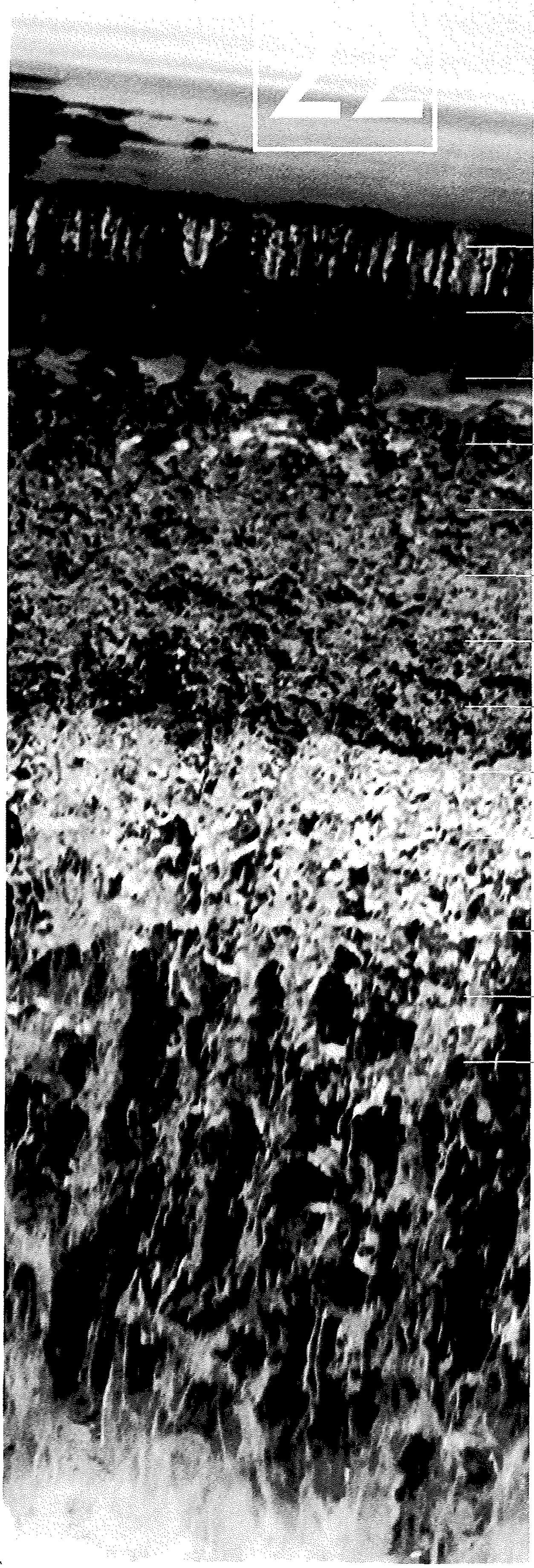


John Walter

# Hydraulics

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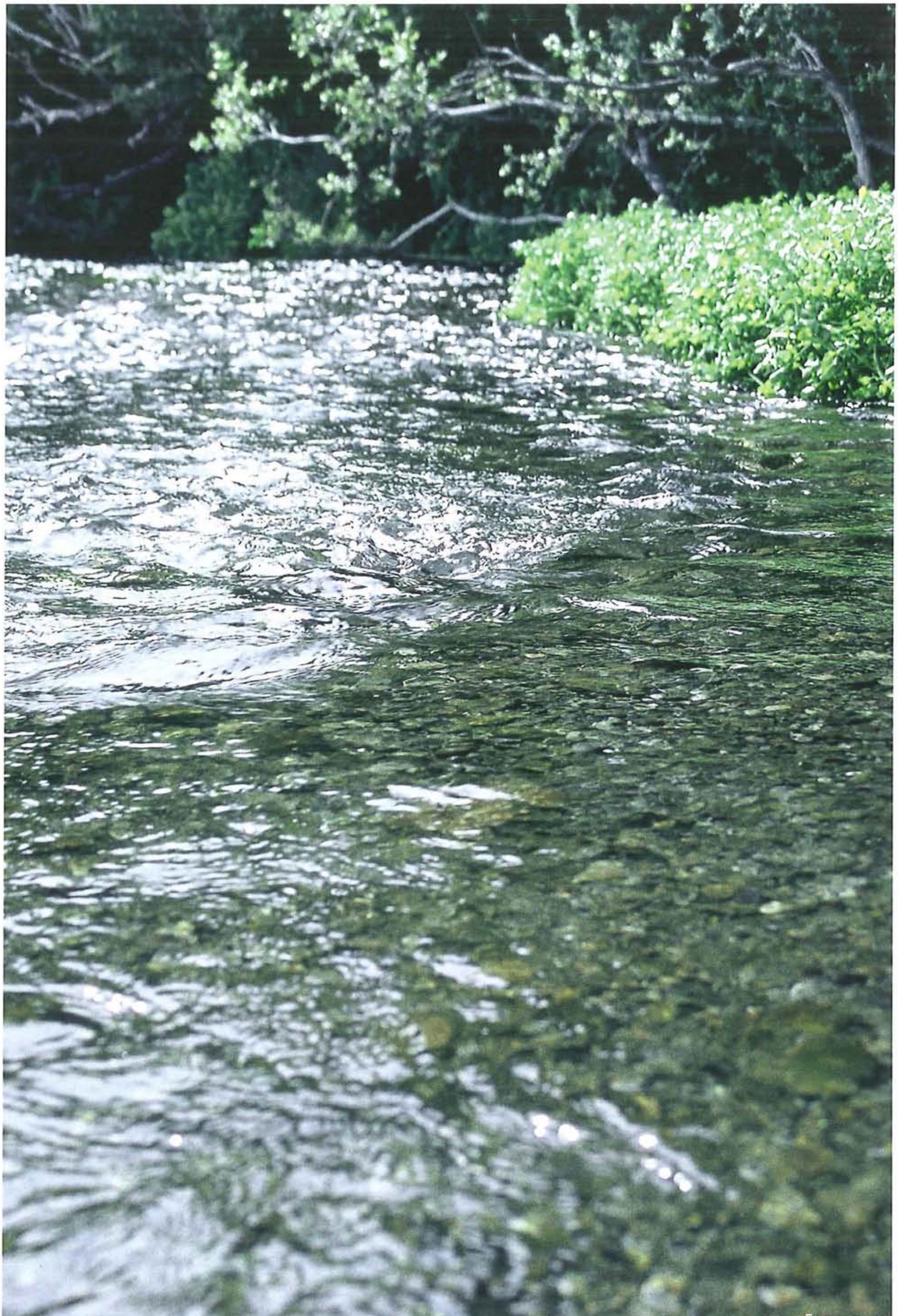

Flow transition from pool to riffle (Ōtūkaikino)

## 22.1 Introduction

Designers are referred to a standard hydraulics textbook for procedures by which flow depth, velocity, and energy state can be analysed. Useful books are Pilgrim (1987), Henderson (1966), French (1985), Chow (1959), Brater and King (1976).

Some specific aspects of hydraulic design such as Manning's roughness coefficient, outfall water levels, scour velocities, and the hydraulics of structures such as bridges, sumps, and weirs are described here. Refer to a hydraulic textbook for further information.

## 22.2 Uniform Flow Resistance Formulae

Over the years, various expressions have been developed to predict velocity and depth of steady uniform flow in a conduit under given conditions. The most widely used formulae are the Darcy-Weisbach Formula and the Manning Formula. Both of these formulas involve empirically determined roughness parameters.

### 22.2.1 The Darcy-Weisbach Formula

The Darcy-Weisbach Formula is commonly applied in the calculation of head losses in closed conduits of circular cross-section based on friction factor, velocity, length, and diameter. The friction factor has been further related to surface roughness and Reynolds Number via the empirical Colebrook-White formula developed from pipe experiments. The Pipe Flow Nomograph (Appendix 11) is based on a similar empirical relationship.

