Further advice on such layouts should be sought from the Overseeing Department.

3.3 Co-ordination of the layout of safety fences and surface water channels must be arranged at an early stage in design and not left to compromise at later stages. Where safety fences are not immediately deemed necessary, sufficient space should be provided in the verge or central reserve to allow for their possible installation. The combined layout must comply with the requirements of TD 19, TD 32 and the HCD in terms of set-back and clearance dimensions and the mounting height of the safety fence.

3.4 The geometric constraints given in this document should also be applied to channel outfall details. In outfall design and at other channel terminations, slopes exceeding 1:4 should not be used on any faces, particularly those orthogonal to the direction of traffic, unless such faces are behind a safety fence.



Note: Vertical scale exaggerated

Figure 2 Cross-section of surcharged channel

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4. BASIS OF DESIGN METHOD

4.1 The design method is based on the results of kinematic wave theory [Refs 3, 4 and 5] which is able to provide a mathematical description of the time-varying flow conditions that can occur during a storm in a channel receiving inflow along its length. It is assumed that the main factors determining the depth and velocity of the water at any point are the rate of flow, the longitudinal slope of the channel and its hydraulic resistance. The effect of other factors associated with the inertia of the water and the slope of the water surface along the channel are assumed to be small in comparison and are neglected. These assumptions are normally reasonable in long drainage channels where changes in velocity occur relatively slowly with time and distance.

4.2 In using kinematic wave theory to determine time-varying flow conditions in a collecting channel, no account has been taken of any flow attenuation due to water temporarily held on the road surface during a storm. Water depths on roads are normally only a few millimetres and any depression storage is likely to have been filled by rainfall occurring before the most intense part of the storm that constitutes the design event. It is therefore assumed that the hydrograph of the lateral inflow to the channel is equal to the product of the instantaneous rainfall intensity and the effective width of the catchment but with a time shift corresponding to the time of travel of the runoff across the road surface.

4.3 The relationship between the flow rate and water depth at any point along the channel is obtained from the Manning resistance equation:

$$Q = \frac{AR^{2/3}\,S^{1/2}}{n} \qquad (1)$$

where Q is the discharge (m^3/s), A is the cross-sectional area of flow (m^2), S is the longitudinal gradient of the channel (m/m) and n is the Manning roughness coefficient; the hydraulic radius R (m) is given by:

$$R = A/P \qquad (2)$$

where P is the wetted perimeter of the flow (m).

4.4 For a channel of trapezoidal cross-section with a base width B_b, side-slopes of 1: b_1 and 1: b_2 (vertical: horizontal) and flow depth y (see Figure 1c), the Manning resistance equation becomes:

$$Q = \frac{S^{1/2}}{n}\left[\frac{(2B_b + by)^5}{32(B_b + by)^2} r^2 y^5\right]^{1/3} \qquad (3)$$

where the effective cross-fall b of the channel is defined as:

$$b = b_1 + b_2 \qquad (4)$$

and the hydraulic-radius factor r (= flow width/wetted perimeter) is given by:

$$r = \frac{B_b + (b_1 + b_2)y}{B_b + \left[(1 + b_1^2)^{1/2} + (1 + b_2^2)^{1/2}\right]y} \qquad (5)$$

For a wide shallow channel, r tends towards unity.

4.5 For the special case of a triangular channel, the sole width B_b=0 so that Equations (3) and (5) can be written as:

$$Q = \frac{bS^{1/2}}{n}\left[\frac{r^2 y^8}{32}\right]^{1/3} \qquad (6)$$

$$r = \frac{b_1 + b_2}{(1 + b_1^2)^{1/2} + (1 + b_2^2)^{1/2}} \qquad (7)$$

A triangular channel with one side vertical can be catered for by putting $b_1 = 0$.

4.6 For the special case of a rectangular channel of width B_b, the side-slopes $b_1 = b_2 = 0$ so that:

$$Q = B_b S^{1/2}\left[r^2 y^5\right]^{1/3} \qquad (8)$$

$$r = \frac{B_b}{B_b + 2y} \qquad (9)$$

4.7 Kinematic wave theory enables the peak depth of flow at the downstream end of a channel to be determined for a given intensity and duration of rainfall. For design, it is then necessary to identify the critical storm duration which will produce the maximum possible depth of flow for a given frequency of occurrence. This has been done by using the following equation:

$$I_o = 32.7\,(N-0.4)^{0.223}\,\frac{(T-0.4)^{0.565}}{T}\,(2minM5) \qquad (10)$$

where I_0 is the mean rainfall intensity (mm/h) occurring in a storm of duration T (minutes) with a return period of N (years), such that a storm of this intensity will occur on average once every N years. The quantity 2minM5 is the depth of rainfall (in mm) occurring at the particular geographical location in a storm with T= 2 minutes and N = 5 years. Equation (10) was obtained for the purpose of this document by curve-fitting data on UK short-period rainfalls given by the Meteorological Office in British Standard BS 6367: 1983 [Ref 6]. Further information is given in Annex A.

4.8 The intensity of rainfall normally varies with time during a storm, and this affects the way in which a channel responds to run-off. Typical profiles of rainfall intensity in summer and winter storms are given in Volume II of the Flood Studies Report [Ref 7]. For impervious surfaces such as roads, the largest flows tend to be produced by heavy short-period storms which occur more frequently in summer than winter. The design method therefore uses what is termed the 50% summer profile, in which the peak intensity at the mid-point of the storm is approximately 3.9 times the average intensity; 50% of summer storms in the UK can be expected to have lower ratios of peak to mean intensities. Data from the M6 motorway [Ref 5] showed that using the 50% summer profile in the design method gave conservative results.

4.9 As explained in paragraphs 4.7 and 4.8, the critical storm duration for a channel depends on the geometric properties of the channel and the catchment that it drains and also upon the local rainfall characteristics and the type of storm profile. However, in the design equations given in Chapter 5, the critical storm duration has been eliminated as an independent variable and does not need to be determined separately when calculating the length of road that a channel can drain.

5. DESCRIPTION OF DESIGN EQUATIONS

5.1 For economic reasons, it will normally be appropriate to use only one or two sizes of channel for a particular road scheme and to vary the spacing between outlets in order to allow for the effect of changes in channel slope or drained width of road. The length of road that a channel can drain may be determined directly by use of Equation (13) in paragraph 5.3. A second type of design problem may sometimes occur if it is required to find the size of channel needed to drain a given length of road. As explained in paragraphs 5.4 and 5.5, a direct solution is possible for triangular channels but iterative procedures are necessary for rectangular and trapezoidal channels.

5.2 The flow capacity of a channel depends upon its shape as well as its cross-sectional area. The shape characteristics can be expressed in terms of a parameter "m" defined as:

$$m = \frac{By}{A} - 1 \qquad (11)$$

where B and A are, respectively, the width of flow and the cross-sectional area of flow corresponding to the design depth of flow in the channel. For a triangular profile m = 1, and for a rectangular profile m = 0; trapezoidal channels will have values of m between these two limits. The design equations given in the next clause can also be applied to dished channels in which the flow width and flow depth are related by an equation of the form:

$$B = cy^m \qquad (12)$$

where c is a constant for a particular type of channel.

