SCOUR AT BRIDGE PIERS AND ABUTMENTS

by

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of the requirements for the degree
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DECLARATION

I, Patrick Robert Little, hereby declare that this thesis is my own work and that it has not been submitted for a degree at another University

Signed by candidate

December 1977

SYNOPSIS

In this thesis several of the formulae that have been put forward to predict scour are examined and some of their limitations pointed out in an attempt to clarify some of the research work that has been done on scour. The formulae are applied to two known bridge failures and their accuracy in those particular situations examined.

The usual methods of scour prevention and protection are discussed.

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LIST OF SYMBOLS

The letter symbols used are defined where they appear, in the illustrations or the text, and are also arranged below in alphabetical order and defined.

Wherever possible the symbols as originally used by the various authors have been adhered to, but many of the symbols as defined below have, of necessity, been altered.

Subscript 'o' attached to any symbols refers to the section at the bridge (the contracted reach).

Subscript '1' refers to a section upstream of the bridge in the uncontracted reach.

a	area of sand particle over which stress acts (m^2)
A	waterway area at bridge at peak flood using unscoured profile (m^2)
b	actual or estimated width of scour hole (m)
b_s	calculated width of scour hole (m)
B	diameter of circular shield or pier lip in experiments by Thomas (Figure 4.4).
$\bar{c}$	mean sediment concentration (bed load plus suspended load) transported by stream as percent by weight. Subscripts show section being considered
C_d	a constant
C_s	a constant
C_D	the coefficient of drag of a sediment particle
d_a	depth of scour when using a scour arrester (m)
d_p	depth of protective layer (e.g. rip-rap) below bed level (m)

d_s depth of scour (measured from original bed level) (m)

d'_s a calculated depth of scour which may still be subject to alteration by various factors such as angle of flow attack et al. (m)

D the diameter of a sediment particle (m)

D_r the diameter of rock used in rip-rap protection (m) subscripts 15, 16, 50, 84, 85 in association with D and D_r refer to the size of material for which 15, 16, 50, 84, 85 percent by weight is finer

E Distance between abutment edge and centre line of adjacent pier (m)

f Lacey silt factor (Table 2.4)

F Froude number $V/\sqrt{gy}$ subscripts show the section being considered

F_{bo} Blench's zero-bed factor (Figure 2.22)

k a coefficient in Laursen's sediment concentration formula

k_s equivalent roughness of bed

K a factor of value one or less used by Holmes

K_A a coefficient (Ahmad, Table 2.5)

K_B a coefficient used by Blench

K_G a coefficient dependent on the drag coefficient of a typical sedimentary particle of bed material used by Garde et al (Figure 2.28)

K_L a coefficient for the effect of pier nose shape and alignment of the axis of symmetry to the flow direction used by Larras, Table 2.8

K_s a coefficient for the effect of pier (Table 2.2) or abutment (Table 2.3) shape on the depth of scour

K_α a coefficient greater than unity for the effect on the depth of scour of the angle of incidence between the direction of flow and the axis of the pier (Figure 2.5)

K_θ a coefficient for the effect on the depth of scour of the angle of incidence between the approach fill of the abutment and the direction of flow (Figure 2.9)

K_τ a coefficient greater than unity for the effect on the depth of scour of a change in mode of sediment movement (Figure 2.6)

K_1 a coefficient depending on pier geometry and angle of flow attack in the Maza and Sanchez equation (Figure 2.31)

K_2 a coefficient depending on depth of flow, pier width and velocity of flow in the Maza and Sanchez equation (Figure 2.32)

ℓ pier length (m)

ℓ_a nominal length of the abutment (m)

ℓ_e effective length of an encroaching abutment (m)

L length of the constriction or contracted reach (m)

m mass of sand particle (kg)

n the Manning roughness coefficient. Subscripts refer to the section being considered (Table 2.6)

N_s Carsten's sediment number

p an exponent dependent on the drag coefficient of a typical sedimentary particle of the bed material used by Garde et al (Figure 2.28)

q discharge intensity - flow divided by waterway width ($m^3/s/m$)

q' the rate at which sediment is transported out of the scour hole or area (m^3/s)

q" the rate of deposition of sediment into the scour hole or area (m^3/s)

Q total discharge of stream passing under bridge (m^3/s)

Q_a that portion of the total discharge flowing on the overbank area or flood plain (m^3/s)

Q_b the discharge over nominal width b (m^3/s)

Q_c that portion of the discharge flowing within the confines of the normal banks of the main channel of the stream (m^3/s)

Q the discharge approaching an encroaching abutment (m^3/s)

r the ratio of maximum depth of scour to the depth of scour in a long contraction

R_e pier Reynolds number ($= V_1 w/\nu$)

R_H hydraulic radius (m)

S gradient of bed

S_r relative density of rock

S_s relative density of sediment

V velocity of flow. Subscripts show the section being considered (m/s)

V_a velocity of flow approaching bridge - as defined by Holmes in equation (37) (m/s)

V_c the critical velocity or the flow velocity at which particles on the bed of the stream begin to move (m/s)

V_f the fall velocity of a sedimentary particle (m/s)

V_s the shear or friction velocity (m/s)

$V_{\frac{1}{2}}$ mean velocity in half the channel width on the side of the abutment or spur dike (m/s)

w pier width (m)

W surface waterway width. Subscripts show section being considered (m)

W_2 waterway width of main channel flow upstream of bridge (m)

x_a lateral extent of scour arrester (m)

y mean depth of flow equal to waterway area divided by waterway width. Subscripts show section being considered (m)

y_B depth of flow as defined by Blench (m)

y_L Lacey mean depth (m)

y_m depth of flow upstream of the zone of scour (m)

y_s total scoured depth (measured from water surface) (m)

y_{sg} scoured depth due to general scour (measured from water surface) (m)

$y_{s\ell}$ depth of scour due to local effects (m)

z_s depth of circular shield, or flat plate, below bed level in experiments by Schneible (Figures 4.6 and 4.7)

z_T height of circular shield above bed level in experiments by Thomas (Figure 4.4)

α angle between axis of symmetry of pier and flow direction

β constriction ratio (W_o/W_1)

γ_s specific weight of sediment (kN/m^3)

γ_w specific weight of water $(9,81\ kN/m^3)$

θ angle of incidence between the approach fill of an abutment and the direction of flow

ν kinematic viscosity of water (Table 2.7)

ρ_s mass density of sediment

ρ_w mass density of water $(1000\ kg/m^3)$

σ_ϕ the standard deviation of the particle size distribution of the bed material $(= \sqrt{D_{84}/D_{16}})$. Subscripts show the section being considered.

