CHAPTER 12

# OPEN CHANNELS

## STORMWATER CHANNELS

Open stormwater channels provide an economical and sometimes essential alternative to closed drains. It is a fact that open channels offer less frictional resistance due to the smaller wetted perimeter, and consequently require smaller cross sections, than closed conduits. The factor of safety against flooding of a channel is generally greater than that of a pipeline. A small water level rise in a channel increases the discharge capacity disproportionately (discharge is nearly proportional to flow depth to the power of 5/3) whereas in the case of a pipe, the discharge capacity is only proportional to the square root of the head. Thus rapid overflowing of manholes associated with closed drains and excess surface flow will result with storms more severe than the design storm.

An open channel can be an obstruction or hazard, especially if wide or deep. Open channels may also require regular maintenance due to deposits or vegetation growth. On the other hand, grassed and contoured channels can be attractive. Shallow channels can form natural barriers between traffic and pedestrians without being dangerous.

Above all, channels are more economical than closed drains. Where land is not too valuable and space is allowed at planning stage, channels may be the solution. Away from built up areas, natural watercourses are generally employed, although protective works may be necessary to avoid erosion.

## FLOW CLASSIFICATION

The depth of flow in a channel will depend on the flow rate, slope, cross sectional shape and boundary roughness. Even then the water depth and velocity can change in time and space. This may be due to variation in flow rate, non-equilibrium or variations in channel alignment and cross section. Thus the assumption of steady, uniform flow consistent with the discharge rate is often incorrect and depth requirements may be greater than indicated by uniform flow theory.

Methods for the analysis of different types of flow vary with the circumstances. In some cases energy principles are employed and in other cases momentum principles. Analysis of many conditions, such as rapidly varied flow, transient flow and transitions, are beyond the scope of this text. Many dynamic systems require numerical methods of analysis. Here computer models may be of use or else the engineer may write his own program. There are many good textbooks on basic hydraulics of open channels (Chow, 1959; Henderson, 1966), and equally suitable texts on numerical methods of analysis (Abbott, 1979 ; Connor and Brebbia, 1976). The basic methods of flow analysis sketched here are intended for straightforward conduits. As well as listing some standard classifications of flow profiles, the reader is told of the more complicated problems should he need to consider them in designing a drainage system. Some more common types of flow such as gradually varied flow receive more coverage, and the engineer should be able to select a channel shape and cross sectional area. Although erosion is mentioned, channel linings are generally the construction engineer's problem, and as such are not covered here. The flow classifications following are illustrated in Figure 12.1.

*Uniform flow* is that which does not vary in space i.e. depth is constant along a regular channel.

*Steady flow* does not vary in time.

*One-dimensional flow* means that flow is in one direction (along the axis of the conduit) and lateral and vertical velocity components are neglected. Rapid changes in depth are therefore excluded from the methods of analysis based on this assumption. It is also assumed that the mean velocity across a section is representative and that pressure is represented by the hydrostatic head.

Unsteady flow implies that flow velocity and depth vary significantly in time. This may be in the form of transient surges or waves. If flow is not uniform, it varies (along the conduit). It may be *gradually varied* such as in a backwater caused by a weir, or *rapidly varied* such as at a hydraulic jump.

Uniform flow implies an equilibrium along the channel. Thus if there is a slope to the channel the energy consumed by friction should equal the fall in bed level along any length. The steeper the channel, the greater should be the energy gradient. In order to increase the energy loss for any channel section and discharge rate, velocity must increase and depth must decrease. The equilibrium depth is referred to as the *normal depth*.

If the Froude number $F_r = v/\sqrt{gy}$ (12.1)

for a rectangular channel is greater than unity the flow is said to be *supercritical*. The water velocity is then faster than the speed of