5.3 The length of road L that can be drained by a surface water channel is obtained from the general equation:

$$L = G_m \frac{S^{1/2}}{n} (ry)^{2/3} (N - 0.4)^{-0.362} \left[\frac{A}{W_e\,(2minM5)}\right]^{1.62} \qquad (13)$$

where the factor G_m for channel shape is:

$$G_m = 2.90 \times 10^6 \,(2.65 - m) \qquad (14)$$

and W_e is the effective width of the catchment drained by the channel (see Section 12). Equation (13) is dimensional and the following units must be used: L, y and W_e in m; S in m/m; N in years; A in m^2; and 2minM5 in mm.

5.4 In the second type of design problem described in paragraph 5.1 it is required to find the size of channel needed to drain a given length of road. For the case of a triangular channel, Equations (13) and (14) can be used to obtain a direct solution for the flow depth:

$$y = 2.60 \times 10^{-2} \left(\frac{nL}{S^{1/2}}\right)^{0.256} r^{-0.171} \times (N - 0.4)^{0.093} \left[\frac{W_e\,(2minM5)}{b}\right]^{0.415} \qquad (15)$$

where the effective cross-fall b and the hydraulic-radius factor r are given by Equations (4) and (7) respectively. For the situation of a rectangular channel of width B_b, the corresponding result is:

$$y = 9.75 \times 10^{-4} \left(\frac{nL}{S^{1/2}}\right)^{0.437} \left(1 + \frac{2y}{B_b}\right)^{0.292} \times (N - 0.4)^{0.158} \left[\frac{W_e\,(2minM5)}{B_b}\right]^{0.708} \qquad (16)$$

In this case, the unknown flow depth y appears on both sides of the equation so a small amount of iteration is needed to find the solution. The units for the quantities in Equations (15) and (16) must be as specified in paragraph 5.3.

5.5 No general equation for directly determining the flow depth y in trapezoidal channels can be obtained because different solutions are possible depending upon the particular values of base width and side-slope chosen. The following design procedure is therefore recommended.

(a) Make an initial estimate of the size and shape of channel needed.

(b) Calculate the flow area A and the hydraulic-radius factor r (from Equation (5)).

(c) Determine the values of m from Equation (11) and G_m from Equation (14).

(d) Calculate from Equation (13) the length L of road that can be drained and compare with the required length.

(e) Revise the channel geometry and repeat steps (a) to (d) until the required drainage length is achieved.

5.6 If it is necessary to determine the critical storm duration that produces the design flow conditions in a channel, a method of estimating the duration is given in Annex A.

5.7 Information on how to calculate or choose suitable values for the various parameters in Equations (13), (15) and (16) are given in Chapters 6 to 14.

6. STORM RETURN PERIOD

6.1 The degree of security against flooding that should be provided by a surface drainage channel needs to be decided on a case-by-case basis.

6.2 On critical lengths of road, it may be necessary to design channels for storms of higher return period than normal in order to reduce the risk of overflowing. Examples are lengths in which a change in super-elevation from one side of the road to the other causes the cross-fall to be locally zero; flooding at such a point might spread across the full width of the road. A higher standard of design may also be appropriate for sections draining to longitudinal sag points where it is important to prevent ponding.

6.3 In the absence of special factors of the type described in paragraph 6.2, it will normally be appropriate for a channel in the verge to be designed to just flow full for a storm with a return period of N = 1 year. The HCD (MCHW 3) and HA 39 (DMRB 4.2) permit the outer edge of a verge channel (farthest from the carriageway) to be set higher than the inner edge, as shown in Figure 2. This allows some surcharging to occur on to the adjacent hardstrip or hardshoulder during rarer storm events. The maximum permissible widths of flooding are 1.0m for all-purpose roads and 1.5m for motorways. As an example, a 1.0m width of flooding on a road with a cross-fall of 1:40 can be allowed by constructing the outer edge of the channel so that it is 25mm higher than the inner edge. Larger flows will cause the channel to overflow into the verge rather than to encroach farther on to the carriageway. Unless there are special factors, it will normally be appropriate to check that a surcharged channel is able to cater for a storm with a return period of N = 5 years without overflowing.

6.4 Channels in the central reserve may need to be designed to a higher standard than those in the verge because it is important to prevent water from encroaching on to the adjacent fast lane or from overflowing on to the opposite carriageway. HA 39 (DMRB 4.2) specifies certain geometric requirements in order to prevent such occurrences. If no surcharging of a channel in the central reserve is permissible, it should normally be designed to flow just full for a storm with a return period of N = 5 years (in the absence of other special factors such as those described in paragraph 6.2).

If the cross-sectional profile in the central reserve does safely permit some surcharging, then the normal design requirements should be similar to those for verge channels: ie, N = 1 year for the channel just flowing full; and N = 5 years for the channel with the permitted amount of surcharging.

6.5 The standard of performance appropriate for critical sections of road (see paragraph 6.2) will depend upon the particular circumstances but design storm return periods of N = 10 years or N = 20 years may be suitable choices.

7. GEOGRAPHICAL LOCATION

7.1 The effect of geographical location on rainfall characteristics is taken into account in Equation (10) by means of the value of 2minM5 - the rainfall depth (in mm) occurring in a storm event with a duration of T = 2 minutes and a return period of N = 5 years. The value of 2minM5 appropriate for any particular road scheme in the UK is obtained from Figure 3 (taken from BS 6367: 1983, Ref 6). This value is then used in the design Equations (13), (15) or (16) as appropriate.

7.2 It should be noted from Figure 3 that the most severe rainfall conditions are to be expected in East Anglia and the South-East of England. Although these areas have much lower values of average annual rainfall than parts of Wales, Scotland and the North-West of England, they experience heavier and more frequent short-duration storms, of the kind typically associated with summer thunderstorms.