ΣF_m sum of all forces tending to disturb the sedimentary particle on the bed

ΣF_R sum of all forces stabilising a particle on the bed

τ average shear stress on bed (N/m^2)

τ_c critical tractive force or the critical stress at which a sedimentary particle may be first disturbed (N/m^2)

ϕ the natural angle of repose of sedimentary bed material under water

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CHAPTER 1

Introduction

Although bridge failure may be induced by several causes in the majority of cases that failure has been caused by scour and subsequent foundation settlement and collapse. A great deal of money is spent, and wasted, each year both in protecting bridges from this phenomenon and in replacing and repairing them after their failure.

Scour has been defined in the ASCE Manual No 43 as 'The erosive action of running water in streams, in excavating and carrying away material from the bed and banks. Scour may occur in both earth and solid rock material'.

The scour problem has been reviewed by numerous researchers and a host of methods has arisen which can be used to predict the maximum likely scour depth at any site in order that bridge foundations may be safely set below that level. The object of this thesis is to present these methods in such a way that they can be easily applied in the design of a bridge.

Unfortunately the terminology used by the various researchers has not been consistent. To prevent confusion the following terms will be used - as defined by C.R. Neill (58).

1. Erosion - removal of solids by waves and currents; a general term embracing overland erosion, bank erosion, cliff erosion, etc.

2. Scour - erosion by flowing water resulting in undermining of foundations or banks, or in lowering of a stream bed below its natural or average level.

3. Degradation - progressive lowering of a channel bed as a result of geological or regime changes.

4. General or regional scour - scour over a substantial area or across a channel, generally resulting from enhanced velocity of flow - as below a spillway or in a contraction.

5. Local scour - scour confined to a small area around an obstruction or geometric anomaly as at a pier, spur or sharp bend; generally associated with three dimensional flow and vortices.

6. Depth of scour - depth to which material is removed below its original or normal level. (The normal level may have to be defined somewhat arbitrarily)

7. Scoured depth - depth from water surface to bottom of scour; equal to depth of scour plus normal depth of flow.

When a bridge is used to span a river there are four interrelated factors that may cause a change in bed elevation under the bridge.

i) Degradation or aggradation. This is a slow continuous process which can be calculated if bed levels for many previous years are available. However the process is so slow that it can be ignored in all but the most exceptional cases.

ii) River Morphology. This is of particular importance in braided or meandering streams. In a braided river at low stages only a small part of the channel may be used. In floods the deep part of the channel shifts and may move alongside a pier. Similarly the deepest part of a meandering river shifts with the stream and may uncover bridge foundations. The cut-off of a meander will lower the river bed for a considerable distance upstream as the slope readjusts. Meandering rivers tend to be deepest at the outside of their bends. These and similar effects should not take the bridge designer by surprise but as yet no rigid formula has been established to describe them and the engineer must rely on his judgement. Their unpredictability is such though that D.V. Joglekar (29) can cite examples of two bridges on tributaries of the Manas River in India which had to be rebuilt respectively three and seven times to accommodate changes in the braided river.

iii) General or Regional Scour. This type of scour will be caused at bridges by a contraction of the normal river width which results in an increase in flow velocity. The flow in the contracted section is capable of transporting more sediment than the upstream reach and thus the river bed in the vicinity of the bridge will scour. Several factors, other than the encroachment of abutments into the river, may reduce the normal flow area. These may be a) the presence of bridge piers b) accumulation of debris on piers and abutments and c) the convergence of two rivers just upstream of the bridge.

iv) Local Scour. This occurs in the immediate vicinity of the obstruction that causes it. The extent of local scour is dependent on the size and shape of that obstruction, the local flow velocity and other factors. The obstruction which may be a pier or abutment creates turbulence and vortices in the flow causing a portion of the flow to dive. This diving portion impinges on the bed material and excavates it while the remainder of the flow sweeps the scoured particles downstream.

This thesis is a study primarily of the scour problem, or factors three and four above.

Scour may be further subdivided into two types. The first is clear water scour where the flow approaching the bridge contains no sediment. This type of scour may occur immediately downstream of a dam, at the mouth of a lagoon or lake or at a relief bridge. The second type is probably the more commonly occurring and is termed scour with general sediment motion. In this type of scour the approaching flow contains a load of sediment part of which may be deposited in the scour hole or area. From a general scour equation the fundamental difference between these two types may be appreciated. The equation states that the rate of scour is equal to the difference in the rates of excavation and deposition in the scour hole or area. In symbols this can be written

$$\frac{dV}{dt} = q' - q''$$

where $\frac{dV}{dt}$ is the rate of scour (m^3/s)

q' is the rate at which sediment is transported out of the scour hole or area (m^3/s)

q'' is the rate of deposition of sediment into the hole (m^3/s)

Clear water scour occurs only when $q'' = 0$ and $q' > 0$ and limiting scour is reached when the flow no longer has any capacity to excavate material from the scour hole. Scour with general sediment motion occurs with $q'' > 0$ but $q' > q''$ or while the flow's capacity to excavate material is greater than its rate of sediment depostion into the hole. Scour ceases when these two become equal or $q'' = q'$.