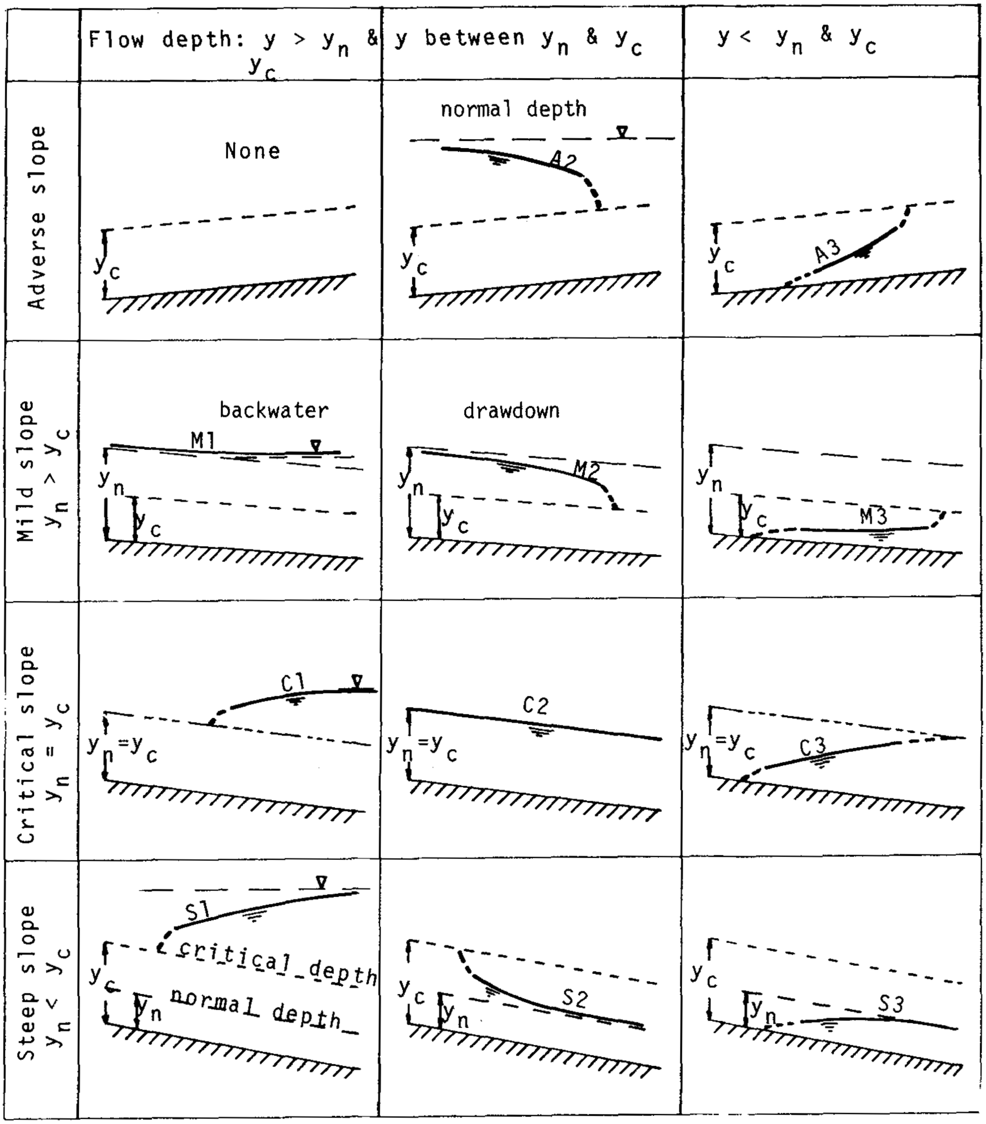



Fig. 12.1  Classification of gradually varied flow profiles.

a wave and violent fluctuations in water level are possible at changes in alignment. *Subcritical flow* occurs if the Froude member is less than unity. The minimum energy per unit weight of water occurs when the depth is *critical*. This depth may be determined by differentiating the equation for energy:

$$E = z + p/w + v^2/2g \qquad (12.2)$$

where z is the elevation of the bed above any selected datum p is the pressure, equal to $\bar{y}w$ in open channels, $\bar{y}$ is the depth to the centroid, w is the unit weight of water, v is the flow velocity and g is gravitational acceleration.

*Specific energy* is generally taken as the unit energy without the z term i.e. it is a local property dependent on the depth of flow only, provided discharge and cross section are specified.

Differentiating this expression with respect to velocity v and setting this equal to zero, one can prove that the critical depth $y_c$ is given by the expression

$$y_c = \sqrt[3]{q^2/g} \qquad (12.3)$$

The specific momentum of flow in a rectangular channel is

$$M = \frac{q^2}{gy} + \frac{y^2}{2} \qquad (12.4)$$

where q is the discharge per unit width.

In situations where there is a rapid change in depth, there may be a considerable energy loss, but there must be momentum balance across the interface for equilibrium. This happens when supercritical flow meets subcritical flow; a discontinuity is formed, termed a hydraulic jump. Employing this concept, one can prove that the relationship between the sequent depths before and after a hydraulic jump is

$$\frac{y_2}{y_1} = \tfrac{1}{2}\left(\sqrt{1 + 8\,F_{r1}^{\,2}} - 1\right) \qquad (12.5)$$

The location of a hydraulic jump can usually only be determined by backwater calculations, proceeding upstream from a known control section for the subcritical flow and downstream for the supercritical flow. The jump will occur where the sequent depth to either assumed depth is equal to the depth computed from the other end. Allowance can be made for the length of the jump.

The energy loss at a jump is determined by comparing the specific energy on each side. In fact although there are two alternate depths (termed conjugate depths) for any energy level, there is only one possible specific energy for any water depth.

The relationship between depth and specific energy is illustrated in general form in Fig. 12.2b and a similar relationship for the momentum function in Fig. 12.2c.

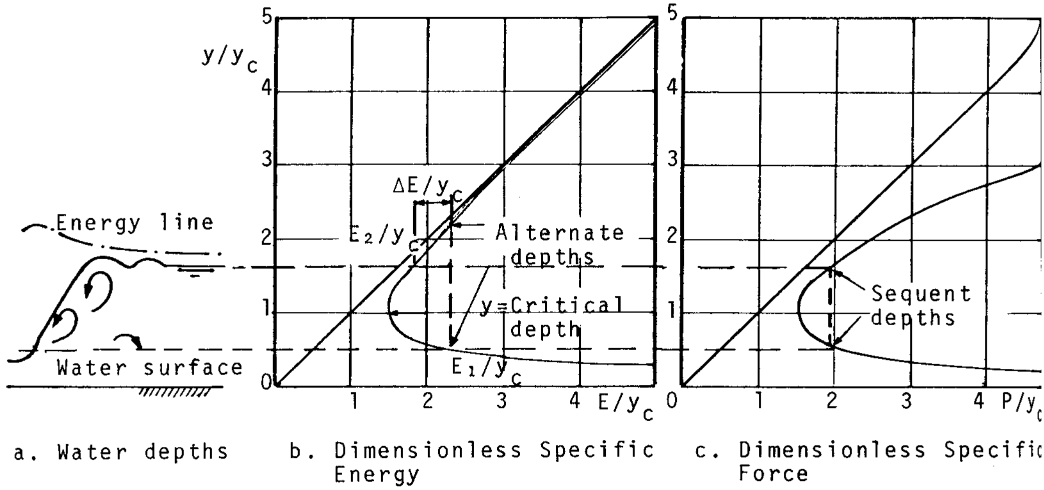



Fig. 12.2 Specific energy and momentum function for rectangular channels.

CHANNEL CROSS SECTION

The cross sectional shape of a channel should be designed for optimum flow. The bigger the wetted perimeter the higher the friction drag. Conversely the smaller the cross-sectional area the more economical the section as a general rule. The maximum discharge is obtained if the wetted perimeter is minimized. Alternatively the minimum cross-sectional area is obtained for any flow if the wetted perimeter is minimized. The geometric properties of various channel shapes are summarized in Fig. 12.3.