### 22.2.2 The Manning Formula

The Manning formula is generally adopted in calculations associated with steady flow in channels of medium to large size. The Manning formula states that in steady uniform flow:

$$V = R^{2/3}S^{1/2}/n \qquad \text{Eqn (22-1)}$$

$$Q = AR^{2/3}S^{1/2}/n \qquad \text{Eqn (22-2)}$$

where V = mean velocity of flow (m/s)

Q = discharge ($m^3/s$)

R = hydraulic radius

= the ratio A/P; where A is the cross sectional area and P is the wetted perimeter (i.e. the length of the line of contact on the cross-section between the water and the channel boundary)

S = the slope of the energy line (also known as the friction slope, $S_f$). Under uniform flow conditions this is equal to the channel slope

n = Manning's roughness coefficient which depends only on the roughness characteristics of the boundary surface

### 22.2.3 Factors Affecting Manning's Roughness Coefficient

The channel characteristics and other factors that affect the value of the roughness parameter are:

#### Surface Roughness

This characteristic (also termed grain roughness or texture roughness) is associated with the surface texture of the channel boundaries. The value of 'n' associated with the surface roughness may be estimated by means of the empirical relation known as Strickler's Formula (see Eqn 22-14).

#### Form Roughness

The presence of dunes and ripples on the bed of an alluvial channel and the existence of medium-scale irregularities (typically having dimensions of the order of a metre) in the bed and banks of a channel will increase the resistance to flow and hence will increase the effective value of 'n'. The effect of such features is referred to as form roughness.

In lined artificial channels, form roughness will not usually be significant. In natural channels, on the other hand, form roughness may contribute a significant component of the total resistance.

#### Size and Shape of Channel

It is generally assumed that the value of 'n' is dependent only upon the "absolute" roughness characteristics of the channel boundaries, rather than upon the "relative" roughness, and accordingly that 'n' is independent of the size and shape of the channel cross-section.

#### Channel Bends, Irregularities, and Obstructions

Features such as bends and changes in cross-section shape or size, and obstructions such as bridge piers, will increase the resistance to flow and hence will increase the effective value of 'n'.

The effect of channel bends is usually relatively minor, unless the bends are numerous or very sharp. Bend head losses can be determined from the procedure given in *Section 22.4.4: Head Loss Due to Change in Channel Alignment*.

Gradual variations in the channel cross-section have relatively little effect; abrupt variations, particularly abrupt expansions, may have a significant effect.

### Vegetation

The effect of vegetation on the flow resistance can be very significant, particularly in the case of small channels and overbank flow areas or flood plains. The effect of vegetation can be complex and variable - for example, small shrubs and grasses, which at low discharges offer significant resistance to the flow, may be bent over or flattened by higher flows, with a consequent decrease in the resistance offered by the vegetation. Seasonal variations and long-term change in vegetation may require consideration, and it may be necessary to take into account the extent and frequency of maintenance operations.

In general, the effects of vegetation on flow resistance will tend to be strongly dependent upon the water depth or stage.

The effects of vegetation on flow resistance are discussed by Chow (1959) and Henderson (1966), and illustrated in the table of Manning's 'n' values listed in Table 22-1 (opposite page).

### Stage

In a given unlined channel, the value of 'n' is likely to be relatively high at low stages, when much of the flow boundary will consist of the relatively coarse material characteristically found in the channel bed. At very high stages, the resistance to flow is likely to be increased as a result of the flow coming in contact with vegetation and other obstructions, and the value of 'n' is again likely to be relatively high.

In a given channel reach, a minimum value of 'n' will usually occur at a stage below or approaching bankfull condition. The relation between the stage and the value of 'n' in a channel reach will depend upon the way in which the boundary roughness varies from point to point on the channel perimeter.

In a lined channel, the value of 'n' is less likely to be dependent upon the stage, as the roughness characteristics of the channel boundaries are more likely to be uniform.

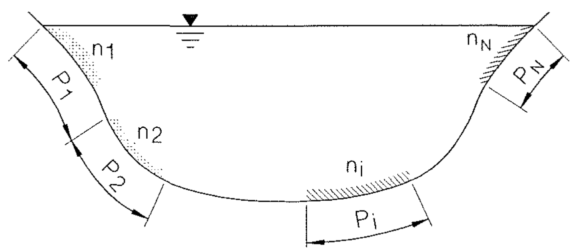


*Figure 22-1: Channel of composite roughness.*

## 22.2.4 Estimation of Manning's Roughness Coefficient

### Channel of Single Roughness

The estimation of the value of 'n' for a given flow in a given channel reach involves a level of engineering judgement. The factors that affect the value of the roughness parameter, and which should thus be borne in mind when estimating the value of 'n' for a given reach, have been outlined in *Section 22.2.3: Factors Affecting Manning's Roughness Coefficient*. The resources and procedures that are available to assist the investigator include:

Tables:

- Table 22-1 provides a range of Manning's 'n' values. Further values can be found in tables presented by Chow (1959), Henderson (1966) and French (1985). Caution is advised, as in most cases the extent to which the reliability of the tabulated values has been established is not explicitly stated.

Photographs:

- Photographs of channel reaches of known 'n' values are potentially of assistance in estimating the value of 'n' for a different reach having recognisably similar characteristics.
- Chow (1959) has presented 24 black and white photographs of channel reaches with 'n' values ranging from 0.012–0.150. Barnes (1967) has presented approximately 100 colour photos from 50 channels with 'n' values ranging from 0.024–0.075. Hicks and Mason (1991) present a comprehensive set of photos and values.

### Channel of Composite Roughness

In many cases, the boundary roughness varies from point to point at a given channel cross-section. For example, bed roughness may be recognisably different from the roughness of the channel banks.

With reference to Figure 22-1, the total wetted perimeter (P) is imagined to be divided into N segments, identified as $P_1$, $P_2$ ... $P_N$. A particular value of the roughness parameter is associated with each segment, the respective values of the roughness parameter being $n_1$, $n_2$ ... $n_N$.

An estimate of 'n' can be obtained from the following expression:

$$n = \left[\frac{\Sigma\left(P_i n_i^{1.5}\right)}{P}\right]^{2/3} \qquad \text{Eqn (22-3)}$$

where n = the roughness parameter for the cross-section as a whole

*Table 22-1: Manning's Roughness Coefficients.*

## Manning's Roughness Coefficients

**I. Closed Conduits:**

A. Concrete pipe ... 0.011 - 0.013
B. Corrugated metal pipe or pipe arch:
  1. 68 mm by 13 mm corrugation ... 0.024
  2. 150 mm by 50 mm corrugation (field bolted) ... 0.030
C. Vitrified clay pipe ... 0.012 - 0.014
D. Cast iron pipe, uncoated ... 0.013
E. Steel pipe ... 0.009 - 0.011
F. Brick ... 0.014 - 0.017
G. Monolithic concrete:
  1. Wood forms, rough ... 0.015 - 0.017
  2. Wood forms, smooth ... 0.012 - 0.014
  3. Steel forms ... 0.012 - 0.013

**II. Open Channels, Lined (straight alignment):**

A. Concrete, with surfaces as indicated:
  1. Formed, no finish ... 0.013 - 0.017
  2. Float finish ... 0.013 - 0.015
  3. Float finish, some gravel on bottom ... 0.015 - 0.017
  4. Sprayed concrete, good section ... 0.016 - 0.019
  5. Sprayed concrete, wavy section ... 0.018 - 0.022
B. Concrete, bottom flat finished, sides as indicated:
  1. Random stone in mortar ... 0.017 - 0.020
  2. Dry rubble (riprap) ... 0.020 - 0.030
C. Gravel bottom, sides as indicated:
  1. Formed concrete ... 0.017 - 0.020
  2. Random stone in mortar ... 0.020 - 0.023
  3. Dry rubble (riprap) ... 0.023 - 0.033
D. Brick ... 0.014 - 0.017