E.M. Laursen, probably the most prominent of the researchers of the scour problem, argues that this fundamental difference leads to important conclusions - namely that although the extent of clear water scour is dependent on flow velocity and sediment size, the depth of scour with general sediment motion is independent of these two quantities. He reasons that, although an increase of flow velocity, or decrease in sediment size, may enhance the capacity of the flow to transport sediment, the rates of excavation and deposition will be increased by similar amounts and the rate of scour unaffected. The principal qualification he makes to this argument is that the mode of sediment movement should not change (that is it should remain either largely bed load or suspended load). Several researchers disagree with this argument but the results of experiments and their interpretation remain inconclusive on this point.

However, while there is still a great deal of uncertainty and controversy regarding scour prediction, it is probably true, that while many bridges have failed due to scour most of these failures have resulted from complete oversight of the scour problem. Thoughtful application of the scour prediction methods already existing and forms of scour prevention and protection should result in a safe bridge - though perhaps at excessive cost.

The U.S. Synthesis of Highway Practice No 5 (26) shows very clearly the confusion that exists in the field of scour prediction. It examines the predictions made by thirteen different formulae at two bridges. The first bridge was an example of clear water scour and the predicted scour depths varied from zero or no scour at all (authors of regime school) to twenty four metres (Laursen and Toch). In the second bridge scour with general sediment motion was anticipated and the scour depth predictions varied from 1,5 metres to 6 metres. These formulae however have all been produced under different conditions and an understanding of these conditions will show some of the limits to their application and perhaps allow judicious selection of the best formula.

A great deal more research work is still needed however before a universal formula can be produced, even before the best formula, or combination of formulae, for a particular site can be confidently selected. There is a lack particularly of results recorded from actual bridge sites which could be compared to those obtained in the various prediction methods. Only in this way can confidence be built up in any method and the problem of scour prediction solved.

CHAPTER 2

The Scour Prediction Formulae

Many method have been developed to estimate the depth of scour that may occur at a particular bridge site under described conditions. The methods have been developed using various approaches and fulfil certain of the conditions that may actually dictate the scour depth that should occur.

This Chapter attempts to describe the approach used in the development of the formula, its method of application and the limitations of that application. For instance certain formulae are developed using laboratory model studies, and may apply only to the local scour occurring under conditions of general sediment motion. Twenty nine different methods by nineteen different researchers are examined in this way. A chart summarising the methods and application limitations is included at the end of this Chapter.

Notation The letter symbols used are defined where they appear, in the illustrations or the text, and for convenience of reference are arranged alphabetically on page 177.

Because many of the references are drawn from U.S.A. literature imperial units (feet, feet/sec, cusecs) are unavoidable in the discussion of the derivation of some of the formulae.

2.1 Laursen

The most thorough and complete research done on scour has been done by Laursen and he has developed several methods of predicting scour. These methods must be applied with care as they are not universal and each method has its own particular application. The methods are set out below and in addition Table 2.1 below has been drawn up to assist the designer in the choice of a method to suit his particular circumstances.

Table 2.1

Method	Application	Type of Scour
I	General Scour	Scour with general sediment motion
II	Local Scour at a Pier	Scour with general sediment motion
III	Local Scour at an Abutment	Scour with general sediment motion
IV	General Scour	Clear-water Scour
V	Local Scour at a Pier	Clear-water Scour
VI	Local Scour at an Abutment	Clear-water Scour

To obtain the total depth of scour the local scour calculated at an abutment or pier must be added to the general scour. The extent of general scour is calculated from the degree of contraction of the flow and if the waterway width does not contract to pass under the bridge there will be no general scour. Total scour will then be equal to local scour.

Method I — To calculate general scour - Scour with general sediment motion

A formula is derived to predict general scour by an analysis of a long contraction.

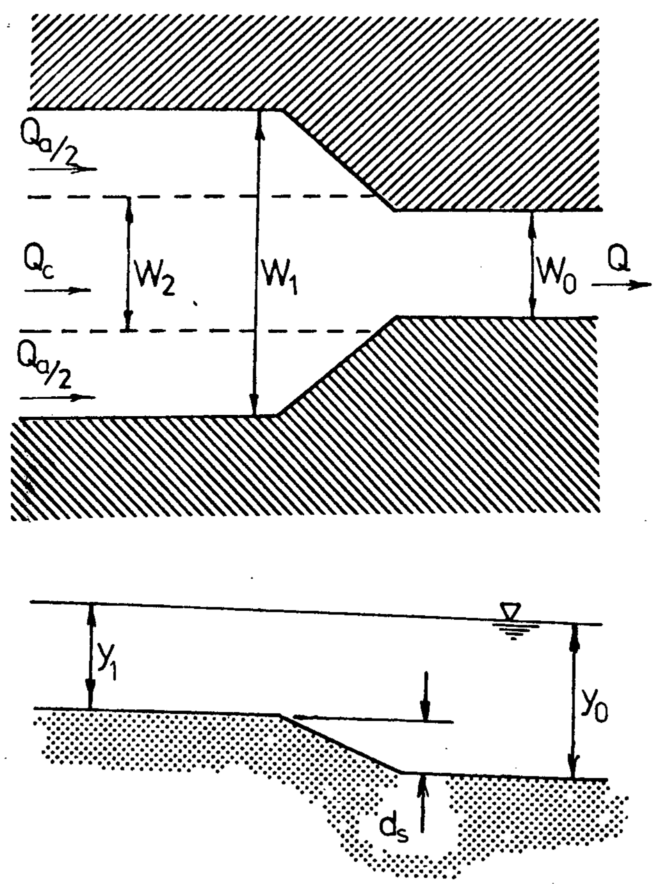


Figure 2.1 Scour in a Long Contraction

Laursen (45) derives an equation to define scour in a long contraction by assuming that the principle of continuity of both water discharge and sediment discharge can be applied to the uncontracted and contracted reaches.

The flow is described by the Manning equation.

In the main channel then

$$Q_c = \frac{1,49}{n_1} W_2 y_1^{\frac{7}{6}} \sqrt{y_1 S} \tag{1}$$

and in the contracted section

$$Q = \frac{1,49}{n_o} W_o y_o^{\frac{7}{6}} \sqrt{y_o S} \tag{2}$$

if the sediment discharge is continuous and the overbank flow carries no sediment then

$$\bar{c}_1 Q_c = \bar{c}_o Q \tag{3}$$

if $\bar{c}_1$ and $\bar{c}_o$ are respectively the mean sediment concentrations at the two sections.