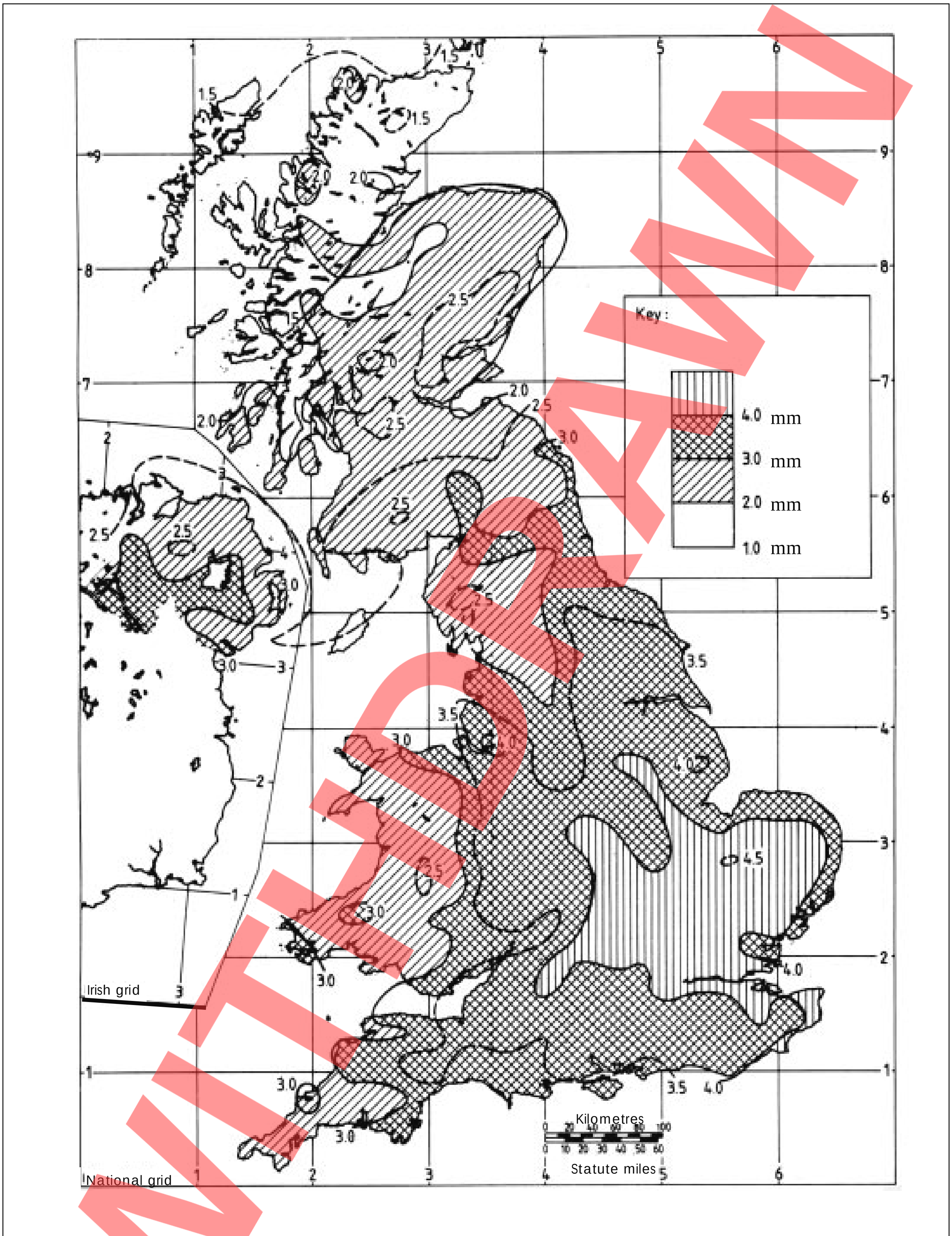

Figure 3 Values of 2min M5 rainfall depth for UK
(Reproduced from BS 6367: 1983 by permission British Standard Institution)

8. CHANNEL GEOMETRY

8.1 The design equations (13), (15) and (16) are valid for all types of triangular, rectangular or trapezoidal channel provided the cross-sectional shape factors b_1, b_2 and B_b (as appropriate) do not vary with the depth of flow or with distance along the drainage length.

8.2 Limits on the permissible depth and side-slopes for surface water channels that may be subject to traffic are given in paragraph 3.1. A trapezoidal channel provides a higher flow capacity than a triangular channel with the same depth and side-slopes and will therefore enable longer lengths of road to be drained between outlets. If a channel is protected or removed far enough from traffic, larger depths or steeper side-slopes can be used; the occurrence of the flow depth y in Equation (13) shows that a relatively deeper channel is more efficient hydraulically than a shallower channel of the same cross-sectional area.

8.3 The base sections of trapezoidal or rectangular channels should be given a 1:40 cross-fall away from the carriageway (see Figure 1) so as to provide self-cleansing characteristics that are similar to those of conventional kerbed channels. The effect of the 1:40 cross-fall on flow capacity is very small and can be neglected; the value of y used in Equations (11), (13) and (16) should be the flow depth measured from the centreline of the channel invert.

8.4 Figure 2 defines the flow depths that may need to be considered when designing a surface water channel. The depth y_1 from the lower inner edge of the channel to the centreline of the invert corresponds to the channel full capacity and is termed the "design flow depth". If water is permitted to encroach on to the adjacent hardstrip or hardshoulder during rarer storms, the "surcharged flow depth" y_3 is the depth from the outer edge of the channel to the centreline of the invert. There should, if possible, be no step between the inner edge of the channel and the top edge of the carriageway. However, if one is formed, it may be necessary in the calculations to take account of the depth y_2 between the top edge of the carriageway and the centreline of the channel invert. In cases where porous asphalt surfacing is used in conjunction with road-edge channels, a step will normally be necessary to allow water to drain from the permeable layer; the vertical distance between the top of the porous asphalt layer and the invert of the channel should not exceed 150mm.

8.5 As described in Chapter 6, surface water channels should be designed to provide specified degrees of security against flooding. As an example, a verge channel might typically be sized so that the design flow depth y_1 provides sufficient flow capacity for storms with a return period of N = 1 year. It might also be specified that the channel should be able to cater for storms of N = 5 years without exceeding the surcharged flow depth y_3. If the second criterion was not satisfied, the size of the channel would need to be increased or the distance between outlets reduced. If no surcharging was permitted (eg, for a channel in the central reserve), it might be specified that the design flow depth y_1 should provide sufficient capacity for storm return periods of N = 5 years. Further guidance on the choice of suitable design criteria is given in Chapter 6.

8.6 If a channel is permitted to surcharge, the cross-sectional shape of the flow area becomes more complex and does not comply with the assumptions given in paragraph 8.1. An approximate method of determining the drainage capacity of surcharged channels is given in Chapter 13.

8.7 In some situations it may be wished to increase the size of a channel part way along a drainage length. In the case of a triangular profile, it is possible to deepen the channel without altering the cross-falls b_1 and b_2, and similarly a rectangular channel can be deepened while keeping the width B_b constant; examples of suitable transitions are shown in Figure 4. The design equations (13), (15) and (16) can be applied to such cases without approximation. The flow capacity of the smaller, upstream channel should be checked to ensure that it is sufficient to drain the length of road upstream of the transition point; the capacity of the larger, downstream channel should be similarly checked using the overall length of road draining to the outlet.

8.8 If a trapezoidal channel is enlarged part way along a drainage length, it is not possible to keep all the shape factors b_1, b_2 and B_b constant. The capacities of the two parts of the channel should be checked in the way described in paragraph 8.7, but the result for the downstream length may be less accurate because the assumption of constant shape is not satisfied. For this reason, it is recommended that the downstream channel should be designed assuming a drainage length 5% greater than the actual length draining to the outlet.