The so-called optimum shape or one with minimum area for a given hydraulic radius is a semi-circle. This may be proved by differentiation of the expression for wetted perimeter. Construction procedures are orientated to trapezoidal shapes and the optimum trapezoidal shape is one into which fits a semi-circle, i.e. the base width is $b = 2y/\sqrt{3}$ (Fig. 12.4b). The associated side slopes are 60° to the horizontal which is impractical for construction purposes. The sides are normally cut at slopes as steep as 1/1 for shallow channels in hard clay, reducing for looser non-cohesive materials. 1/1.5 is representative for

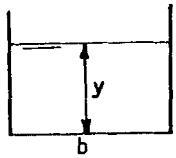
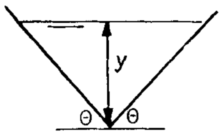
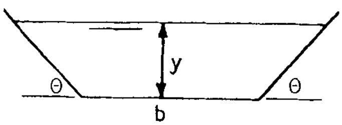
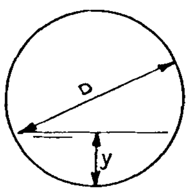
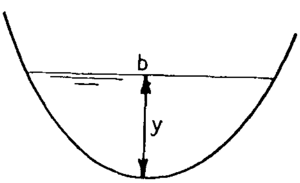
| | Shape | Area A | Wetted Perimiter P |
|---|---|---|---|
|   | Rectangular | $by$ | $b+2y$ |
|   | Triangular | $y^2/\tan\Theta$ | $2y/\sin\Theta$ |
|   | Trapezoidal | $by+y^2/\tan\Theta$ | $b+2y/\sin\Theta$ |
|   | Circular | $\frac{D^2}{4}\arccos(1-2\frac{y}{d}) -(D-2y)\sqrt{yD-y^2}$ | $D \arccos(1-2y/D)$ |
|   | Parabolic | $\frac{2}{3}by$ | $\frac{b}{2}[\sqrt{1+16y^2/b^2} + (b/4y)\ln(4y/b+\sqrt{1+16y^2/b^2})]$ $(\simeq b+8y^2/3b$ for $y \leq b/4)$ |

Fig. 12.3 Geometric properties of channel sections

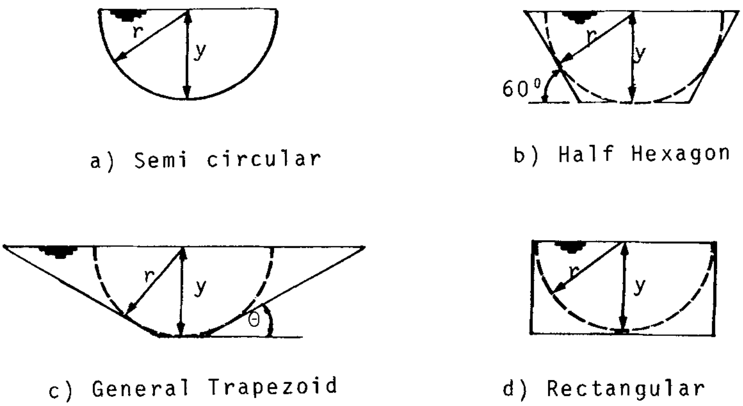



Fig. 12.4 Channel shape for minimum hydraulic radius

firm sandy clay, and for loose fine sand or peaty soils the side slope may be as flat as 1/3 (vertical/horizontal). Flattening of the side slope also facilitates construction and results in a more stable lining.

In built-up zones, the value of land may force the construction of deep rectangular channels. The sides may have to be supported with concrete retaining walls or gabions.

*Variable roughness*

Where a channel has a variable roughness across the section, the mean velocity may still be computed without actually subdividing the section. For example, if a channel has m sub-sections with wetted perimeters $P_j$ and Manning roughness $N_j$, then the equivalent average roughness may be calculated as follows:

$$N = \left[\frac{P_1N_1^{1.5} + P_2N_2^{1.5} + \ldots P_mN_m^{1.5}}{P}\right]^{2/3} \qquad (12.6)$$

The equation is based on the assumption that the same mean velocity exists for each section, and is derived by equating total cross-sectional area to the sum of areas of the individual elements assuming the velocity in each section is similar. The technique is thus only really applicable to a channel with one main section and no flood plains. It is not applicable to composite section channels.

In a similar manner, it is possible to obtain a weighted mean velocity distribution coefficient. Chow (1959) derives formulae for the average velocity distribution coefficients in terms of the coefficients and flow rates for each section. The analysis does not offer a solution for calculating the total discharge or water level.

*Composite section*

The velocity distribution across a complex section such as in Fig. 12.5 is far from uniform. The velocity is dependent on the ratio of hydraulic radius to roughness, and is consequently much lower in shallow sections such as over flood plains. The highest velocities and core of the flow will occur in the deepest section.

As an approximation, the hydraulic radius and cross-sectional area could be calculated for the section as a whole provided the side sections were small compared with the main section. It is more accurate to treat the section as comprising a number of separate sections with a common water level.

In the case of a simple cross section, the flow rate is related to the cross section as follows

$$Q = A^{5/3} S^{\frac{1}{2}}/NP^{2/3} \quad \text{(S.I. units)} \qquad (12.7)$$

where S is the slope and N is the Manning roughness coefficient. The cross sectional area is A and wetted perimeter is P.

For more complex shapes the area should be subdivided as shown in Fig. 12.5. The cross-sectional area $A_j$ and wetted perimeter $P_j$ of each section should be computed in terms of the depth $y_j$. If the water surface level is known then the flow velocity and discharge may be computed for each section using Equ. 12.7. The total discharge is then the sum of the individual discharges.

Where the discharge is known and the water surface elevation is to be computed, then the solution is more difficult. An equation expressing depth in terms of discharge for each section is established. The cross-sectional area $A_j$ and wetted perimeter $P_j$ are expressed in terms of water depth $y_j$ for each section. Since the water surface level is the same for each section, all depths may be expressed in terms of that in the deepest section. This depth is preferred as flow is most sensitive to this value. Thus for each section

$$Q_j = A_j^{5/3} S^{1/2}/N_j P_j^{2/3} \qquad (12.8)$$

$$= f_j(y_j) S^{1/2}/N_j$$

where the function $f_j(y_j) = A_j^{5/3}/P^{2/3}$ depends on the geometry and depth. Thus for a trapezoidal section it is $(by + y^2/\tan\theta)^{5/3}/(b + 2y/\sin\theta)^{2/3}$ Now $Q = \Sigma Q_j$ and $f_j(y_j) = f(y_m)$ where Q is the total flow and $y_m$ the

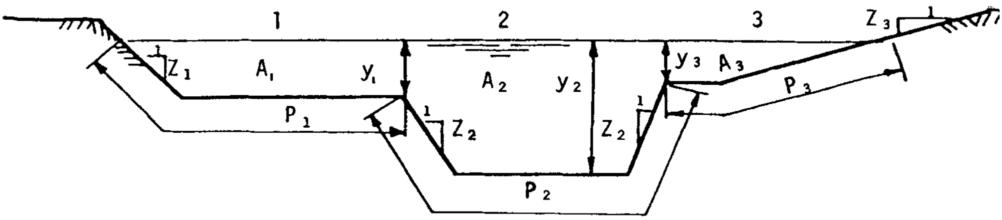



Fig. 12.5 Composite cross section channel

-depth at the deepest section.