Laursen uses his own formula (from Ref. 42) to describe $\bar{c}$ at any section

$$\bar{c} = \left(\frac{D}{y}\right)^{\frac{7}{6}} \left[\left(\frac{V^2}{120\ y^{\frac{1}{3}} D^{\frac{2}{3}}}\right) - 1\right] k \left(\frac{\sqrt{gyS}}{V_f}\right)^a$$

where

$\bar{c}$ = sediment concentration (bed load plus suspended load) of the material that is scoured, in percent by weight

D = diameter of bed material in feet

y = depth of flow in feet

V = velocity of flow in feet per second

g = acceleration due to gravity in feet per second squared

S = slope in feet per foot

V_f = fall velocity of the sediment in feet per second

The exponent 'a' and coefficient 'k' are dependent on the shear velocity to fall velocity ratio. 'a' varies approximately as follows:

$\frac{\sqrt{gyS}}{V_f}$	a
$< \frac{1}{2}$	$\frac{1}{4}$
1	1
> 2	$\frac{9}{4}$

This relationship is represented graphically in Figure 2.2 below

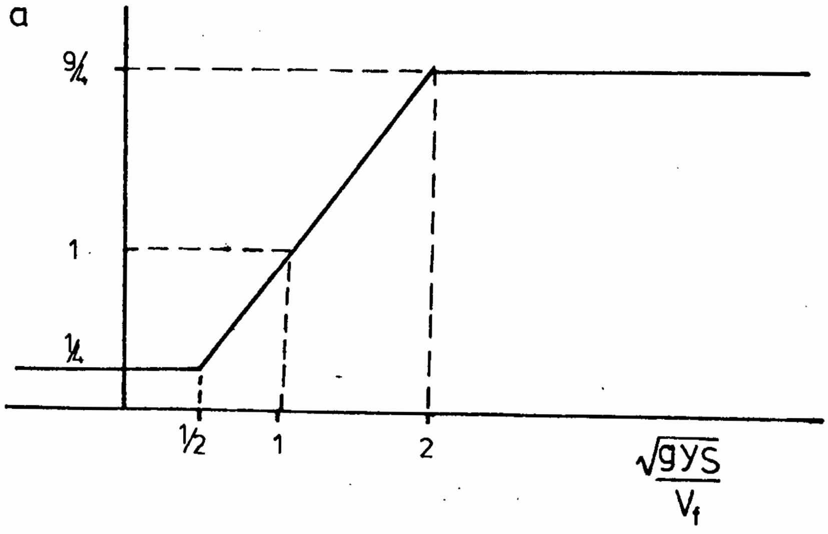


Figure 2.2 Laursen's Exponent 'a' Related to Shear Velocity - Fall Velocity Ratio

Figure 2.2 has been derived from Figure 2.3 below originally presented by Laursen (44).

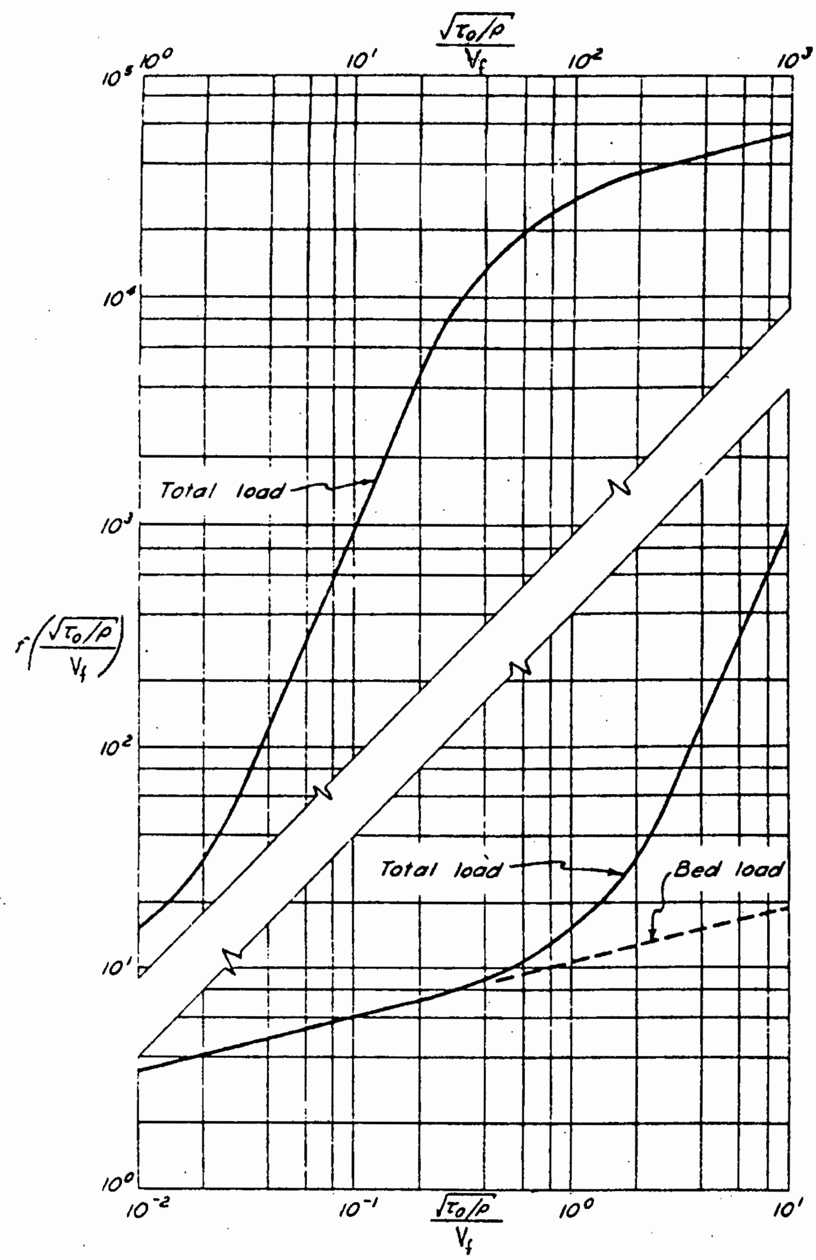


Figure 2.3 Laursen's Relationships for Sediment Load

(The value of shear velocity to fall velocity ratio equal to unity represents the approximate changeover point of the principal mode of sediment transport from bed load to suspended load).