8.9 Transitions of the type shown in Figure 4 should be gradual in order to minimize energy losses. If the invert is lowered, the length of the transition should not be less than 15 times the change in depth. Similarly, the side of a channel should not diverge outwards from the longitudinal centreline at an angle greater than 1:3 in plan.

8.10 Fixed obstructions in a drainage channel, such as a longitudinal line of posts for a safety barrier, will reduce its flow capacity. In order to prevent excessive local disturbance of the flow, no more than 15% of the cross-sectional area of the flow should be blocked by an obstruction if it is located within the downstream half of a drainage length; in the upstream half the blockage should not be more than 25% of the cross-sectional area of the flow. The energy losses produced by a longitudinal line of posts can be taken into account by using a higher value of the Manning resistance coefficient n, as described in paragraph 11.2. Allowance need not normally be made for one or two isolated posts in a particular drainage length.

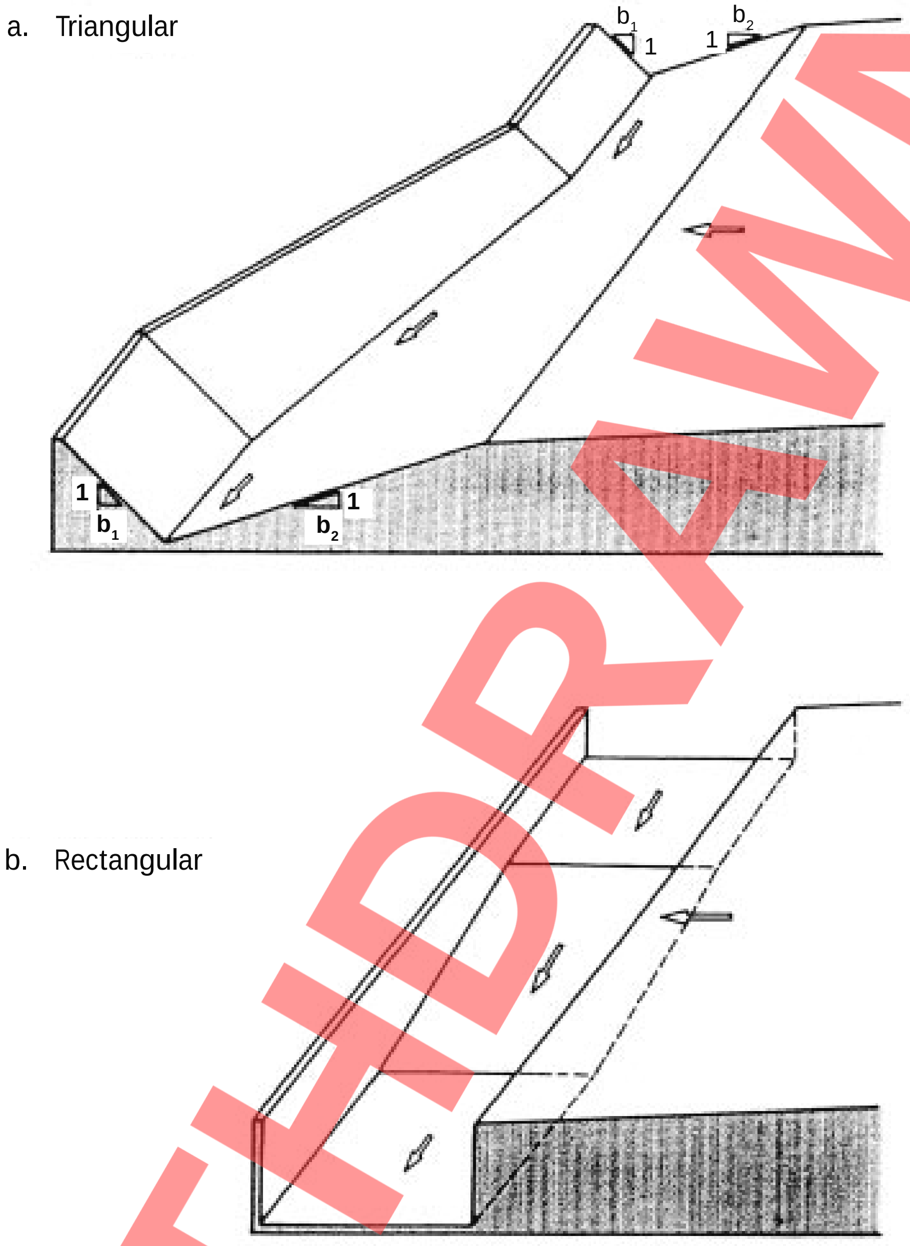

Figure 4 Transitions for channel enlargements

9. CHANNEL GRADIENT

9.1 The longitudinal gradient S of a channel is defined as the vertical fall per unit distance measured along the channel. A channel will normally have the same longitudinal gradient as the adjacent carriageway, but this is not a condition for use of the design method.

9.2 If the gradient of a channel varies with distance, an equivalent value of uniform slope S_e can be calculated for use in the design method described in Chapter 5. To evaluate S_e, values of the local gradient S_j should be determined at eleven equally-spaced points (j = 1 to 11) along the length L of channel draining to a particular outlet: S_1 is the gradient at the upstream end and S_{11} the gradient at the outlet. The distance between adjacent points is not limited to any particular value, but should be equal to L/10. The equivalent value of gradient is calculated from:

$$S_e = 400 \left[S_1^{-1/2} + S_{11}^{-1/2} + 2 \sum_{j=2}^{j=10} S_j^{-1/2} \right]^{-2} \qquad (17)$$

9.3 The design method can be used if the longitudinally-varying gradient is locally zero (but not adverse) at the upstream or downstream end of a channel. In order to obtain a valid result for the equivalent gradient S_e, the zero value S_1 or S_{11} should be replaced in Equation (17) by:

$$S_1 = S_2 / 9 \qquad (18a)$$

or

$$S_{11} = S_{10} / 9 \qquad (18b)$$

9.4 The design method is not valid if the gradient becomes zero at an intermediate point between the upstream and downstream ends of a length of channel. In such cases, it is necessary to place an outlet at the intermediate point and design the channel as two separate lengths.

10. DRAINAGE LENGTHS

10.1 The drainage length L is the distance along a channel between two adjacent outlets on a continuous slope, or the distance between a point of zero slope and the downstream outlet.

10.2 Surface water channels can be used most effectively if their layout is considered at an early stage in the design of a road scheme. Where possible, horizontal and vertical alignments should be chosen so that suitable drainage lengths can be defined taking into account the location of outlets discharging to natural watercourses.

10.3 If the drainage length to a natural watercourse requires too large a channel capacity, intermediate outlets should be used to remove water from the road-edge channel. The flow from the outlets may be conveyed to a suitable discharge point by carrier pipes or an open ditch.