**III. Open Channels, Excavated (straight alignment, natural lining):**

A. Earth, uniform section:
  1. Clean, after weathering ... 0.018 - 0.020
  2. With short grass, few weeds ... 0.022 - 0.027
  3. In gravelly soil, uniform section, clean ... 0.022 - 0.025
B. Earth, fairly uniform section:
  1. No vegetation ... 0.022 - 0.025
  2. Grass, some weeds ... 0.025 - 0.030
  3. Dense aquatic plants in deep channels ... 0.030 - 0.035
  4. Sides clean, gravel bottom ... 0.025 - 0.030
  5. Sides clean, cobble bottom ... 0.030 - 0.040
C. Dragline excavated or dredged:
  1. No vegetation ... 0.028 - 0.033
  2. Light shrubbery on banks ... 0.035 - 0.050
D. Rock:
  1. Smooth and uniform ... 0.035 - 0.040
  2. Jagged and irregular ... 0.040 - 0.045
E. Channels not maintained:
  1. Dense weeds, high as flow depth ... 0.080 - 0.120
  2. Clean bottom, shrubbery on sides ... 0.050 - 0.080
  3. Clean bottom, shrubbery on sides, highest stage of flow ... 0.070 - 0.110
  4. Dense shrubbery, high stage ... 0.100 - 0.140

Note: the value of 'n' for natural channels must be increased to allow for the additional energy loss caused by bends. The increase may be in the range of perhaps 3 to 15 %

**IV. Roadside Channels and Swales with Maintained Vegetation (for velocities of 0.6 m/s to 1.8 m/s):**

A. Depth of flow up to 210 mm:
  1. Good stand, any grass:
    a. Mowed to 50 mm ... 0.070 - 0.045
    b. Length 100 mm to 150 mm ... 0.090 - 0.050
  2. Fair stand, any grass:
    a. Length about 300 mm ... 0.140 - 0.080
    b. Length about 600 mm ... 0.250 - 0.120
B. Depth of flow 210 mm to 460 mm:
  1. Good stand, any grass:
    a. Mowed to 50 mm ... 0.120 - 0.070
    b. Length 100 mm to 150 mm ... 0.200 - 0.100
  2. Fair stand, any grass:
    a. Length about 300 mm ... 0.100 - 0.060
    b. Length about 600 mm ... 0.170 - 0.090

**V. Roadside Concrete Side-Channels:**

A. Concrete channel, trowelled finish ... 0.012
B. Asphalt pavement:
  1. Smooth texture ... 0.013
  2. Rough texture ... 0.016
C. Concrete side-channel with asphalt pavement:
  1. Smooth ... 0.012
  2. Rough ... 0.013
D. Concrete pavement:
  1. Float finish ... 0.014
  2. Broom finish ... 0.016
E. For side-channels with small slope, where sediment may accumulate, increase above values of n by ... 0.002

**VI. Natural Stream Channels:**

A. Streams:
  1. Fairly regular section:
    a. Some grass & weeds, little/no shrubs 0.030 - 0.035
    b. Dense growth of weeds, depth of flow greater than weed height ... 0.035 - 0.050
    c. Light shrubbery on banks ... 0.035 - 0.060
    d. Heavy shrubbery on banks ... 0.050 - 0.070
    e. Some weeds, dense willow on banks ... 0.060 - 0.080
    f. For trees within channel & branches submerged at high stage, increase above values by ... 0.010 - 0.020
  2. Irregular sections, with pools, slight channel meander: Increase values given in 1a. to 1e. by ... 0.010 - 0.020
  3. Mountain streams, no channel vegetation, banks usually steep, trees & shrubs on banks submerged at high stage:
    a. Gravel, cobble, few boulders on bed ... 0.040 - 0.050
    b. Cobbles and large boulders on bed ... 0.050 - 0.070
B. Flood plains, adjacent to natural streams:
  1. Pasture, no shrubs:
    a. Short grass ... 0.030 - 0.035
    b. High grass ... 0.035 - 0.050
  2. Cultivated areas:
    a. No crop ... 0.030 - 0.040
    b. Mature row crops ... 0.035 - 0.045
    c. Mature field crops ... 0.040 - 0.050
  3. Heavy weeds, scattered shrubbery ... 0.050 - 0.070
  4. Light shrubbery and trees:
    a. Winter ... 0.050 - 0.060
    b. Summer ... 0.060 - 0.080
  5. Medium to dense shrubbery:
    a. Winter ... 0.070 - 0.110
    b. Summer ... 0.100 - 0.160
  6. Dense willows ... 0.150 - 0.200

## 22.3 Non Uniform Flow

Truly uniform flow rarely exists in either natural or man-made channels, because changes in channel section, slope, or roughness cause the depths and average velocities of flow to vary from point to point along the channel, and the water surface will not be parallel to the streambed. Flow that varies in depth and velocity along the channel is called non-uniform.

Open channel flow, where gravity dominates, is best characterised by the Froude Number (Fr) and is a useful criteria that distinguishes tranquil subcritical flow and rapid supercritical flow. At the critical flow threshold Fr = 1, for tranquil subcritical flow Fr < 1, and for rapid supercritical flow Fr > 1.

$$Fr = \frac{v}{\sqrt{g y_m}} \qquad \text{Eqn (22-4)}$$

where $v$ = mean velocity
$g$ = gravitational acceleration
= 9.81 $m/s^2$
$y_m$ = mean depth

or, more generally

$$Fr = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} \qquad \text{Eqn (22-5)}$$

where $Q$ = flow ($m^3/s$)
$B$ = surface width (m)
$A$ = cross sectional area ($m^2$).

Other related commonly used flow parameters are:
$y_0$ = uniform flow depth
$y_c$ = critical flow depth
$S_o$ = bed slope
$S_c$ = bed slope for critical flow

With **subcritical flow** (Fr < 1), a change in channel shape, slope, or roughness affects the flow for a considerable distance upstream, and thus the flow is said to be under downstream control. If an obstruction, such as a culvert, causes ponding, the water surface above the obstruction will be a smooth curve asymptotic to the normal water surface upstream and to the pool level downstream.

With **supercritical flow** (Fr > 1), a change in channel shape, slope, or roughness cannot be reflected upstream except for very short distances. However, the change may affect the depth of flow at downstream points, thus, the flow is said to be under upstream control. An example is the flow in a steep chute where the water surface profile draws down from critical path depth at the chute entrance and approaches the lesser normal depth in the chute.

Most problems in channel drainage do not require the accurate computation of water surface profiles, however, designers should know that the depth in a given channel may be influenced by conditions either upstream or downstream, depending on whether the slope is steep (supercritical) or mild (subcritical).

Figure 22-2 shows a channel on a mild slope ($S_o < S_c$), discharging into a pool. The depth of flow (y) between sections 1 and 2 is changing and the flow is non-uniform. The water surface profile between the sections is known as a backwater curve and is characteristically very long.

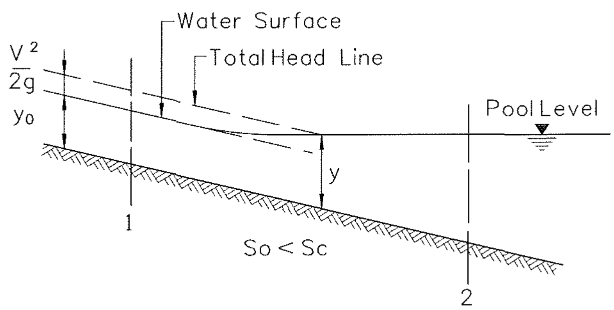


*Figure 22-2: Water surface profile where the channel changes to a pool, for a channel with mild slope.*

Figure 22-3 shows a channel in which the slope changes from subcritical to supercritical. The flow profile passes through critical depth ($y_c$) near the break in slope (section 1). This is true whether the upstream slope is mild ($S_o < S_c$), as in the sketch, or whether the water above section 1 is ponded, as would be the case if section 1 were the crest of a spill way of a dam. Downstream of section 1 the flow depth quickly approaches the normal depth for the steep channel ($S_o > S_c$).