Manning's equation may be used to express shear velocity as

$$\sqrt{gyS} = \frac{Q\sqrt{g}\,n}{1{,}49\,W\,y^{\frac{7}{6}}}$$

Writing out equation (3) in full then

$$\left(\frac{D}{y_1}\right)^{\frac{7}{6}} \left(\frac{Q_c^2}{120\,W_2^2\,y_1^{\frac{7}{3}}\,D^{\frac{2}{3}}}\right) k \left(\frac{Q_c\sqrt{g}\,n_1}{1{,}49\,W_2\cdot y_1^{\frac{7}{6}}\,V_f}\right)^a Q_c$$

$$= \left(\frac{D}{y_o}\right)^{\frac{7}{6}} \left(\frac{Q^2}{120\,W_o^2\,y_o^{\frac{7}{3}}\,D^{\frac{2}{3}}}\right) k \left(\frac{Q\sqrt{g}\,n_o}{1{,}49\,W_o\,y_o^{\frac{7}{6}}\,V_f}\right)^a Q \qquad (4)$$

Laursen then cross multiplies to produce

$$\left(\frac{y_o}{y_1}\right)^{\frac{7}{6}+\frac{7}{3}+\frac{7}{6}a} = \left(\frac{Q}{Q_c}\right)^{3+a}\left(\frac{W_2}{W_o}\right)^{2+a}\left(\frac{n_o}{n_1}\right)^{a}$$

and D, $\sqrt{g}$, 1,49, 120 and k have all disappeared.

From which he arrives at the following equation which is the solution for scour in a long contraction

$$\frac{y_o}{y_1} = \frac{d_s}{y_1} + 1 = \left(\frac{Q}{Q_c}\right)^{\frac{6}{7}}\left(\frac{W_2}{W_o}\right)^{\frac{6}{7}\left(\frac{2+a}{3+a}\right)}\left(\frac{n_o}{n_1}\right)^{\frac{6}{7}\left(\frac{a}{3+a}\right)} \qquad (5)$$

The Manning n-ratio is close to unity and drops out of the equation. If there is no overbank flow then $\frac{Q}{Q_c} = 1$ and the equation further simplifies to

$$\frac{d_s}{y_1} = \left(\frac{W_2}{W_o}\right)^{\frac{6}{7}\left(\frac{2+a}{3+a}\right)} - 1$$

To determine the solution then for scour in a long contraction

(i) Calculate $\frac{\sqrt{gyS}}{V_f}$ ratio

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

y = depth of flow (m)

S = gradient of bed (m/m)

V_f = fall velocity of sediment (m/s)

$$V_f = 1,73\sqrt{Dg(S_s - 1)}$$

D = diameter of bed material (m)

S_s = relative density of sediment particle

(ii) Solve

$$\frac{d_s}{y_1} = \left(\frac{Q}{Q_c}\right)^{\frac{6}{7}} \left(\frac{W_2}{W_o}\right)^{0,59} - 1 \quad \text{for} \quad \frac{\sqrt{gyS}}{V_f} < \frac{1}{2} \qquad (6)$$

$$\frac{d_s}{y_1} = \left(\frac{Q}{Q_c}\right)^{\frac{6}{7}} \left(\frac{W_2}{W_o}\right)^{0,64} - 1 \quad \text{for} \quad \frac{\sqrt{gyS}}{V_f} = 1 \qquad (7)$$

$$\frac{d_s}{y_1} = \left(\frac{Q}{Q_c}\right)^{\frac{6}{7}} \left(\frac{W_2}{W_o}\right)^{0,69} - 1 \quad \text{for} \quad \frac{\sqrt{gyS}}{V_f} > 2 \qquad (8)$$

Comment It is worthy of note that the cross multiplication as carried out above in the derivation of equation (5) is only valid if the exponent 'a' has the same value in the main channel and in the contracted reach.

For the purposes of bridge design the value of $\frac{\sqrt{gyS}}{V_f}$ should be evaluated at each section, and if the exponent 'a' found is unequal in the two sections the appropriate value of 'a' should be used on each side of equation (4).

Method II   <u>To calculate local scour at a pier - Scour with general sediment motion</u>

From a series of laboratory experiments it was found that scour was dependent on pier width and shape, flow depth and angle of attack, and the mode of sediment transport, that is whether it is transported principally as bed load or suspended load. To calculate local scour at a pier the following method may be used:

(i) calculate y_o/w ratio. where w is pier width (m)

(ii) read off d_s'/w ratio from Figure 2.4 below.