11. CHANNEL ROUGHNESS

11.1 The hydraulic resistance of a road-edge channel depends upon its surface texture, the standard of construction, and the presence of deposited silt and grit. Recommended values of the Manning roughness coefficient for use in conjunction with the design equations are given in Table 1.

Channel Type	Condition	n
Concrete	average	0.013
Concrete	poor	0.016
Black top	average	0.017
Black top	poor	0.021

Table 1 Values of Manning's n

Further information about the factors influencing the hydraulic resistance is given in Annex B.

11.2 Posts located in a surface water channel will disturb the flow of water and produce additional energy losses. The effect of a line of posts on flow capacity can be estimated by increasing the appropriate value of roughness coefficient from paragraph 11.1 by an amount n_p given by:

$$n_p = 0.7\left[\frac{I}{gL_p}\left(\frac{A_p}{A}\right)\right]^{1/2}\left(\frac{ry}{m+1}\right)^{2/3} \qquad (19)$$

where g is the acceleration due to gravity (=9.81 m/s^2), L_p is the average distance between successive posts (in m) and A_p is the wetted cross-sectional area (in m^2) of a post normal to the flow, when the cross-sectional area of the flow is A (in m^2). The derivation of Equation (19) is explained in Annex B.

12. CATCHMENT WIDTH

12.1 The effective catchment width W_e is equal to the width of the carriageway drained, plus the width of the channel and any other impermeable surface draining to the channel, plus an allowance (if appropriate) for run-off from a cutting.

12.2 The design method assumes that W_e does not vary along a particular drainage length. However, minor local variations can be allowed for by using an average width, calculated by dividing the total effective area draining to an outlet by the drainage length L.

12.3 It is assumed that 100% run-off occurs from concrete and black-top surfaces during design storms.

12.4 Surface channels may be used to collect run-off from cuttings as well as from roads. The amount of run-off from a cutting depends upon many factors including its height, slope, soil type and antecedent wetness, and also upon the quantity of rainfall and the direction of the wind.

12.5 Information on the additional run-off contributed by cuttings is very limited. In the absence of suitable field data, the effective width W_e of a catchment should be calculated from the formula:

$$W_e = W + \alpha \bar{C} \qquad (20)$$

where W is the width of the impermeable part of the catchment (m), α is the run-off coefficient for the cutting, and is the average plan width of the cutting (m) drained by the length of channel being considered. Details of the derivation of Equation (20) are given in Annex C.

12.6 Suitable values of the run-off coefficient α can be estimated from Table 2. These figures contain some allowance for the relative steepness of road cuttings, which may result in more run-off than from equivalent natural catchments. For design purposes it is assumed that the antecedent wetness of the cutting is dependent upon the average annual rainfall at the site. Appropriate choices of antecedent wetness for Northern Ireland, Scotland, Wales and English Counties are given in Table 3.

Soil Type	Antecedent Wetness	α
High permeability	low	0.07
	medium	0.11
	high	0.13
Medium permeability	low	0.11
	medium	0.16
	high	0.20
Low permeability	low	0.14
	medium	0.21
	high	0.26

Table 2 Run-off coefficients for cuttings

Low	Medium	High
Bedfordshire	Berkshire	Northern Ireland
Buckinghamshire	Cleveland	Scotland
Cambridgeshire	Derbyshire	Wales
Essex	Durham	Avon
Greater London	East Sussex	Cheshire
Hertfordshire	Hampshire	Cornwall
Norfolk	Hereford & Worcester	Cumbria
Rutland	Humberside	Devon
Suffolk	Isle of Wight	Dorset
	Kent	Gloucestershire
	Leicestershire	Greater Manchester
	Lincolnshire	Lancashire
	North Yorkshire	Merseyside
	Northamptonshire	Somerset
	Northumberland	Wiltshire
	Nottinghamshire	
	Oxfordshire	
	Shropshire	
	South Yorkshire	
	Staffordshire	
	Surrey	
	Tyne & Wear	
	Warwickshire	
	West Sussex	
	West Yorkshire	

Table 3 Antecedent wetness categories

The basis of the data in Tables 2 and 3 is explained in Annex C.

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13. SURCHARGED CHANNELS

13.1 As described in Chapters 6 and 8, some surcharging of surface drainage channels may often be permissible. The type of channel geometry shown in Figure 2 allows storms of higher return period to be accommodated without overflowing and without causing water to encroach beyond the edge of the hardstrip or hardshoulder.

13.2 Surcharged channels have compound cross-sectional shapes and do not comply with all the assumptions given in paragraph 8.1 for the validity of the design equations in Chapter 5. However, an approximate estimate of the length of road that may be drained by a surcharged channel can be obtained by defining an equivalent channel that has the same values of overall depth, cross-sectional area and flow capacity. The following procedure refers to the general case of the trapezoidal channel shown in Figure 2. However, the equations can equally be applied to the case of a triangular channel (with $B_b = 0$) or of a rectangular channel (with $b_1 = 0 = b_2$).

13.3 First calculate the cross-sectional area A of the surcharged channel which is given by:

$$A = \frac{1}{2}\left[\begin{array}{l}(b_1 + b_2)y_3^{\,2} - b_2(y_3 - y_1)^2 \\ + b_3(y_3 - y_2)^2 + 2B_b y_3\end{array}\right] \qquad (21)$$

where b_3 is the transverse slope of the carriageway adjacent to the channel. As explained in paragraph 8.4, it is preferable if channels can be constructed without a step between the inner edge of the channel and the top edge of the carriageway; in such cases $y_2 = y_1$ in Equation (21). The depths y_1, y_2 and y_3 are all measured relative to the centreline of the invert, see paragraph 8.4. The corresponding hydraulic-radius factor r (= surface width/wetted perimeter) is determined from:

$$r = \frac{b_1 y_3 + b_2 y_1 + b_3(y_3 - y_2) + B_b + (y_2 - y_1)}{(b_1^{\,2} + 1)^{1/2} y_3 + (b_2^{\,2} + 1)^{1/2} y_1 + (b_3^{\,2} + 1)^{1\,2}(y_3 - y_2) + B_b + (y_2 - y_1)} \qquad (22)$$

Also required is the hydraulic conveyance factor K for the surcharged channel which is given by:

$$K = \frac{3}{8}\left[\begin{array}{l}(b_1 + b_2)y_3^{\,8/3} - b_2(y_3 - y_2)^{8/3} \\ + \left(\frac{n}{n_c}\right)b_3(y_3 - y_2)^{8/3} + \frac{8}{3}B_b y_3^{\,5/3}\end{array}\right] \qquad (23)$$

where n_c is the Manning roughness coefficient of the carriageway.

13.4 The next step is to determine the shape factor m, see Equation (11), for an equivalent channel having the same values of y_3, A and K as the surcharged channel. This is done by calculating the value of the quantity:

$$X = \frac{K}{y_3^{\,2/3} A} \qquad (24)$$

from which m can be determined using:

$$m = \frac{1}{2}\left[X - 1 + \left(X^2 + \frac{14}{3}X + 1\right)^{1/2}\right] \qquad (25)$$

Values of m for the equivalent channels may sometimes be slightly greater than unity but this does not affect the design procedure.