Thus $Q = S^{1/2} \sum_j f(y_m) N_j$ (12.9)

This equation will be a lengthy algebraic expression which may be solved by trial and error for $y_m$ if Q is known, or graphically by plotting Q versus $y_m$ or by successive approximation.

It is unlikely that individual sections will have the same velocity and consequently the same energy line. The total head for the section as a whole may be taken to be $\alpha v_m^2/2g$ above the water surface where $v_m$ is the mean velocity Q/A and the energy coefficient $\alpha$ is defined by the equation

$$\alpha = \frac{\Sigma v_j^3 A_j}{v_m^3 \Sigma A_j} \qquad (12.10)$$

## NON-UNIFORM FLOW COMPUTATION IN IRREGULAR CHANNELS

Calculation of normal depth in a uniformly flowing channel can be performed directly employing a resistance equation such as that of Manning. Where compound sections are involved, the computational procedure may be complicated but a unique solution is still achievable.

In the case of artificial channels, such as may be constructed for storm water collection and discharge, the channel may possess a constant cross-sectional shape and grade. In such cases the assumption of uniform flow is applicable and flow depth determination is simple.

Where the discharge rate, cross-sectional shape and bed gradient vary along the channel, the assumption of uniform flow at any section will lead to errors. Conditions at one section may influence those at an upstream section. For example if there is an obstruction or reservoir, it will back up water for a considerable distance upstream. A free fall or chute may draw down the water surface and this effect would also manifest itself upstream. This is assuming subcritical flow. (Froude number F less than unity, or water depth greater than critical depth). In the case of supercritical flow (F greater than 1) upstream conditions would control.

The engineer often needs to know the water depths along a channel in order to decide on a channel depth with suitable freeboard, if it is an artificial channel, or for flood plain and catchment management in the case of natural channels. Where overtopping of the banks or even high water levels in the river are likely to occur frequently, the engineer faces a choice of one or both of two solutions. The channel may be improved by widening, deepening, steepening or smoothing The resulting water levels will subside. Alternatively with appropriate catchment management such as the provision of detention or retention storage, soakaways, indirect runoff routing and avoidance of smooth planes, the discharge can be controlled.

If all else fails the authorities may have to restrict building or other development within the flood plain.

In any case, the engineer has to determine flood limits for various conditions, in order for planning to proceed. The degree of sophistication with which the engineer must compute flood levels will depend on the complexity of the channel.

A variety of methods is available and described in many text books on open channel flow (e.g. Henderson, 1966, Chow, 1959). The following list describes the limits of applicability of techniques of increasing sophistication. The problem is assumed to be the determination of water surface elevation, all other variables such as channel geometry and discharge rate being known. Other situations may arise where flow depth or channel properties have to be determined, in which case a trial and error approach may be employed.

In practical problems involving river channels, cross sections are surveyed at pre-defined positions. The computations should thus proceed from one section to the next, proceeding upstream in the case of subcritical flow in order to determine water levels at each section. The method of computation most favoured in practice is the standard step method (e.g. Henderson, 1966). It is a numerical method, suited to digital computer programming, (e.g. Weiss and Midgley, 1978).

The method is based on the calculation of energy level at successive cross sections by two means. The water surface elevation is initially estimated and refined after comparing the two energy levels so resulting. One energy head is simply the assumed water level plus velocity head which we will refer to as $H_e$. The other is derived by adding to the energy level at the downstream section, the friction head loss

TABLE 12.1 Summary of methods of computation of water levels in open channel

| METHOD | EQUATIONS | LIMITATIONS |
| --- | --- | --- |
| 1. Direct solution of resistance equation for flow depth | $Q = \frac{S^{1/2}}{N} \frac{A^{5/3}}{P^{2/3}}$<br>$A,P = f(y)$ | Uniform flow in regular channel |
| 2. Direct step method. Computation of distance at which flow depth reaches a preselected figure. | $\Delta x = \frac{\Delta E}{S_o - S_f}$<br>$S_f = (NQ)^2 P^{4/3} A^{10/3}$ | Uniform channels, i.e. constant cross section and slope. |
| | $E = y + Q^2/2gA^2$<br>$= \Delta y(1-F^2)$<br>$F^2 = Q^2 B/gA^3$ | Require interpolation for fixed cross sections |
| 3. Direct integration | $\frac{dy}{dx} = S_o \frac{1-(y_o/y)^n}{1-(y_c/y)^m}$ | Uniform channels |
| Bress (m=n=3) : | $S_o x = y - y_o[1-(y_c/y_o)^3]\phi$ | Require tabulations of Bresse's function $\phi$ |
| Bakhmeteff : | $S_o \frac{dx}{dy} = 1-(1-\beta)/[1-(y/y_o)^n]$<br>$\beta = F^2 S_o/S_f$ | |
| Chow : | $x = \frac{y_o}{S_o}\left[u - F(u,n) + \left(\frac{y_c}{y_o}\right)^m \frac{J}{n} F(v,J)\right] + A_1$ | Require Tabulations |