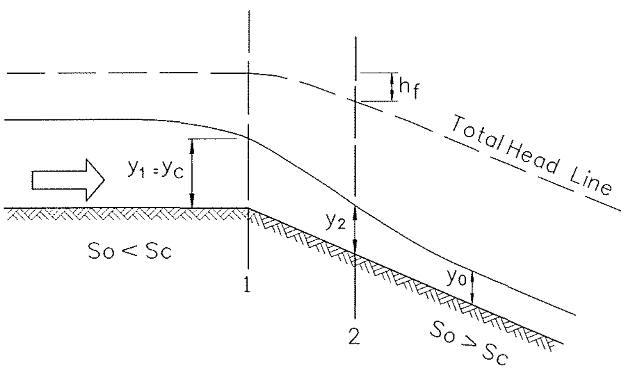


*Figure 22-3: Water surface profile where the channel slope changes from subcritical to supercritical slope.*

Figure 22-4 shows a special case for a steep channel ($S_o > S_c$) discharging into a pool. A hydraulic jump makes a sudden transition from the supercritical flow in the steep channel to the subcritical flow in the pool. The situation differs from that shown in Figure 22-2 because the flow approaching the pool in Figure 22-4 is supercritical and the total head in the approach channel is large relative to the pool depth.

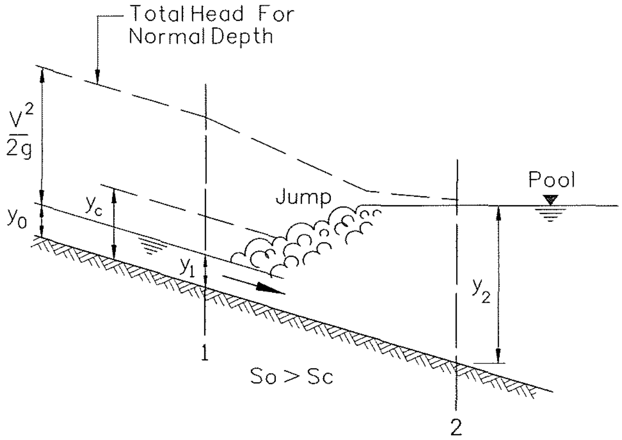


*Figure 22-4: Water surface profile illustrating the hydraulic jump that occurs when a steep channel discharges into a pool.*

In general, supercritical flow can be changed to subcritical flow only by passing through a hydraulic jump. The violent turbulence in the jump dissipates energy rapidly, causing a sharp drop in the total head line between the supercritical and subcritical states of flow. A jump will occur whenever the ratio of the depth in the approach channel ($y_1$) to the depth in the downstream channel ($y_2$) reaches a specific value corresponding to the balance of momentum between the subcritical and supercritical flow.

Note in Figure 22-4 that normal depth in the approach channel persists well beyond the point where the projected pool level would intersect the water surface of the channel at normal depth. Normal depth can be assumed to exist on the steep slope upstream from section 1, which is located about at the toe of the jump.

Note that when the Froude Number (Fr) is less than approximately 1.7, a well-defined turbulent jump does not form because the energy difference between the upstream and downstream sections is so small. Instead there will be a train of unbroken standing waves that can propagate well downstream and cause erosion. Designers must check and allow for this possibility.

## 22.4 Determination of Water Surface Profiles

### 22.4.1 General Considerations

Over the years, methods of various types have been developed for the determination of water surface profiles. Most methods applicable to irregular channels are essentially finite-difference methods, in which the form of the water surface profile is deduced by calculating the stage at each of a series of discrete cross-sections, these usually coinciding with the surveyed cross-sections. The following relation can be deduced from Figure 22-5:

$$\Delta H = H_2 - H_1 = H_f + H_e \qquad \text{Eqn (22-6a)}$$

$$H_2 = H_1 + H_f + H_e \qquad \text{Eqn (22-6b)}$$

where $H_1$ = the total head at cross-section 1, located at the downstream end of the channel reach of length $\Delta x$

$H_2$ = the total head at cross-section 2, which is located at the upstream end of the channel reach of length $\Delta x$

$H_f$ = the head loss due to boundary resistance within the reach

$H_e$ = head loss within the reach due to changes in channel cross-section and alignment

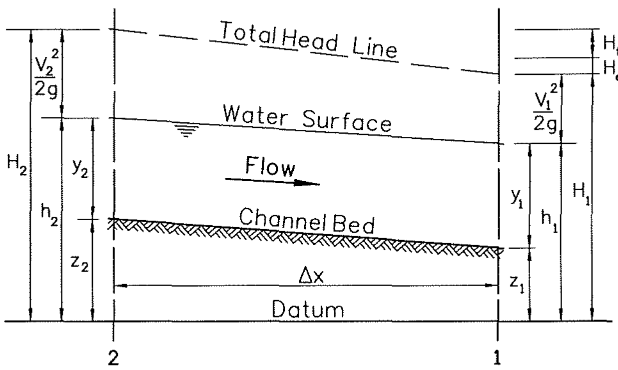


*Figure 22-5: Gradually varied flow longitudinal section.*

When written in terms of the stage the relation is:

$$h_2 + V_2^2/2g = h_1 + V_1^2/2g + H_f + H_e \qquad \text{Eqn (22-7)}$$

where $V^2/2g$ = the velocity head

In principle, each step in the determination of a water surface profile involves the determination of $h_2$ for a given value of the discharge Q and a given stage $h_1$, with the geometric properties and chainages of the cross-sections known along with the data required for the calculation of head losses within the reach.

### 22.4.2 Data for Determination of Water Surface Profiles

In general, the determination of a water surface profile for gradually varied flow at a given discharge is dependent upon the availability of sufficient data to define the geometry of the channel and the boundary roughness.

Definition of the channel geometry will require surveyed cross-sections extending across the full width of the channel and overbank flow areas. Close spaced cross-sections will be required at locations where changes in discharge, channel slope, or boundary roughness occur. In an irregular channel, the spacing and location of the surveyed cross-sections must be such that variations in the shape and size of the channel cross-section are adequately defined.

Changes in channel geometry between cross-sections should be smooth and monotonic, so that, for example, the effect of a narrowing and the subsequent expansion of the channel will be adequately represented. Cross-sections will also be required at structures such as bridges, culverts and weirs, at possible controls, and adjacent to channel branches and junctions. The cross-sections should be oriented at right angles to the expected local flow direction.

### 22.4.3 Head Loss Due to Changes in Channel Cross-section

The term $H_e$ in Eqn (22-7) represents the total head loss within the channel reach due to changes in the size and shape of the channel cross-section and to changes in channel alignment.

In subcritical flow, the head loss ($H_e$) associated with a change in channel cross-section can be expressed in terms of a loss coefficient ($C_L$) and the change in the velocity head. The expression is of the form:

$$H_e = C_L \Delta\left(V^2/2g\right) \qquad \text{Eqn (22-8)}$$

where $\Delta\left(V^2/2g\right)$ = the absolute difference between the velocity heads upstream and downstream from the change in cross-section

The values given in Table 22-2, for the loss coefficient $C_L$ in Eqn (22-8), are suggested in the US Army Corps of Engineers (2001). Note that the head losses associated with channel contractions tend to be somewhat less than those associated with expansions.

### 22.4.4 Head Loss Due to Change in Channel Alignment

The head loss in a channel bend can be estimated from the expression:

$$H_e = C_L\left(V^2/2g\right) \qquad \text{Eqn (22-9)}$$

where V = the mean flow velocity in the channel at the bend

Values of the loss coefficient $C_L$ for channel bends have been discussed by Henderson (1966), who has suggested that for channel bends with deflection angles between 90° and 180°, an estimate of the loss coefficient can be obtained from the expression:

$$C_L = 2B/r \qquad \text{Eqn (22-10)}$$

where B = the channel width

r = the radius of curvature of the channel centre-line

Henderson (1996) notes that in some cases, use of Eqn (22-10) may cause the loss coefficient to be over-estimated, by factors up to about three. Where records of field observations are available it may be possible to derive values of the loss coefficients for particular sites on the basis of these records.