13.5 Having determined values of y_3, A, r and m for the equivalent channel, the length of road that may be drained is calculated using the equations given in paragraph 5.3.

14. BY-PASSING AT OUTLETS

14.1 Recommendations on the types and sizes of outlet to be used with surface water channels are contained in HA 78 (DMBR 4.2). The flow rate approaching an outlet is given by Equation (3), (6) or (8) depending upon the particular cross-sectional shape of the channel. If the longitudinal gradient of the channel varies with distance, the effective slope S_e (see paragraph 9.2) should be substituted for S in these equations.

14.2 A more economic overall design of drainage system may often be achieved by allowing a certain degree of flow by-passing to occur at intermediate outlets; HA 78 specifies that collection efficiencies between 80% and 100% can be used in such cases. The design equations given in Chapter 5 assume that there is no inflow at the upstream end of a drainage length. Further analysis of systems with and without by-passing indicates that the additional inflow resulting from up to 20% by-passing will not normally produce any increase in the peak design flow occurring at the downstream end of a drainage length. This is because the by-pass flow does not arrive at the next outlet until later in the storm when the rainfall intensity has already started to decrease. By-passing will therefore produce two peak flows at the downstream end of a drainage length but the second peak will normally be smaller than the first.

14.3 Although the general conclusion is that by-passing will not normally increase peak design flows in drainage channels, there may be some special cases where its effect could be significant. One possible example is by-passing from a long channel into a much shorter one, thus producing an inflow that is large compared with the amount of water collected by the downstream channel. To provide a simple rule that should be safe for almost all cases, it is recommended that the following allowance should be made when designing any channel subject to by-passing. If the upstream and downstream channels have lengths of L_1 and L_2 respectively, the downstream channel should be designed according to the recommendations in Chapter 5 so that it is able to drain a length:

$$L = L_2 + \frac{1}{2}(1-\eta)L_1 \qquad (26)$$

where η is the collection efficiency of the outlet draining the upstream channel; note that an efficiency of 80% corresponds to a value of $\eta = 0.80$ in Equation (26).

15. CONSTRUCTION TOLERANCES

15.1 The effect of allowable construction tolerances on the capacity of a channel should be considered in design. As an approximate guide, flow capacity will be within ±5% of the design capacity if the local channel gradient is within ±10% or the depth of the channel within ±2% of the required values. In general, the design capacity of the channel should be determined for the combination of tolerances that give minimum channel capacity. Adherence to tolerances is most important at the downstream ends of drainage lengths where the flows are greatest.

16. WORKED EXAMPLES

16.1 Determine the length of road that can be drained by a triangular surface water channel having the following characteristics.

Symmetrical cross-falls: $b_1 = b_2 = 5$

Channel depths (see Fig 2): $y_1 = 0.120\text{m}$
$y_3 = 0.145\text{m}$

Longitudinal channel gradient:
$S = 1/200 = 0.005$

Manning's roughness coefficient: $n = 0.013$
(concrete, average condition)

The value of the hydraulic-radius factor corresponding to the design flow depth y_1 is calculated from Equation (7) to be:

$$r = \frac{5 + 5}{2\left(1 + 5^2\right)^{1/2}} = 0.981$$

The width of flow corresponding to the design flow depth y_1 is:

$B = 10 \times 0.120 = 1.200\text{m}$

and the corresponding flow area is:

$A = ½\ B\ y_1 = ½ \times 1.200 \times 0.120 = 0.0720\ \text{m}^2$

The shape parameter m of the channel (see Equation (11) and paragraph 5.2) has a value of:

$$m = \frac{1.20 \times 0.120}{0.0720} - 1 = 1.00$$

The width of the two-lane carriageway drained by the channel is 9.300m (including two 1.000m wide hardstrips). The overall width of the channel itself is:

$B = b_1 y_3 + b_2 y_1 = 5 \times 0.145 + 5 \times 0.120 = 1.325\text{m}$

The road is on embankment and there is no run-off from the verge into the channel. The effective catchment width is therefore:

$W_e = 9.300 + 1.325 = 10.625\text{m}$

The road is located near Coventry and from Figure 3 it is found that the characteristic value of rainfall depth occurring in 2 minutes with a return period of 5 years is:

2minM5 = 4.0mm

The channel is to be designed so that the design flow depth y_1 is not exceeded by run-off from storms occurring once every year on average; the design return period is therefore:

N = 1 year

The length of road that can be drained by the channel is calculated from Equation (13), in which the factor G_m corresponding to the triangular shape of the channel is obtained from Equation (14) as:

$G_m = 2.90 \times 10^6\ (2.65 - 1.00) = 4.79 \times 10^6$

The maximum drainage length is therefore:

$$L = 4.79 \times 10^6\ \frac{(0.005)^{1/2}}{0.013}\ (0.981 \times 0.120)^{2/3}$$
$$\times\ (1.0 - 0.4)^{-0.362} \left[\frac{0.0720}{10.625 \times 4.0}\right]^{1.62} = 244\text{m}$$

The critical storm duration corresponding to the design flow condition can be estimated from Equation (A.1) in Annex A as:

$$T_c = 0.085 \left[\frac{0.013 \times 244}{0.005^{1/2}}\right] (0.981 \times 0.120)^{-2/3} = 15.9 \text{ minutes}$$

16.2 Consider the example in paragraph 16.1 for the same road and channel but constructed in a cutting which contributes run-off to the channel. The road is located in Warwickshire so from Table 3 the antecedent wetness is "medium". The soil in the cutting is a fairly heavy clay with a low permeability so from Table 2 the run-off coefficient is $\alpha = 0.21$. The average width of the cutting draining to the channel is $\bar{C} = 15.0\text{m}$. Compared to the example in 16.1, the effective catchment width is increased to the following value given by Equation (20):

$W_e = 10.625 + 0.21 \times 15.0 = 13.775\text{m}$

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All the other parameters in Equation (13) are unchanged so the revised length of road that can be drained by the channel is:

$$L = 244 \left(\frac{10.625}{13.775}\right)^{1.62} = 160\text{m}$$

16.3 Determine the length of road that can be drained by a trapezoidal channel having the following characteristics.

Base width of channel:	$B_b = 0.300$m
Symmetrical cross-falls:	$b_1 = b_2 = 5$
Channel depths (see Fig 2):	$y_1 = 0.150$m $y_3 = 0.175$m
Longitudinal channel gradient:	$S = 1/200 = 0.005$
Manning's roughness coefficient: (concrete, average condition)	$n = 0.013$

The value of the hydraulic-radius factor given by Equation (5) for the design flow depth y_1 is:

$$r = \frac{0.300 + (5 + 5) \times 0.150}{0.300 + 2\left(1 + 5^2\right)^{1/2} \times 0.150} \quad 0.984$$