TABLE 12.1 continued

| METHOD | EQUATIONS | LIMITATIONS |
|---|---|---|
| 4. Graphical Methods: Depth from distance | Plot E, U against y, where $U = E-\frac{1}{2}S_f\Delta x$ | |
| Ezra | $h_2 + F(h_2)=h_1+F(h_1)$<br>$F(h)=\frac{\alpha v^2}{2g} + S_f\frac{\Delta x}{2}$ | |
| Grimm | $Q = KS_w^{1/2}$ | Requires only stage-discharge relationships, but inaccurate. |
| Escoffier | $\Delta h = \frac{S_{f_1} + S_{f_2}}{2}\Delta x$ | |
| 5. Standard Step Methods | $H_{2e} = y_2+z_2 + \frac{v_2^2}{2g}$ | Trial and error method speeded by |
| Single channels | $H_{2f} = H_1+(S_{f1}+S_{f2})\Delta x/2$ | $\Delta y_2 = \dfrac{H_{2f} - H_{2e}}{1-\alpha F_2^2 + \dfrac{3S_{f2}\Delta x}{2R_2}}$ |
| Compound sections | $H_2 = y_2 + z_2 + \frac{\alpha v_2^2}{2g}$<br>$\alpha = \frac{(\Sigma A_i)^2}{(\Sigma K_i)^3}\Sigma\frac{(K_i^3)}{(A_i^2)}$, $K = \frac{AR^{2/3}}{N}$ | At bridges<br>$\frac{\Delta y}{y_3} = KF_3^2(K+5F_3^2-.6)(r+15r^4)$<br>K = 0.9 to 1.25, r = pier width/span ratio |

determined from the mean friction gradient at the two sections, and any eddy losses which may occur. We thus obtain $H_f$. Successive estimates of water surface level, or stage, may be determined from the following approximation: We require to eliminate the difference between $H_e$ and $H_f$ where

$$H_{2e} = z_2 + y_2 + \frac{\alpha v_2{}^2}{2g} \qquad (12.11)$$

and $$H_{2f} = H_1 + \Delta x \ (S_{f1} + S_{f2})/2 \qquad (12.12)$$

where subscript 1 refers to the previous (downstream) section and 2 to the present section. $\alpha$ is the velocity coefficient

$$\alpha = \Sigma(v_i{}^3 A_i)/v_m{}^3 \Sigma A_i \qquad (12.13)$$

which may be nearly unity for simple sections.

Thus if $\Delta H = H_e - H_f$ (12.14)

then $$\frac{dH}{dy} = \frac{d}{dy}\left(y_2 + \frac{\alpha v_2{}^2}{2g} - \Delta x \ S_{f2} \ /2\right) \qquad (12.15)$$

$$= 1 - \frac{Q^2 B}{gA^3} - \frac{\Delta x}{2} \frac{dS_f}{dy_2} \qquad (12.16)$$

$$\doteqdot 1 - \alpha F_2{}^2 + \frac{3 \ S_{f2} \ \Delta x}{2 \ R_2} \qquad (12.17)$$

$$\therefore \Delta y_2 = \frac{H_{2f} - H_{2e}}{1 - \alpha F_2{}^2 + \dfrac{3S_{f2} \ \Delta x}{2R_2}} \qquad (12.18)$$

Thus the second approximation to y is obtained by adding $\Delta y_2$ to the original estimate. It is seldom necessary to make more than one such correction unless the water surface slopes steeply.

Where there are eddy losses in addition to friction losses, e.g. due to a bend or expansion in section, there will be an additional head loss $K_t \dfrac{\alpha v^2}{2g}$ (12.19)

This should be added to the friction loss in computing $H_f$. Similarly losses at bridges and culverts should be added as explained later. Thus, the loss due to bridge pier contractions may be estimated from Yarnell's equation

$$\frac{\Delta y}{y_3} = K_c F_3{}^2 (K_c + 5F_3{}^2 \ -0.6) \ (r + 15r^4) \qquad (12.20)$$

where $y_3$ is the downstream depth, $\Delta y$ is the increase in depth through the bridge, F is the Froude number, $F^2 = Q^2 B/gA^3$, and r is the total pier width to span ratio. $K_c$ is a contraction coefficient which varies from 0.9 for rounded pier fronts to 1.25 for square fronted piers.

With compound sections the assumption of a uniform velocity across the section may not be satisfactory. In such cases the cross section must be divided and treated as follows.

Define $K = \frac{A}{N} R^{2/3}$ (S.I. units) (12.21)

Then $Q = K S_f^{1/2}$ (12.22)

Equating friction slope $S_f$ for each subsection of the river,

$$\frac{v_1 A_1}{K_1} = \frac{Q_1}{K_1} = \frac{Q_2}{K_2} = \ldots\ldots = \frac{\Sigma Q_i}{\Sigma K_i} = \frac{v_m \Sigma A_i}{\Sigma K_i} \qquad (12.23)$$

where subscript m refers to the mean and i to the subsections.

$$\text{Now } \alpha = \frac{\Sigma(v_i^3 A_i)}{v_m^3 \Sigma A_i} \qquad (12.24)$$

$$= \Sigma\left(\frac{K_i^3}{A_i^2}\right) \frac{(\Sigma A_i)^2}{(\Sigma K_i)^3} \qquad (12.25)$$

An example (Table 12.2) demonstrates the application of the techniques to determine the water surface profile along the channel configuration illustrated in Fig. 12.6. The columns are almost self explanatory. The computations for each successive section start with an initial assumption for water surface level (except for the first, downstream, section which must have water surface level defined). If the energy levels $H_e$ and $H_f$ do not correspond after performing the computations for any column, the assumed water surface level is revised by the amount in the last column, and the computations are repeated. If there is reasonable agreement, the mean H is inserted in the last column and this is the value to use for subsequent sections.

## UNSTEADY FLOW

If discharge rate or flow depth vary at any point in time, the flow is said to be unsteady. In many cases of slowly changing flow the steady state equations of motion are still applicable. The steady-state friction equation and discharge-depth relationships are assumed to apply. Thus accelerations in time and space are neglected. Many flood routing techniques are based on this premise. In particular the kinematic equations were so derived. The only allowance for the dynamic condition was in the continuity equation. It is rarely the drainage engineer needs to adopt more sophisticated methods of analysis.