### 22.4.5 Computer Software Systems for the Determination of Water Surface Profiles

The determination of water surface profiles in gradually varied flow can be efficiently computerised, as it involves (in the case of a channel of irregular cross-section) relatively large bodies of data and repetitive calculations. Software suitable for water profile determination includes:

| | |
|---|---|
| HEC-RAS: | US Army Corp of Engineers |
| Mouse: | Danish Hydraulic Institute |
| Mike11: | Danish Hydraulic Institute. |

*Table 22-2: Loss coefficients for a change in channel area.*

| Change Type | Gradual | Abrupt |
|---|---|---|
| Contraction | 0.1 | 0.6 |
| Expansion | 0.3 | 0.8 |

## 22.5 Outfall Water Levels

Pipes and open channels should be designed by checking backwater profiles calculated from an appropriate outfall water level. Failure to do so will likely result in unnecessarily large conduit sizes for free outlet conditions and undersized conduits for drowned outlet conditions. At intermediate inlets and at the upper end of a system, water levels computed from the design flow shall be low enough to prevent overflow and to allow existing and future connections to function satisfactorily. Systems should be reviewed in the light of secondary flow characteristics Refer to *Chapter 14.6: Pipe Inlet Structures,* and *Chapter 14.7: Pipe Outfall Structures* for further information.

If work is being designed upstream of an existing system that has been designed for a lower flow, the new work should be sized as if the old work was about to be upgraded with a system capable of handling the current design flows. Similarly, the new work should be sized on the assumption that future upstream renewal will allow the passage of current design flows.

Increased flow rates from new works may have negative consequences on lower capacity sections in the short-term. Some consideration should be given to temporary 'chokes'.

Stormwater pipelines are often located so as to operate in a surcharged condition at full design flow. The relationship between pipe flow and head loss is shown on the pipe flow nomograph *(Appendix 11: Pipe Flow Nomograph)*.

### 22.5.1 Tidal Outfalls

For areas with tidal outfalls the design must satisfy the basic criteria in Table 22-3. Alternatively it is acceptable to demonstrate by full probability assessment, in combination with tidal cycle data, that the inundation performance standards of *Chapter 20: Inundation Design Performance Standards,* have been satisfied. In addition, for structures, allowance shall be made for sea level rise over the life expectancy of the structure. Adopted sea level rise values shall be as set out in *Appendix 1: Definitions and Useful Numbers.*

When existing development is too low to meet these criteria, each case will need to be determined according to its particular circumstances.

*Table 22-3: Tidal outfall design water levels*

| Flow Scenario | Outfall Water Level | | |
|---|---|---|---|
| | Estuary | Sea/Tidal | River |
| Design storm flow | 10.30 m | 10.3 m | * |
| Half of design storm flow | 10.70 m | 10.8 m | * |
| Typical base flow | 10.85 m | 10.9 m | * |

* Corresponding levels in rivers to be set by the Christchurch City Council.

### 22.5.2 Outfalls to Rivers and Larger Waterways

When a waterway flows into a much larger waterway, the peak flows are unlikely to coincide. Backwater profiles should produce satisfactory water levels when assessed for the two following scenarios.

Firstly:

a) set tributary AEP based on the requirements of *Chapter 20: Inundation Design Performance Standards*
b) determine the tributary critical storm duration D
c) for duration D and AEP, determine the tributary flow ($Q_{trib}$).

Determine Profile 1:

This assumes that a lesser flood is present in the receiving waterway at the time of the tributary peak flow arrival.

d) determine the receiving waterway flood water level for the AEP shown in Table 22-4
e) starting with the level from (d), determine the tributary water profile (Profile 1) at flow $Q_{trib}$.

*Table 22-4: Recipient catchment design AEP*

| Tributary Catchment Area (ha) | Recipient Catchment Area including Tributary (ha) | | | |
|---|---|---|---|---|
| | Up to 500 | 500 to 800 | 800 to 1500 | 1500 plus |
| Up to 500 | 5 | 5 | 5 | 10 |
| 500 to 800 | | 10 | 10 | 20 |
| 800 to 1500 | | | 20 | 20 |
| 1500 plus | | | | 50 |

Determine Profile 2:

This assumes that the tributary waterway peak flow has already passed by at the time of arrival of the receiving waterway's peak flow.

f) determine the receiving waterway peak water level at the AEP from (a) above
g) starting with level (f), determine the tributary water profile (Profile 2) at a flow of 75% of $Q_{trib}$
h) the design profile is the higher of Profile 1 or 2.

## 22.6 Afflux at Bridges and Other Structures

### 22.6.1 Introduction

The design guide here is ancillary to *Chapter 13.2: Bridges and Culverts,* which should be read first.

The provision of a bridge or other structure within a channel often involves a constriction of the channel since, with the objective of reducing the cost of the structure, the width of the waterway at the structure is commonly made less than the width of the channel upstream and downstream. As a result, the flow accelerates as it approaches the structure and passes through the constructed waterway at the structure with a velocity greater than that of the approach flow. A subsequent deceleration of the flow occurs downstream from the structure, as the flow expands into the unconstricted channel downstream.

If no loss of head occurred, then water levels in the channel would be affected only in the immediate vicinity of the structure. This is provided that the channel width at the constriction is greater than the limiting value that would cause the occurrence of critical flow at the constriction. In reality however, head losses will occur as a result of the acceleration and deceleration of the flow.

The total head loss through the structure will be the sum of these head losses, together with the head loss due to boundary resistance. In most cases a large proportion of the total head loss will occur in the expansion of the flow downstream from the structure. If the flow in the channel is subcritical, the head loss through the structure will cause an increase in water levels upstream from the structure; this increase in water levels is commonly referred to as "afflux" or "backwater".

Procedures for assessing afflux are described in Pilgrim (1987).

### 22.6.2 Effect of Channel Scour on Afflux

Increased velocities that will occur in the constructed waterway at a bridge will tend to scour material from the channel bed in the vicinity of the structure.

Where scour of the channel bed is expected, it is necessary to examine the effect of this scour on the stability of bridge piers and abutments and on any other natural or artificial feature likely to be affected.

## 22.7 Bed Shear Stress and the Stable Bed

Stability of a substrate depends on the gravitational restraining force exceeding the fluid drag force. The average fluid drag on the water/substrate interface can be expressed as a bed shear stress, also known as "tractive force", as below:

$$\tau_0 = \gamma RS \quad (N/m^2) \qquad \text{Eqn (22-11)}$$

where
$\gamma$ = $\rho g$ $(N/m^3)$
$\rho$ = $1000\ kg/m^3$
= fluid density
$g$ = $9.81\ m/s^2$
$R$ = hydraulic radius (m)
$S$ = friction slope (m/m)
= approx water surface slope

#### Low Turbulence Flow

If flow turbulence is low (typically $y/d_{75} > 5$ and $Fr < 1.5$ where $y$ = water depth) then the US Bureau of Reclamation recommended safe value of shear stress $\tau_0$ for noncohesive materials greater than 6mm diameter can be determined from Eqn (22-10; refer, Henderson 1966, Pg417). $d_{75}$ is the reference particle size because it is this size that tends to armour the natural stream.

$$\tau_0 = 0.75\ d_{75} \quad (N/m^2) \qquad \text{Eqn (22-12)}$$

Where $d_{75}$ is in mm

Combining Eqn (22-11) and Eqn (22-12) an expression is obtained that enables the stable $d_{75}$ size to be obtained directly for a wide channel:

$$d_{75} = 11RS \qquad \text{Eqn (22-13a)}$$

where $d_{75}$ is in m
$R$ is in m
$S$ is in m/m

For a channel of shallow parabolic cross section the peak bed shear stress on the line of maximum depth is higher than the average value of (22-13a), thus:

$$d_{75} = 19RS \qquad \text{Eqn (22-13b)}$$

Equation (22-13b) above, can be combined with the following Strickler formula for Manning's n:

$$n = 0.012d^{1/6} \qquad \text{Eqn (22-14)}$$

where $d = d_{75}$ (mm)

and Manning's Eqn (22-1) to give a safe maximum channel slope, thus:

$$S = 33d^{1.15}Q^{-0.46} \ (\text{m/m}) \qquad \text{Eqn (22-15)}$$

where d is in m

Q is in $m^3/s$

Table 22-5 below includes a tabulation of velocity limit values at $y/d_{75} = 5$. For $y/d_{75} > 5$ these velocities will be conservative and so the designer could instead make use of Eqn (22-15) to directly assess bed stability.