The width of flow corresponding to the design flow depth y_1 is:

$$B = 0.300 + 10 \times 0.150 = 1.800\text{m}$$

and the corresponding flow area is:

$$A = B_b y_1 + \tfrac{1}{2}(b_1 + b_2)y_1^{\,2}$$

$$= 0.300 \times 0.150 + \tfrac{1}{2}(5 + 5) \times 0.150^2 = 0.158\text{m}^2$$

The shape parameter of the channel has from Equation (11) the value:

$$m = \frac{1.800 \times 0.150}{0.158} - 1 = 0.71$$

The width of four-lane motorway drained by the channel is 17.900m (including a 3.300m hardshoulder). The road is on embankment and there is no run-off from the verge into the channel. The effective catchment width is therefore:

$$W_e = 17.900 + 1.925 = 19.825\text{m}$$

The section of motorway is located near Watford and from Figure 3 the characteristic rainfall depth is:

2minM5 = 4.1mm

The channel is to be designed so that the design flow depth y_1 is not exceeded by run-off from storms with a return period of N = 1 year. The shape factor G_m for the channel is obtained from Equation (14) as:

$$G_m = 2.90 \times 10^6 \,(2.65 - 0.71) = 5.63 \times 10^6$$

The maximum length of road that can be drained by the channel is calculated using Equation (13):

$$L = 5.63 \times 10^6 \,\frac{(0.005)^{1/2}}{0.013}(0.984 \times 0.150)^{2/3}$$

$$\times\ (1.0 - 0.4)^{-0.362} \left[\frac{0.158}{19.825 \times 4.1}\right]^{1.62} = 417\text{m}$$

16.4 Along part of the motorway considered in paragraph 16.3, superelevation causes run-off to drain from one of the carriageways towards the central reserve. Both sides of the central reserve are protected by safety barriers so it is possible to use a channel of rectangular cross-section in this location. The width of the channel is chosen to be $B_b = 1.000$m and it is required to determine the design depth of flow when draining a maximum distance of L = 300m. The effective catchment width (carriageway + hardshoulder + channel) is:

$$W_e = 17.900 + 1.000 = 18.900\text{m}$$

The values of n, S and 2minM5 are as given in paragraph 16.3. Since the channel is not permitted to surcharge on to the carriageway that it drains, it is decided to determine the design depth of flow y of the channel for storms with a return period of N = 5 years (see paragraph 6.4). The value of y for a rectangular channel is determined from Equation (16) but it should be noted that y also appears on the right-hand side of the equation. A short iterative procedure is therefore necessary as illustrated by the following calculations.

Guess a likely value for the design flow depth, eg y = 0.150m, and substitute this on the right-hand side of Equation (16) so that:

$$y = 9.75 \times 10^{-4} \left(\frac{0.013 \times 300}{0.005^{½}} \right)^{0.437} \left(1 + \frac{2 \times 0.150}{1.000} \right)^{0.292}$$

$$\times (5 - 0.4)^{0.158} \left[\frac{18.900}{1.000} \times 4.1 \right]^{0.708} = 0.168\text{m}$$

Substituting this calculated value of y on the right-hand side of the equation gives a revised value of y = 0.169m; one final iteration converges to the solution y = 0.170m, which is the required design depth of flow in the 1.0m wide rectangular channel.

17. GLOSSARY OF SYMBOLS

		Units
A	Cross-sectional area of flow	m^2
A_p	Cross-sectional area of post	m^2
B	Surface width of flow	m
B_b	Base width of channel	m
b	Effective cross-fall of channel (rate of increase of surface width per unit depth)	
b_1	Slope of side of channel remote from carriageway	-
	(1 unit vertical: b_1 units horizontal)	-
b_2	Slope of side of channel adjacent to carriageway	-
	(1 unit vertical: b_2 units horizontal)	-
b_3	Transverse slope of carriageway adjacent to channel	
	(1 unit vertical: b_3 units horizontal)	-
$\bar{C}$	Average plan width of cutting drained by channel	m
c	Coefficient in Equation (12)	variable
C_d	Drag coefficient of post	-
G_m	Coefficient defined by Equation (14)	-
g	Acceleration due to gravity	m/s^2
I_o	Mean rainfall intensity	mm/h
j	Integer indicating section number	-
K	Hydraulic conveyance factor, Equation (23)	$m^{8/3}$
L	Drainage length of channel; distance between outlets	m
L_p	Average distance between posts	m
L_1	Value of L for upstream channel	m
L_2	Value of L for downstream channel	m
m	Shape factor of channel defined by Equation (11)	-
N	Return period of storm	years
n	Manning roughness coefficient of channel	$s/m^{1/3}$
n_c	Manning roughness coefficient of carriageway	$s/m^{1/3}$
n_p	Additional roughness coefficient due to posts	$s/m^{1/3}$
P	Wetted perimeter of channel	m
PIMP	Percentage impermeable area of catchment	%
PR	Percentage run-off coefficient	%
Q	Flow rate at downstream end of channel	m^3/s
R	Hydraulic radius of flow (= A/P)	m
r	Hydraulic-radius factor (= B/P)	-
S	Longitudinal gradient of channel (vertical fall per unit distance along channel)	m/m
S_e	Effective value of S for channel with non-uniform slope	m/m

		Units
S_j	Local value of S	m/m
SOIL	Soil index	-
T	Duration of storm	minutes
T_c	Critical storm duration for channel	minutes
UCWI	Urban Catchment Wetness Index	-
W	Width of impermeable part of catchment	m
W_e	Effective width of whole catchment	m
X	Factor defined by Equation (24)	-
y	Design depth of flow	m
y_1	Depth of channel from lower edge of carriageway to centreline of invert (see Figure 2)	m
y_2	Depth of channel from top edge of carriageway to centreline of invert (see Figure 2)	m
y_3	Overall depth of surcharged channel to centreline of invert (see Figure 2)	m
α	Run-off coefficient for cutting	-
η	Collection efficiency of outlet (= flow rate collected by outlet / flow rate approaching outlet)	-
2minM5	Rainfall depth occurring in 2 minutes with return period of 5 years	mm

18. REFERENCES

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   HA 78 Design of Outfalls for Surface Water Channels (DMRB 4.2)

   TD 19 Safety Fences and Barriers (DMRB 2.2).

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19. ENQUIRIES

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ANNEX A RAINFALL DATA

A.1 The Meteorological office is able to provide individual data on rainfall characteristics for any location in the UK. However, in order to produce a general design method for surface water channels, it is necessary to be able to describe the rainfall characteristics by means of an equation relating mean intensity to the duration and return period of the storm event.

A.2 Relevant information on short-period storms with durations between 1 minute and 10 minutes was provided by the Met Office for use in British Standard BS 6367: 1983 [Ref 6]. The following general calculation procedure is given in Annex A of BS 6367:

(a) Determine from a map of the UK (reproduced in this document as Figure 3) the value of rainfall depth (2minM5) occurring in 2 minutes with a return period of 5 years at the chosen location.