Situations where the dynamic forces are important are in the case of

TABLE 12.2 Example - backwater analysis of river by standard step method. See Fig. 12.6

| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| River chain. m | River stage m | River area A $m^2$ | P m | R m | $K = \frac{AR^{2/3}}{N}$ | $K^3/A^2$ $\times 10^6$ | $\alpha = \frac{(\Sigma A)^2}{(\Sigma K)^3}\Sigma\left(\frac{K^3}{A^2}\right)$ | $v_m$ m/s | $\frac{\alpha v_m^2}{2g}$ | $H_e$ m | $S_f = \left(\frac{Q}{\Sigma K}\right)^2$ $\times 10^{-3}$ | $S_f$ mean $\times 10^{-3}$ | $h_f = S_f \Delta x$ | $H_f$ | $\pm\Delta y$ or $H_m$ |
| 0 | 8.50 | 264 | 49 | 5.39 | 27060 | 284 | 1.00 | 2.765 | 0.390 | 8.890 | 0.728 | | | | |
| 180 | ~~9.10~~ | 196 | 33 | 5.94 | 21430 | 256 | | | | | | | | | |
| | | 38 | 20 | 1.90 | 1940 | 5 | | | | | | | | | |
| | | 234 | 53 | 7.84 | 23370 | 261 | 1.12 | 3.12 | 0.556 | 9.656 | 0.976 | 0.852 | 0.153 | 9.043 | -0.7 |
| | 8.40 | 178 | 31 | 5.74 | 19030 | 217 | | | | | | | | | |
| | | 26 | 17 | 1.53 | 1150 | 2 | | | | | | | | | |
| | | 204 | 48 | 7.27 | 20180 | 219 | 1.11 | 3.58 | 0.725 | 9.125 | 1.31 | 1.018 | 0.183 | 9.073 | 9.10 |
| 340 | 8.4 | 77 | 19 | 4.05 | 6530 | 47 | | | | | | | | | |
| | | 31 | 21 | 1.48 | 1340 | 3 | | | | | | | | | |
| | | 108 | 40 | 5.53 | 7870 | 50 | 1.20 | 6.76 | 2.800 | 11.20 | 8.60 | 4.96 | 0.793 | 11.29 | 11.25 |
| 570 | ~~10.7~~ | 345 | 53 | 6.51 | 40117 | 542 | 1.00 | 2.12 | 0.228 | 10.93 | 0.33 | 4.47 | 1.028 | 12.27 | +1.13 |
| | ~~11.8~~ | 437 | 60 | 7.28 | 54610 | 853 | 1.00 | 1.67 | 0.142 | 11.94 | 0.18 | 4.39 | 1.010 | 12.26 | 12.10 |
| | 11.95 | | | | | | | | | | | | | | |
| | Through bridge $\Delta y = 10.9 \times 1.0 \times .057(1.0+5 \times .057-0.6)(.34+15 \times .34^4) = 0.23$ | | | | | | | | | | $:H_e = 11.95+.23+.14=12.32$ | | | | |
| 760 | 12.1 | 402 | 60 | 6.71 | 47690 | 671 | 1.00 | 1.82 | 0.168 | 12.17 | 0.23 | 0.20 | 0.038 | 12.36 | 12.26 |

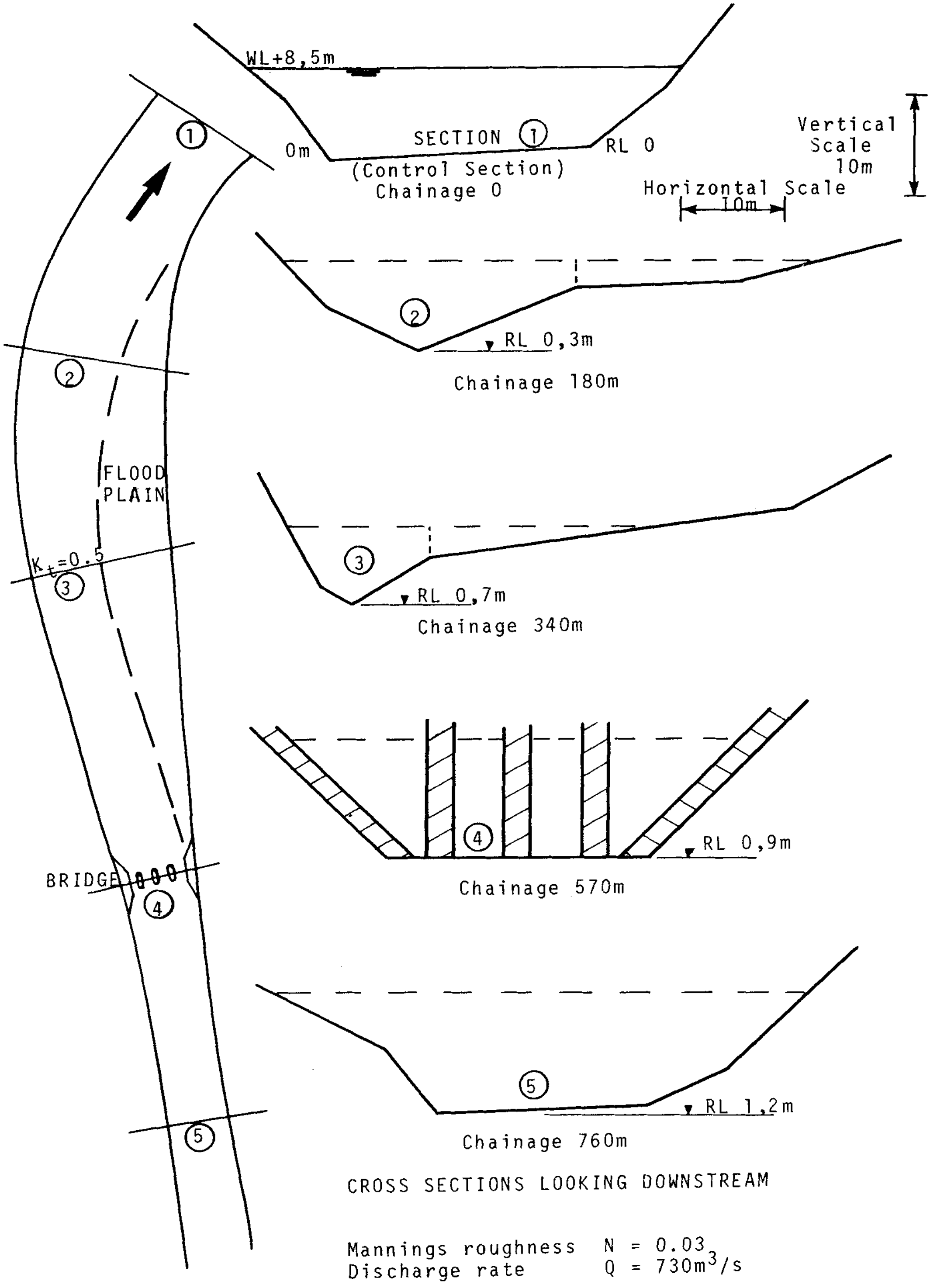



Fig. 12.6 River sections for backwater computation example

surges and waves. A surge can be caused by a rapid variation in discharge rate, such as by closing a sluice gate in a canal. A positive surge will travel upstream at a speed of approximately $c = \sqrt{gy} - v$ (12.26) where y is the water depth, g is gravitational acceleration and v is the initial water velocity. Methods of analysing systems subject to changes in flow are presented by Pickford (1969) and others.