*Table 22-5: Stable bed velocities*

| Isbash | | Eqn(22-13) | y/d = 5 |
|---|---|---|---|
| $d_{50}$ (m) | vel (m/s) | $d_{75}$ (m) | $V_{max}$ (m/s) |
| | | 0.003 | 0.40 |
| | | 0.006 | 0.60 |
| | | 0.010 | 0.75 |
| 0.020 | 1.0 | 0.020 | 1.1 |
| 0.030 | 1.2 | 0.030 | 1.3 |
| 0.040 | 1.4 | 0.040 | 1.6 |
| 0.060 | 1.7 | 0.060 | 1.9 |
| 0.080 | 1.9 | 0.080 | 2.2 |
| 0.100 | 2.2 | 0.100 | 2.5 |
| 0.120 | 2.4 | 0.120 | 2.7 |
| 0.150 | 2.6 | 0.150 | 3.0 |
| 0.200 | 3.1 | 0.200 | 3.3 |
| 0.250 | 3.4 | | |
| 0.300 | 3.7 | | |
| 0.350 | 4.0 | | |
| 0.400 | 4.3 | | |
| 0.450 | 4.6 | | |
| 0.500 | 4.8 | | |
| 0.600 | 5.3 | | |
| 0.700 | 5.7 | | |
| 0.800 | 6.1 | | |
| 0.900 | 6.5 | | |
| 1.000 | 6.8 | | |

### High Turbulence Flow

If flow turbulence is high (typically $y/d_{50} < 5$ and $Fr > 1.5$) then use Eqn (22-16) based on the Isbash formula. Note that Isbash (1936) based his work on placed material based on $d_{50}$ size rather than $d_{75}$ used for Low Turbulence Flow design above.

$$d_{50} = K\frac{v^2}{2g} \qquad \text{Eqn (22-16)}$$

Here K incorporates an allowance for the degree of turbulence and stone angle of internal friction. For strong turbulence above a well-interlocked bed of very angular material of basalt or greywacke (specific gravity > 2.4), and shape factor no greater than 3, K may be taken as 0.42. This value is consistent with recommendations of the MWD Culvert Manual (Ministry of Works and Development 1978). Shape factor is defined as rock maximum dimension divided by minimum dimension.

Where downstream bed slope is > 5% then multiply $d_{50}$ by:

$$ma = \frac{\sin(\Phi)}{\sin(\Phi-\beta)} \qquad \text{Eqn (22-17)}$$

where $\Phi$ = angle of repose

= 45° for broken rock

= 30° for round rock

$\beta$ = slope angle

For channel side slope, multiply $d_{50}$ by:

$$mh = \frac{1}{\sqrt{1-\left(\frac{\sin\beta}{\sin\Phi}\right)}} \qquad \text{Eqn (22-18)}$$

Achievement of a "well interlocked" bed is likely to require a layer thickness of at least $2d_{50}$ on top of smaller sized scour resistant underlayers. The largest rocks ($d_{100}$) should fit within this layer and no more than 15% of rock should be smaller than $0.6d_{50}$. Rock should be sound, durable stone, free from cracks and seams. Deleterious material including soft, friable particles should not exceed 5%.

For granular bed materials, and Fr > 1.5, flow jetting must not be able to penetrate the voids beneath the armouring because of the uplift pressures that may develop. In such situations the filling of voids with cohesive material is recommended. Strongly rooted grass cover or tree root matting can assist in reducing velocities and increasing scour resistance.

### Silt or Sand Substrate

For sand or silt stream beds the assessment is less straightforward; there is nearly always some sediment movement, the suspended sediment load affects bed shear stress, and for fine substrates, the channel roughness relates to factors other than particle size. Work by Simons and Albertson (as outlined by Henderson 1966), could be considered. In the absence of other information, assume a $V_{max}$ of 0.3 m/s for silt or sand substrates.

## 22.8 Riffle, Run, Pool Design

Portions of a river are often characterised as riffle, run, or pool. Riffles are shallow swift flowing areas with broken water surface and larger substrate (gravel, cobbles, or boulders). Pools are deeper slow flowing areas with an almost level smooth water surface and often containing deposits of finer substrate (sand or gravel). Runs are intermediate between pools and riffles and are characterised by an undulating but relatively unbroken water surface.

These classifications appear to best relate to the Froude Number (Fr) that relates inertial forces to gravity forces. This is important hydraulically wherever gravity dominates (e.g. waves and open channel flow), and is the criteria that distinguishes tranquil and rapid flow such as occurs in riffles, runs and pools. See also *Chapter 22.3: Non Uniform Flow.*

The basic Froude Number equation repeated here is:

$$Fr=\frac{v}{\sqrt{gy_m}} \qquad \text{Eqn (22-4)}$$

where v = mean velocity

g = gravitational acceleration

= 9.81 m/s$^2$

$y_m$ = mean depth

The riffle, run, pool criteria are further described in *Chapter 9.3.2.1: Design Considerations for Riffles,* and shown graphically on Figure 9-5.

The riffle/run threshold is set to: Fr = 0.35

The run/pool threshold is set to: Fr = 0.15

### Riffle, Run, Pool Design Steps

1) assume a riffle height and slope; say 100 mm height, 1:100 grade, 10 m length
2) space the riffles as per *Chapter 9.3.2: Riffle, Run, Pool Sequences*
3) assume an intermediate run gradient of say 1:500
4) analyse the channel for "normal" base flow as might occur in say, December
5) ensure the riffle criteria of Figure 9-5 are met (see *Chapter 9.3.2.1: Design Considerations for Riffles)*
6) adjust the riffle/run gradients and riffle spacings as required, then repeat step 4
7) re-analyse the channel for a 6 month "flood" flow
8) determine the minimum $d_{75}$ size using either Eqn (22-13b) or Table 22-5
9) repeat for larger flood events to ensure the $d_{75}$ remains stable
10) ensure the bed shear stress is sufficient to sweep clear any surface sand size particles or smaller.

The final substrate should have a $d_{75}$ size at least as large as determined above, but can be larger. $d_{75}$ should ideally be in the range of 16–160 mm. There should be a good mixture of particle sizes.

## 22.9 Hydraulic Design of Culverts and Pipes

The design guide here is ancillary to *Chapter 13.2: Bridges and Culverts,* which should be read first.

The content of this section is adapted from the Australasian Concrete Pipe Association's hydraulic design manual (Concrete Pipe Association of Australasia 1994), which should be referred to for additional guidelines and design charts.

Culverts, despite their apparent simplicity, are complex engineering structures from a hydraulic as well as a structural viewpoint. Their function adequacy is no better than the estimate of the design flood, and the hydraulic design described below must be preceded by a careful flood evaluation together with an assessment of effects caused by the design flood being exceeded or blockage of the culvert entry.

The hydraulic complexity of culverts is a result of the many parameters influencing the flow pattern. This influence can be summarised to two major types of culvert flow control:

- flow with inlet control
- flow with outlet control

both of which require evaluation to determine which will govern.

While the hydraulic design of culverts and pipes is critical, design must also consider ecological and aesthetic values. Refer to *Chapter 13.2.3: Designing Fish Friendly Culverts and Weirs,* and other sections in *Chapter 13.2: Bridges and Culverts,* for more information.