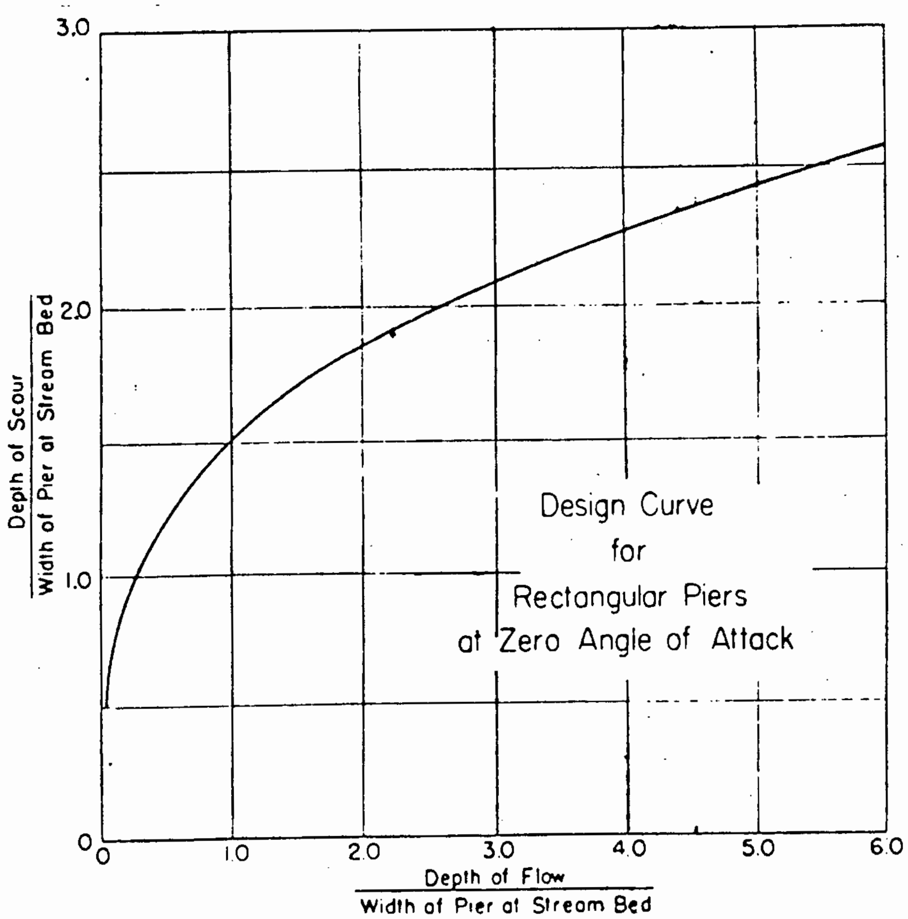


Figure 2.4 Basic Design Curve for Depth of Scour at Piers (Laursen)

(iii) if flow is not aligned with pier determine α, the angle between the direction of flow and the pier axial alignment, and the length to width ratio of the pier (ℓ/w).

(iv) using Figure 2.5 below read off value of K_α coefficient

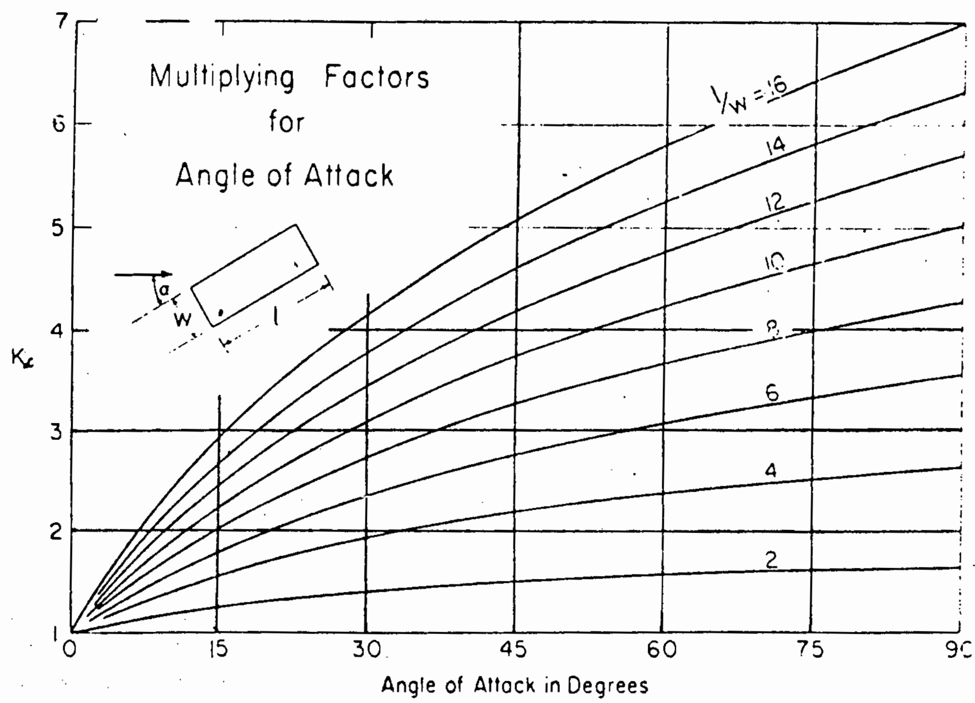


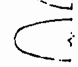
Figure 2.5 Design Factors for Piers Not Aligned with Flow

(v) then scour depth $d_s = K_\alpha \, w \, \frac{d'_s}{w}$

(vi) if α is 0 then apply pier shape correction factor K_s. The factor can be obtained from Table 2.2 below.

The correction factor for pier nose shape K_s is only to be applied when α is 0. Then scour depth $d_s = K_s \, w \, \frac{d'_s}{w}$

Table 2.2 Shape Coefficients K_s for Nose Forms

Nose Form	Length-width ratio	K_s
Rectangular		1,00
Semicircular		0,90
Elliptic	2:1 	0,80
	3:1 	0,75
Lenticular	2:1 	0,80
	3:1 	0,70

(vii) calculate value of shear velocity - fall velocity ratio $\frac{\sqrt{gyS}}{V_f}$ as in Method I step (i) to ascertain the mode of sediment movement.

The scour depth calculated in either (v) or (vi) would be that which would occur if sediment moved primarily as bed load (i.e. $\frac{\sqrt{gyS}}{V_f} < \frac{1}{2}$). If bed material moves as suspended load ($\frac{\sqrt{gyS}}{V_f} \geq 1$) then the depth of scour will increase. The factor K_τ to be used may be obtained from Figure 2.6 below.