Waves are more complicated to predict and analyse than surges. This is because water particle motion in the vertical direction can no longer be neglected. Waves may be caused by the same disturbance that creates a surge. The rapid vertical acceleration at the surge front may cause the water surface to oscillate.

More common, or of greater concern, are waves created by wind action on the surface of the water. Wave height is dependent on the wind speed, the fetch or distance over which the waves build up, and wind speed (e.g. Hasselman, 1976). Freeboards varying from 0.1 to 0.5 metres are often allowed on canals for wave action.

Cross-waves or diagonal standing waves may be caused by sharp bends in the canal, especially if the flow is supercritical. Under supercritical flow conditions transitions should be very carefully designed or even modelled to account for the high velocity head and centrifugal forces.

## CHANNEL STABILIZATION

In nature channels and rivers erode or deposit until they reach stable regimes. Bed slope, meanders, cross sectional area, shape and bed form adjust until they are consistent with the discharge, specific energy and sediment load of the stream. The natural shape of channels is altered by constructing bridges, banks and other works.

Urbanization will affect the regime. Flooding will become more severe unless on-site storage is provided. Streams will be confined to improved channels due to the increased cost of flooding. These actions will aggravate the erosion problem.

Replacement of natural streams by lined channels can make the flood problem more severe downstream. The increased hydraulic radius and the smoother perimeter result in higher velocities and faster concentration times. The construction of impermeable concrete or asphalt linings could thus magnify the flow problem.

Rockfill provides an economical alternative lining and can be adequately designed to prevent scour of underlying soils. The roughness of a natural channel may be maintained so that concentration times are not affected.

The cost of loose rockfill is considerably less than that of concrete or even asphalt. Although a thicker layer may be required for rockfill, especially where graded filters are required, there is a number of added advantages in the use of rockfill:

(i) Rockfill is more economical per unit volume of lining than most other linings.

(ii) It is easier to repair if the lining is damaged.

(iii) It is flexible and consequently does not undergo as much damage as concrete or asphalt due to ground settlement or excessive loads.

(iv) It is permeable, so that there is no danger of soil backpressure causing spalling of the lining.

(v) Rockfill is usually readily available, as it is a natural material, and in fact in many cases it is a waste product from excavations.

(vi) It is aesthetically pleasing, especially gabion walls and reno mattresses. Rockfill blends in with the environment and in fact mellows with age.

(vii) It is environmentally acceptable since it does not prohibit growth of natural vegetation or exclude fish life.

(viii) The weight and roughness of rockfill help maintain the stability of banks and the lining itself.

(x) Well-selected rock will be durable and weather-resistant.

The disadvantages of rockfill can usually be compensated for. It provides a rougher surface than concrete for instance, for the passage of water. Thus channel cross-sectional areas are larger for any discharge rate. In many cases, however, there is excess energy which has to be dissipated and channel friction losses reduce the cost of stilling basins or other artificial energy dissipation devices. Where the channel drops steeply, drops or steps may be constructed of gabions, (Essey and Horner, 1978; Stephenson, 1979).

The rock size and grading should be selected to satisfy the design criteria. Factors which need to be borne in mind are:

(i) Friction factor or channel roughness

(ii) Resistance of the channel lining to erosion - on the bed and on the banks

(iii) Prevention of erosion of the underlying bed material by the use of suitably graded filters.

There is a relationship between the channel roughness and erodibility of the lining. Both criteria depend on rock size. In fact the larger the stone size, the greater the friction factor and the lower the water velocity. The cross sectional area for any specified discharge thus depends on the lining characteristics and is larger the rougher the lining. Large stones are less liable to erosion though. In fact simultaneous solution of the two criteria is feasible and produces an optimum stone size.

## FRICTION LOSSES

The most popular equation for determining the energy gradient due to friction in channels is the Manning equation. This equation applies to the uniform flow of water at normal temperatures, and the relationship is dependent on the roughness of the lining. Manning's equation is

$$v = \frac{K}{N}\left(\frac{A}{P}\right)^{2/3} S^{1/2} \tag{12.27}$$

where K = 1.0 in S.I. units and 1.486 in f.p.s. units. (12.28)
v is the mean flow velocity, N is the Manning roughness coefficient, A is the cross sectional area, P is the wetted perimeter and S is the energy gradient.

Manning's equation has two drawbacks: the form of the equation is dependent on the units used, and empirical data is needed to evaluate the roughness, N. The advantage of the equation over the alternative equations of Darcy-Weisbach and Chezy, is that the roughness N is a unique value for any channel and is not dependent on the flow depth.

Strickler took Manning's equation a step towards rationalization by expressing the friction factor in terms of the boundary roughness, which is analogous to the Nikuradse roughness, an absolute measure of protrusion of particles such as stones on the perimeter of the conduit. The Strickler equation is

$$v = 7.7\ (R/k)^{1/6}\ (RSg)^{1/2} \tag{12.29}$$

where R is the hydraulic radius, A/P, and k is a measure of roughness. k may be approximated by median stone size d in channels with loose rocks lying on the bed. In the case of hand-packed rockfill linings, the roughness is less and it may be approximated by nd where n in the porosity (ratio of voids to total volume) of the rockfill. Fortunately the resulting discharge is not particularly sensitive to the value of k, since k is to the power of 1/6. Acceptable values of roughness k in millimetres and the corresponding Manning roughness N are tabulated in Table 12.3 for straight channels with steady uniform flow.