### 22.9.1 Types of Culvert Flow Control

#### Determination of Operating Condition

The most restrictive of the two flow types, inlet control or outlet control, applies, that is, the one giving least discharge for given headwater level, or requiring higher headwater level for given discharge.

#### Flow with Inlet Control

Culvert flow is restricted to the discharge that can pass the inlet with a given headwater level (see Figure 22-6):

- The discharge is controlled by the headwater depth, the cross-section area at the inlet, and the geometry of the inlet edge. It is not appreciably affected by length, roughness, slope, or outlet conditions, and the culvert is not flowing full at any point except perhaps at the inlet. This culvert type is mostly short or steep.

The culvert flow is restricted to the discharge which can pass through the pipe and get away from the outlet with a given tailwater level (Figure 22-6):

- The culvert can run full over at least some of its length. The discharge is affected by the length, slope, roughness, and outlet conditions, in addition to the depth of headwater, the cross-section area, and inlet geometry.

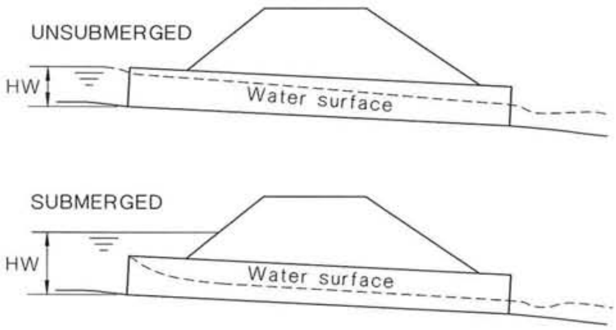


*Figure 22-6: Inlet flow control.*

#### Flow with Outlet Control

The simplest case of outlet control is illustrated in Figures 22-7a and 22-7b. Here the culvert is flowing full for its entire length. The energy head ($\Delta H$) required to maintain this flow equates to the sum of the losses and can be expressed.

$$\Delta H = H_v + H_e + H_f \qquad \text{Eqn (22-19)}$$

where $H_v$ = velocity head

$= V^2/2g$

$H_e$ = entry loss

$= k_e H_v$

$H_f$ = friction loss

The entrance loss coefficient $k_e$ is given in Table 22-6 for various pipe and box culvert entry conditions and culverts flowing with outlet control.

*Table 22-6: Entrance loss coefficient ($k_e$).*

| Culvert Entrance Type | $k_e$ |
|---|---|
| **Pipe Culverts** | |
| Pipe projecting from fill: | |
| square cut end | 0.5 |
| socket end | 0.2 |
| Headwall with or without wingwalls: | |
| square end | 0.5 |
| socket end | 0.2 |
| Pipe mitred to conform to fill slope: | |
| precast end | 0.5 |
| field cut end | 0.7 |
| **Box Culverts** | |
| No wing walls, headwall parallel to embankment: | |
| square edge on 3 edges | 0.5 |
| 3 edges rounded $^1/_{12}$ barrel dimension | 0.2 |
| Wing walls at 30 to 75 to barrel: | |
| square edge at crown | 0.4 |
| crown rounded to $^1/_{12}$ culvert height | 0.2 |
| Wing wall at 10 to 30 to barrel: | |
| square edge at crown | 0.5 |
| Wing walls parallel (extension of sides): | |
| square edge at crown | 0.7 |

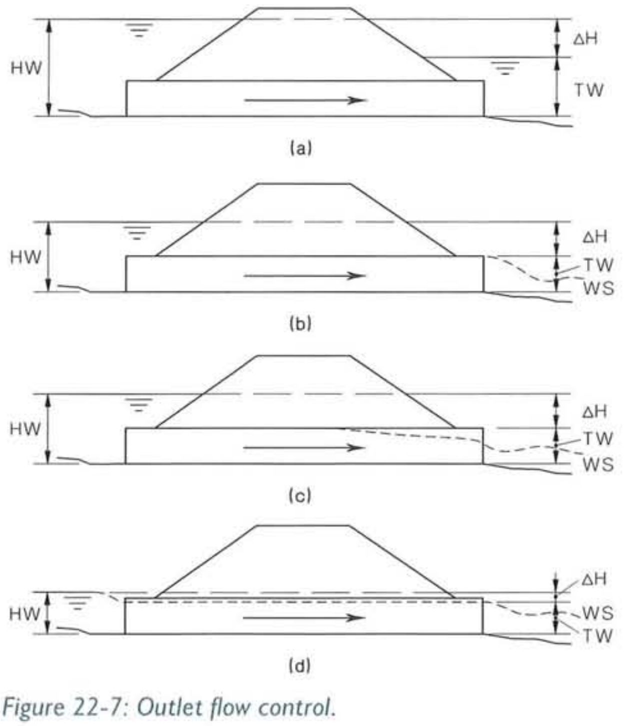


*Figure 22-7: Outlet flow control.*

Note: the velocity head term above ($H_v$) is required at the entrance to accelerate the flow to velocity V. It is then lost at the exit due to sudden expansion.

The friction loss ($H_f$) is ideally calculated from the Colebrook-White equation but is more readily and more widely calculated using the Manning formula. See *Section 22.2.2: The Manning Formula.*

For circular culverts flowing full, Manning's formula reduces to:

$$S_f = 10.3\, n^2\, d^{-16/3}\, Q^2 \qquad \text{Eqn (22-20)}$$

From which the friction loss ($H_f$) can be determined as:

$$H_f = L.S_f \qquad \text{Eqn (22-21)}$$

Knowing the energy differential head ($\Delta H$) and tailwater level (TW), the headwater (HW) can be calculated from the equation:

$$HW = TW + \Delta H - L.S_o \qquad \text{Eqn (22-22)}$$

where L = culvert length

$S_o$ = culvert slope

For culverts and pipes not flowing full, refer to Concrete Pipe Association of Australasia (1994).

### 22.9.2 Headwater

Headwater (HW) is the depth of water at the inlet above the invert of the culvert. It is influenced by factors such as:

- acceptable upstream flooding
- culvert flow velocity
- overtopping of the roadway
- the possibility of water penetration into the embankment.

Culvert hydraulic performance criteria are set out in *Chapter 13.2: Bridges and Culverts*, and *Chapter 14: Pipeline Structures.*

### 22.9.3 Tailwater

Tailwater (TW) is the depth from the natural water surface at the outlet to the invert of the culvert. The tailwater level may be governed by downstream obstructions or the discharge from other streams.

### 22.9.4 Freeboard

Freeboard is the distance between the headwater level and the crown of the culvert. See *Chapter 13.2.2: Design Considerations.*

### 22.9.5 Culvert and Pipe Bend Losses

As a guide Table 22-7 gives typical bend head loss coefficients ($k_b$, excluding changes in hydraulic grade line due to changes in velocity head which should also be allowed for). These figures can also be used for manhole bends when properly benched and with bend radius 1.5 times pipe diameter.

Energy head loss:

$$H_e = k_b V^2/2g \qquad \text{Eqn (22-23)}$$

where h is in metres

V is in m/s

*Table 22-7: Typical bend loss coefficients ($k_b$).*

| Bend Angle | $k_b$ |
|---|---|
| 90 | 0.90 |
| 45 | 0.60 |
| 22.5 | 0.25 |

### 22.9.6 Erosion

A check on velocities must be considered as part of the design process as culverts generally increase the flow velocity over that in the natural water course which can lead to erosive effects both within the culvert and immediately downstream. Examples of limiting velocities are shown in Table 22-5.

### 22.9.7 Air Entrainment

Where a pipe gradient exceeds 1 in 10 over a fall height exceeding 2 metres, an allowance for the bulking of the flow due to air entrainment should be made. This allowance is made by increasing the area of the pipe for the additional volume of air in the flow. Failure to add this allowance can lead to choking in the main line and backflow out sidelines.