Then local scour at a pier $d_s = K_\tau \, K_\alpha \, w \, \frac{d'_s}{w}$ (9)

or $d_s = K_\tau \, K_s \, w \, \frac{d'_s}{w}$ (10)

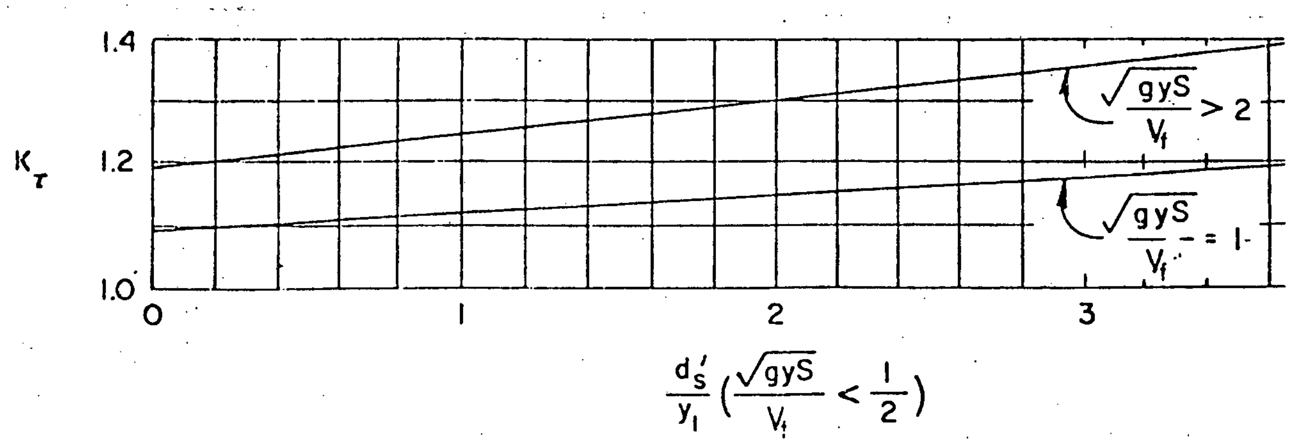


Figure 2.6 Design Factors for Changes in the Mode of Movement

Method III <u>To calculate local scour at an abutment - Scour with general sediment motion</u>

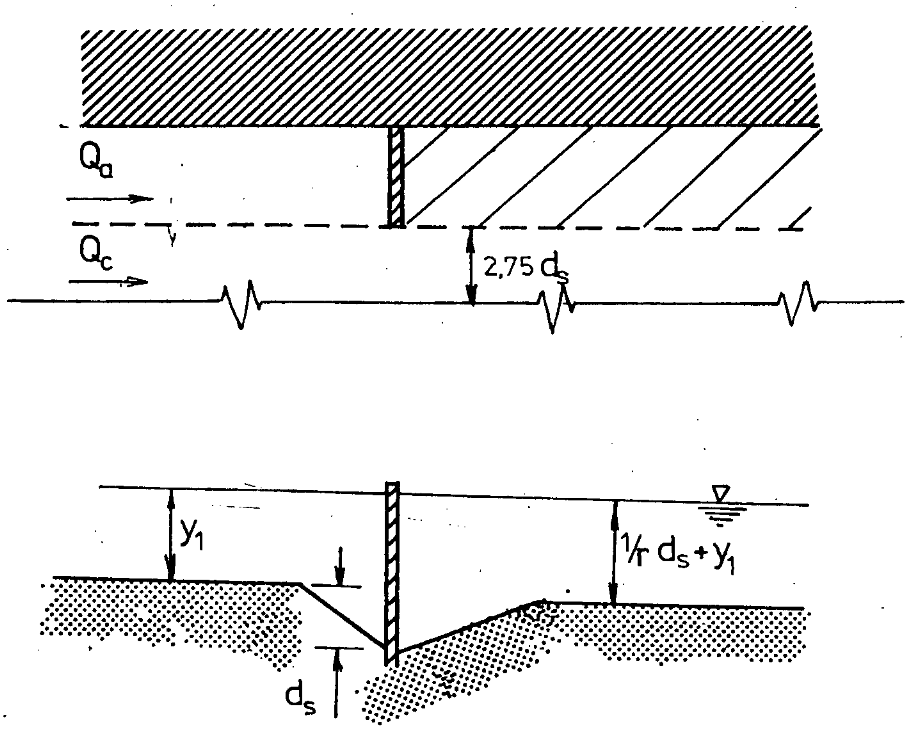


Figure 2.7 Definition Sketch of Abutment Scour

Laursen deduced that scour at abutments was a special case of the type of scour that occurred in a long contraction. The overbank flow must pass round the abutment into the channel. The channel is considered to have a width equal to 2,75 d_s. This width is an average value, taken from experiments, of the width of the scour hole. If the depth of scour in a long contraction is assumed to be a fraction, $\frac{1}{r}$, of the depth of scour at the abutment then the following formula can be applied.

$$\frac{1}{r}\frac{d_s}{y_1} + 1 = \left(\frac{Q_c + Q_a}{Q_c}\right)^{\frac{6}{7}} \qquad (11)$$

The method for solution requires a trial and error approach as the depth of scour is needed to evaluate b, the scour width approximately equal to 2,75 d_s, and Q_b, the flow within the width b.

$$\text{Then } Q_c = \frac{2{,}75 d_s}{b} Q_b \qquad (12)$$

and by algebraic manipulation

$$\frac{Q_a b}{Q_b y_1} = 2{,}75 \frac{d_s}{y_1}\left[\left(\frac{1}{r}\frac{d_s}{y_1} + 1\right)^{\frac{7}{6}} - 1\right] \qquad (13)$$

Figure 2.8 page (22) shows the relationship between $\frac{Q_a b}{Q_b y_1}$ and $\frac{d'_s}{y_1}$ and is the principal graph used in predicting scour depths at abutments. Experimentally r was found to equal 4,1.

To calculate local scour at an abutment the following method may be used.

(i) Establish Q_a, Q_b b and y_1 and calculate $\frac{Q_a b}{Q_b y_1}$ where

Q_a is the overbank flow approaching the abutment.

Q_b is the flow in a width 'b' adjacent to the abutment in the main channel. Value of Q_b will depend on value of b and must be estimated.

b is the width of the scour hole at the bed, measured at right angles to the flow direction. This must be estimated such that

$b = 2{,}75\ d_s$.

y_1 is the upstream flow depth in main channel.