In the case of stones which vary in size the weighted mean stone size applies.

Near the bed of the channel the velocity will be lower than the mean cross sectional velocity. In the interstices between rocks the energy

TABLE 12.3 : Roughness of channel linings

| Surface | Manning's N | Roughness, k (mm) |
|---|---|---|
| Reno mattress, grouted | 0.016 | 4 |
| Reno mattress with selected stone hand packed | 0.022 | 40 |
| Reno mattress with quarry stone mechanically placed | 0.025 | 90 |
| Gabions, mechanically filled with quarry stone | 0.027 | 125 |
| Boulder lined | 0.03 | 180 |

loss gradient will be as in the channel and the corresponding velocity may be calculated from the equation

$$S = \frac{K_f v_v^2}{gd} \tag{12.30}$$

where $v_v$ is the velocity in the rock voids. $K_f$ is a friction factor which is given by the equation

$$K_f = \frac{800\nu}{v_v d} + K_t \tag{12.31}$$

where $\nu$ is the kinematic viscosity of the water (about $1.1\text{x}10^{-6}m^2/s$) and $K_t$ is the turbulent friction factor which depends on stone shape and roughness. $K_t$ varies from 1 for smooth marbles to 4 for sharp angular stone with a mean value of 3 for crushed stone.

A relationship between flow velocity in the channel and in the interstices of the rock may be established for the turbulent flow case. Eliminating S and putting k = d and R = y for a wide channel with one stone layer,

$$v_v/v \doteqdot 0.075\ (d/y)^{2/3} \tag{12.32}$$

Thus if one knows the scour velocity of the underlying bed material (e.g. Table 12.4), the rockfill size d on the bed may be selected to ensure $v_v$ is maintained below the scour figure. Unfortunately the flow depth y is affected by the overlying rock size.

## STABILITY OF ROCKFILL LININGS

The lining must be stable under the action of flowing water. The erosive velocity of particles on the perimeter of a channel depends on the bed slope, bank slope, and stone characteristics. An isolated stone will wash away at a lower velocity than rockfill packed into a layer

TABLE 12.4 Scour Velocities according to USBR

| Material | Particle Size (mm) | Scour Velocity (m/s) | ft/s |
|---|---|---|---|
| Fine silt | 0.01 | 0.17 | 0.5 |
| Fine sand | 0.1 | 0.24 | 0.75 |
| Medium sand | 1.0 | 0.55 | 1.5 |
| Gravel | 10 | 1.0 | 3 |
| Pebbles | 100 | 3.0 | 10 |

with an even finish. Ravelling of rockfill packed in Reno mattresses will occur at even higher velocities owing to the wire mesh holding down stones against uplift and thus increasing friction. An expression for the stable size of stones on a bed or bank exposed to flowing water may be derived by equating overturning to stabilizing forces (see fig. 12.7).

When the flow is horizontal, parallel to the bank i.e. such as along the banks of a trapezoidal channel, the minimum stable rock size is approximately

$$d = \frac{0.25\ v^2}{g(G-1)\ \cos\theta(\tan^2\phi - \tan^2\theta)^{1/2}} \qquad (12.33)$$

$\phi$ is the angle of friction of the rockfill and G is the relative density or specific gravity. $\theta$ is the slope angle from the horizontal.

The values of the constants were evaluated experimentally (Izbash and Khaldre, 1970).

## REFERENCES

Abbott, M.B., 1979. Computational Hydraulics, Pitman, London 324pp

Chow, V.T., 1959. Open Channel Hydraulics, McGraw Hill, N.Y. 680pp

Connor, J.J. and Brebbia, C.A., 1976. Finite Element Techniques for Fluid Flow, Newnes-Butterworths, London 310pp

Essery, I.T.S. and Horner, M.W., 1978. The Hydraulic Design of Stepped Spillways. Report 33, CIRIA, London, 45pp.

Hasselman, K., 1976. A parametric wave prediction model, J. Phys. Oceanography, 6(2).

Henderson, F.M., 1966. Open Channel Flow. Macmillan, N.Y. 522pp.

Izbash, S.V. and Khaldre, Kh.Yu. 1970. Hydraulics of River Channel Closure. Transl. Cairns, G.L., Butterworths, London, 174pp.

Pickford, J., 1969. Analysis of Surge, Macmillan, London, 203pp

Stephenson, D., 1979. Rockfill in Hydraulic Engineering, Elsevier, Amsterdam, 215pp.

Stephenson, D., 1980. Rockfill linings for channels. Proc. Symposium on Watershed Management. ASCE. Div. of Irrig. and Drainage, Boise.

Vanoni, V.A. (Editor), 1975. Sedimentation Engineering, ASCE, N.Y., 745pp.
Weiss, H.W., and Midgley, D.C., 1978. Suite of mathematical flood plain models. Proc. ASCE, 104, HY3, 13604, p361-376.

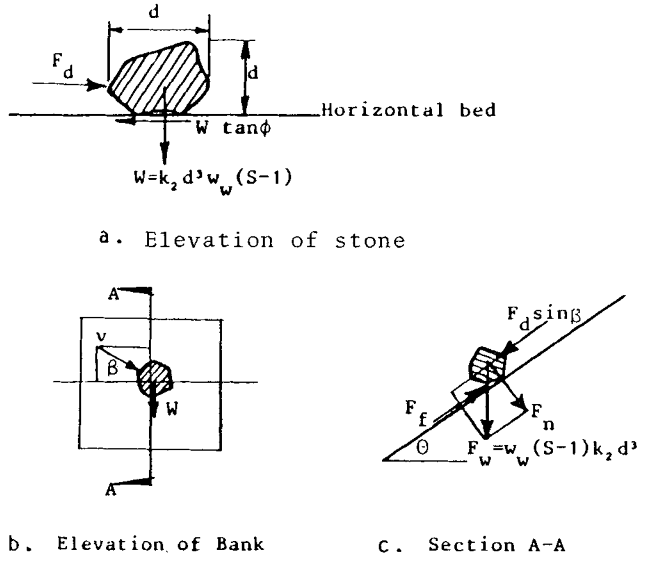



Fig. 12.7 Stability of stone in flowing water