The air to water ratio may be calculated from the formula:

$$\frac{\text{air}}{\text{water}} = \frac{kv^2}{gR} \qquad \text{Eqn (22-24)}$$

where k = coefficient of entrainment

= 0.004 for smooth concrete pipes

= 0.008 for cast-in-situ concrete culverts

v = velocity (m/s)

R = hydraulic radius (m)

g = acceleration due to gravity ($m/s^2$)

Having determined the pipe size for 'basic' water flow, pipe size should be increased to provide cross-section area for 'water + air'. This equates to scaling the basic diameter for water alone as in Eqn (22-25).

$$\text{dia}_{\text{reqd.}} = \left(1 + \frac{kv^2}{gR}\right)^{1/2} \text{dia}_{\text{basic}} \qquad \text{Eqn (22-25)}$$

### 22.9.8 Siltation

If the flow velocity becomes too low, siltation will occur. Flow velocities below about 0.5 m/s will cause settlement of fine to medium sand particles.

Siltation in pipes and culverts mostly occurs if they are placed at incorrect levels leading to low velocity in the pipe or culvert that is lower than the average stream flow.

To be self-cleaning culverts must be graded to the average grade of the watercourse upstream and downstream of the culvert, and water levels must represent the average stream levels before the culvert was built.

## 22.10 Sumps—Collection of Water from Side Channels

### 22.10.1 Side Channel Flow Capacity

Standard side channels are shown on standard details SD601 and SD604 included in the Construction Standard Specification, CSS: Parts 2 and 3 (see Christchurch City Council 2002a, b). Flow capacities are shown in Table 22-8.

In no case shall a side channel be designed to convey more than 175 l/s. The designer should note that where velocity exceeds 1.5 m/s there is a danger of erosive damage to the pavement and special topcourse and pavement design may be required to guard against this. Alternatively the flow should be entirely contained within the fender.

Care should be exercised with flow around bends when velocity is greater than 1m/s as momentum will tend to lift flow out of the channel. Channel reduced capacity around bends can be derived by assuming a water surface crossfall from Eqn (22-26). Plotting this

*Table 22-8: Side channel flow capacity (based on Manning's n = 0.016).*

| Gradient 1 in ... | Slope | Standard Flat Channel (SD201) Top of kerb = 130 mm $A = 0.12\ m^2$ | | | Standard Flat Channel (SD201) Top of fender = 40 mm $A = 0.0076\ m^2$ | | | Hillside Channel (SD204) Top of kerb = 165 mm $A = 0.04\ m^2$ | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | flow (l/s) | velocity (m/s) | depth (mm) | flow (l/s) | velocity (m/s) | depth (mm) | flow (l/s) | velocity (m/s) | depth (mm) |
| 500 | 0.0020 | 50 | 0.40 | 130 | | | | 18 | 0.45 | 165 |
| 400 | 0.0025 | 55 | 0.44 | 130 | | | | 20 | 0.51 | 165 |
| 300 | 0.0033 | 64 | 0.51 | 130 | | | | 23 | 0.59 | 165 |
| 200 | 0.005 | 78 | 0.62 | 130 | | | | 29 | 0.72 | 165 |
| 100 | 0.010 | 110 | 0.88 | 130 | | | | 41 | 1.0 | 165 |
| 70 | 0.014 | 130 | 1.1 | 130 | | | | 49 | 1.2 | 165 |
| 50 | 0.020 | 155 | 1.2 | 130 | | | | 57 | 1.5 | 165 |
| 40 | 0.025 | 175 | 1.4 | 130 | 5 | 0.68 | 40 | 64 | 1.6 | 165 |
| 30 | 0.033 | 175 | 1.6 | 125 | 6 | 0.79 | 40 | 75 | 1.9 | 165 |
| 20 | 0.050 | 175 | 1.8 | 117 | 7 | 0.96 | 40 | 75 | 2.2 | 155 |
| 15 | 0.067 | 175 | 2.0 | 112 | 9 | 1.11 | 40 | 75 | 2.5 | 145 |
| 10 | 0.100 | 175 | 2.4 | 105 | 10 | 1.36 | 40 | 75 | 2.9 | 130 |
| 8 | 0.125 | 175 | 2.6 | 100 | 12 | 1.52 | 40 | 75 | 3.1 | 125 |
| 6 | 0.167 | 175 | 2.9 | 95 | 13 | 1.76 | 40 | 75 | 3.5 | 115 |
| 4 | 0.250 | 175 | 3.4 | 90 | 17 | 2.15 | 40 | 75 | 4.1 | 105 |

Unshaded flows are depth limited.
Shaded flows are set capacity limits.
Capacity must be reduced around bends (see *Chapter 22.10.1: Side Channel Flow Capacity*).

crossfall on the appropriate standard detail will give a flow area reduction that can be taken as proportional to the flow obtained from Table 22-8.

crossfall $= v^2/gr$ towards inside of bend Eqn (22-26)

where $v$ = velocity from Table 22-8

$r$ = bend radius to centreline of flow

$g$ = 9.81

e.g. $v^2/gr = 0.2$

= 1 in 5 water surface crossfall

### 22.10.2 Sump Capacity

The standard flat channel and hillside channel grate types referred to here (SD324 and SD325), are shown in the CCC Construction Standard Specification CSS:Part3 (Christchurch City Council 2002b). Capacities of sump gratings shown below allow for partial blockage of the sump.

Note: The capacities shown here allow for partial blockage of the sump grating.

#### Standard Flat Channel Sump (SD325)

Inline sump capacity = 20 l/s per grating unit

Valley position capacity = 40 l/s per grating unit

Note that this type of sump grating has a much reduced capacity if the flow velocity exceeds 1 m/s.

#### Standard Hillside Sump (SD324)

The maximum capacity of the standard hillside sump shall be taken as 80 l/s for the 2 m length shown on SD324 but this detail can be altered in length as required based on 40 l/s per m length provided a check is made on the flow capacity of the outlet, especially the pipe entrance. The channel beneath the grate may also require enlargement for grating lengths greater than 2 m.

This type of grating may have difficulty with flow interception if velocity exceeds 2.5 m/s.

## 22.11 Weirs and Free Overfalls

### 22.11.1 Weirs

Formulae are presented here for three common weir types. Formulae for other weir profiles can be found in many hydraulic textbooks.

#### Sharp Crested Weirs

The formula for flow over a sharp crested weir of infinite width and infinite upstream depth is:

$$Q = B \times 1.8\, H^{1.5} \quad (m^3/s) \qquad \text{Eqn (22-27)}$$

Where approach depth is shallow, replace the discharge coefficient 1.8 with (1.8 + 0.23 H/W) where H is upstream water level height above the weir crest (metres) and W is weir crest height above upstream invert (metres).

For rectangular weirs, side contraction can also be significant. To allow for side contraction replace crest width B with (B - 0.2 H).

#### Broad Crested Weirs

Eqn (22-28) is applicable where the crest length in the flow direction is approximately 3H and the weir has a rounded leading edge. For greater crest length, boundary layer effects reduce the discharge coefficient.

$$Q = B \times 1.7\, H^{1.5} \qquad \text{Eqn (22-28)}$$

#### Ogee Weir

This is the classic parabolic like weir profile for which:

$$Q = B \times 2.2\, H^{1.5} \qquad \text{Eqn (22-29)}$$

### 22.11.2 Free Overfalls

The gravity free trajectory parabola, beginning with horizontal velocity (v) and zero vertical velocity, is defined by:

$$y = \frac{gx^2}{2v^2} \qquad \text{Eqn (22-30)}$$

where $y$ = the vertical fall below the lip

$x$ = the horizontal distance from the lip

$v$ = the initial horizontal velocity

Residual internal pressure at a free overfall yields a depth and velocity above critical at the lip, leading to a brink depth ($y_b$) of $0.715y_c$ and velocity $v_b$ approximately $1.5v_c$. This initial velocity can be applied to Eqn (22-30) to give an approximation to the centreline of the free overfall jet.

For thin, decorative waterfalls, surface tension can lead to significant lateral contraction of the free falling jet. To minimise this contraction and other effects, the flow should be at least 6 l/s per metre width and depth ($y_c$) should be greater than 15 mm.

## 22.12 References


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