(b) Calculate (using a table and the value of 2minM5) the rainfall depth for the required duration T but still with a return period of 5 years (ie TminM5)

(c) Calculate (using a graph and the value of TminM5) the rainfall depth for the required duration T and return period N (ie TminMN).

(d) Divide the rainfall depth TminMN by the duration to give the mean intensity I_0.

A.3 Equation (10) for predicting I_0 was obtained for this document by curve-fitting the tabular and graphical data corresponding to steps (b) to (d) above. The applicability of the equation to storm durations greater than 10 minutes was checked using data in Volume II of the Flood Studies Report [Ref 7]. The equation is optimized for values of T = 2-20 minutes and N = 1-20 years and tends to overestimate I_0 (ie err on the safe side) outside these ranges. The recommended upper limits for use of Equation (10) are T = 30 minutes and N = 50 years.

A.4 The critical storm duration T_c that gives rise to the design flow conditions in a particular channel can be estimated from:

$$T_c = 0.085 \left(\frac{nL}{S^{1/2}}\right)(ry)^{-2/3} \qquad \text{(A.1)}$$

where T_c is in minutes and the units for the other quantities are as defined in paragraph 5.3.

ANNEX B CHANNEL ROUGHNESS

B.1 The Manning resistance equation is appropriate when a flow is rough-turbulent (ie, with its resistance mainly determined by the surface texture of the channel). This is likely to be the case in most road drainage channels, except perhaps near the upstream end where the velocity or depth of flow is small and the flow may be smooth-turbulent (ie, with its resistance mainly determined by the viscosity of the water). It is often assumed that the roughness coefficient n depends only upon the surface texture of the channel, but experimental evidence indicates that it can vary with the relative depth of flow, the cross-sectional shape of the channel and the intensity of any lateral inflow.

B.2 A modified version of Manning's equation for shallow triangular channels was developed by Izzard [Ref 8] and is recommended by the US Federal Highway Administration. The equation has the form:

$$Q = \frac{3}{8} \frac{bS^{1/2}}{n} y^{8/3} \qquad (B.1)$$

and is obtained by applying Manning's equation to vertical elements in the cross-section and integrating the discharge across the channel; no allowance is made for the resistance of a vertical kerb. Equation (B.1) gives capacities that are approximately 20% higher than the conventional version in Equation (6). Values of n quoted in the literature for triangular channels therefore depend upon which of the two formulae were used to analyse the experimental data.

B.3 The conventional form of Manning's equation has been used in this document (ie, Equations (1), (3), (6) and (8)) so as to provide a common basis for all shapes of channel. Values of n given in the literature vary typically from 0.010 to 0.017 for concrete channels and from 0.012 to 0.022 for asphalt channels. In Table 1 the recommended figures for "average" condition correspond approximately to the mean of the published values; the figures for "poor" condition are slightly less than the corresponding maximum values.

B.4 Factors which will tend to increase the resistance of a channel are lateral inflow from the road surface and the presence of silt and grit in the invert. Data on these effects are not available, but it is probable that at the downstream ends of channels (which are the most critical points) they are not very large in relation to the uncertainties in the basic n values. An "average" value of n = 0.013 for a concrete channel would be appropriate if it has a trowel-type finish, no sharp discontinuities in line or elevation, and is regularly cleaned.

B.5 An approximate procedure is given in Chapter 13 for applying the design method for simple cross-sectional shapes to the case of surcharged compound channels. It was found that a direct solution for compound channels could be obtained only when using a development of the modified Manning's equation in Equation (B.1). The relationships in Equations (21) to (25), between the relevant hydraulic characteristics (flow capacity and storage capacity) for a compound channel and an equivalent "simple" channel, are therefore based on the modified form of Manning's equation.

B.6 Equation (19) in paragraph 11.2 for estimating the extra flow resistance produced by a longitudinal line of posts is obtained by considering the drag force acting on each post. The factor of 0.7 in the equation corresponds to a drag coefficient of $C_d = 1.2$ with an allowance for the effect of varying water depth between the upstream and downstream ends of a channel.

ANNEX C RUN-OFF FROM CUTTINGS

C.1 Measurements of run-off from lengths of motorway in cutting were made in the UK by the Transport Research Laboratory and reported by Swinnerton [Ref 9]. The published data do not enable the relative contributions of the pervious and impervious areas to be assessed.

C.2 In the absence of other suitable data, it was decided to use an existing run-off equation to provide a method for estimating run-off from road cuttings. The selected equation was developed during studies for the Wallingford Procedure [Volume 1 of Ref 10] as a means of predicting surface run-off to urban storm sewers in the UK, and has the form:

$$PR = 0.662\ PIMP + 0.00219\ (100 - PIMP).\ SOIL.\ UCWI \qquad (C.1)$$

where PR is the percentage run-off from the whole catchment, PIMP is the percentage of impervious area, SOIL is a number related to the infiltration potential of the soil and UCWI is an index of the urban catchment wetness. [A different run-off formula was finally adopted for the Wallingford Procedure, but Equation (C.1) is more suitable for application to roads in cuttings].

C.3 The effective width W_e of a road in cutting is defined as the equivalent width of road which will produce the same total amount of run-off as a road of width W and a cutting of average width $\bar{C}$. From Equation (C.1) it can be shown that:

$$W_e = W + \left(\frac{SOIL.UCWI}{300}\right)\bar{C} \qquad (C.2)$$

C.4 In the Flood Studies Report [Ref 7], soils in the UK were classified according to their infiltration potential into five classes and assigned values of the SOIL parameter between 0.15 and 0.5; the lowest value would apply to a well-drained sandy soil and the highest to a rocky soil on a fairly steep slope. Maps showing regional distributions of SOIL classes are given in Volume 5 of Ref 7 and Volume 3 of Ref 10; the type of soil in a cutting should, however, be assessed from a site survey since the maps do not identify small local variations. Cuttings for roads are steeper than most natural catchments, and for a given soil type may produce relatively more run-off. Approximate allowance can be made for this by classifying a soil in a category of lower infiltration potential than normal; values of the SOIL parameter for cuttings might therefore be expected to be in the range 0.3 to 0.6.

C5 Design values of UCWI for use with Equation (C.1) are given in Volume 1 of Ref 10 as a function of the standard average annual rainfall at a site. Representative values for UK regions were obtained by finding the maximum range of UCWI within each region (excluding only peaks in highland areas), and adopting a figure one third of this range below the maximum value. The regions were then grouped into categories of high, medium and low catchment wetness, and an average representative value of UCWI calculated for each group.

C.6 The values of the run-off coefficient α in Table 2 were obtained from Equation (C.2) using figures of SOIL = 0.3, 0.45 and 0.6 for soils of high, medium and low permeability, and representative values of UCWI = 71, 107, and 132 for low, medium, and high categories of antecedent wetness.

C.7 Site data should be collected where possible to improve the estimate of run-off from the cuttings.