If the flow is uniform, across the waterway width, or assumed to be so, then it is not necessary to estimate b and Q_b. Make $b = W_o$ and $Q_b = Q_c$ and the correct $\frac{b}{Q_b}$ ratio will result.

If flow on one overbank is greater than the flow on the opposite overbank then the scour at each abutment must be calculated separately.

(ii) Using Figure 2.8 below and value of $\frac{Q_a b}{Q_b y_1}$ obtained in (i) above, find value of $\frac{d'_s}{y_1}$.

Figure 2.8 contains two curves and a choice must be made as to which curve is more appropriate. To choose the lower curve it is necessary to have a good knowledge of flow conditions at the bridge, particularly of the extent of cross flow occurring.

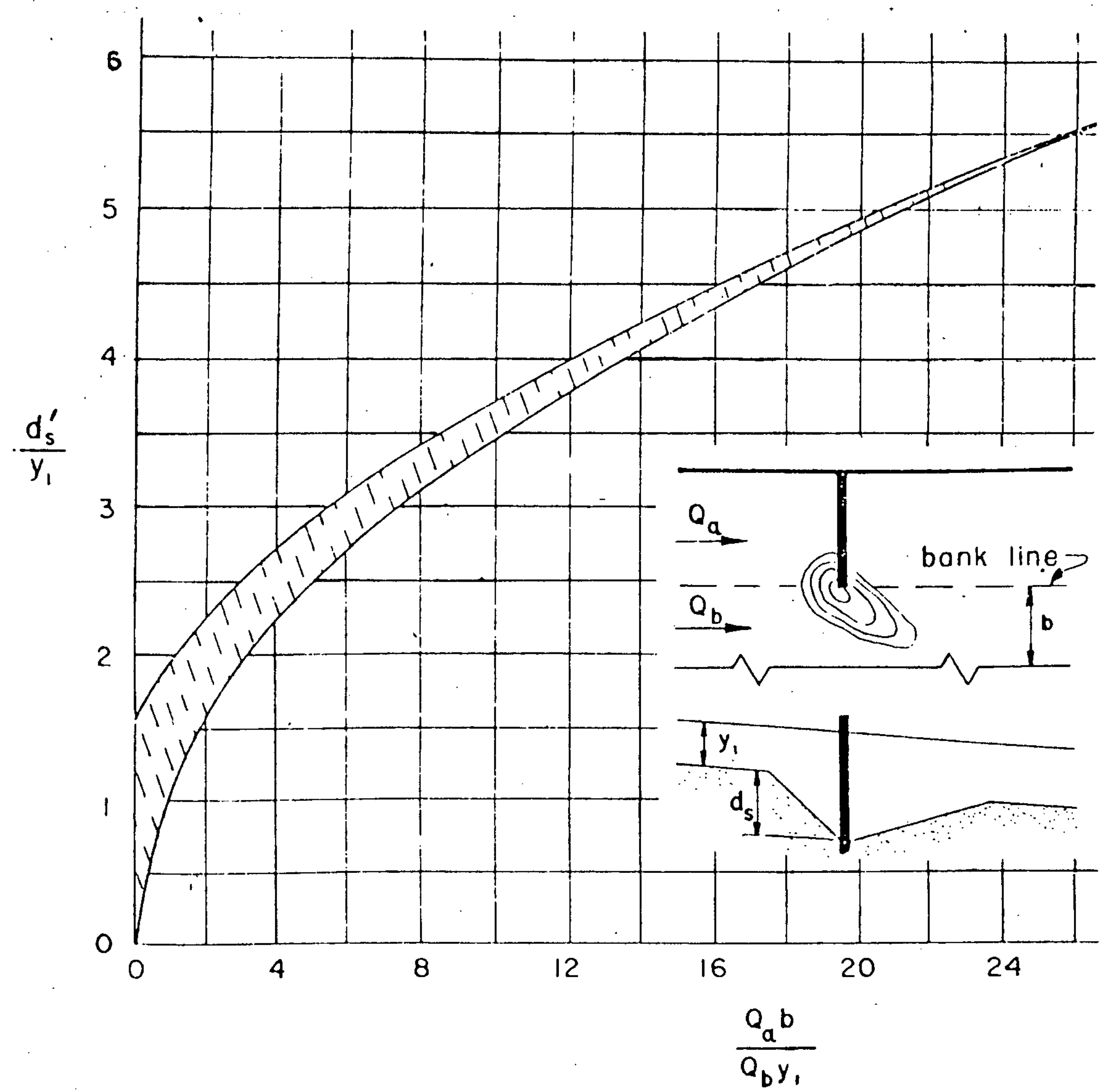


Figure 2.8 Basic Design Curve for an Overbank Bridge Constriction

If overbank flow is small and cross flow is likely to occur well upstream of the bridge, or if cross flow can be accurately determined and will be small, then the lower curve can be chosen. If cross flow is likely to occur and cannot be evaluated then greater scour will occur than would be predicted by the lower curve and the upper curve must be used.

(iii) Then local scour at the abutment will be

$$d_s = y_1 \left(\frac{d'_s}{y_1} \right)$$

(iv) If it was necessary in (i) to estimate b and Q_b the estimation of b should be checked at this stage.

Calculate b_s by $b_s = 2{,}75\ d_s$

If b (estimated) and b_s (calculated) are not approximately equal a new estimate of b and Q_b should be made. The cycle should then be repeated. In practice because there will be only a very slight change in the value of the ratio $\frac{Q_a b}{Q_b y_1}$ the effect of re-estimating b and Q_b on the predicted scour depth will be small.

There are several factors, as yet unconsidered that may alter the predicted scour depth. These are mode of sediment movement, angle of flow attack and type of abutment.

(v) Mode of sediment movement. In the same way that local scour around piers (see Method II stage (vii)) was calculated assuming the mode of sediment movement was primarily bed load so has the scour depth in (iii) above. It is therefore necessary to check if that was the actual condition, and if not to make the necessary alterations to the predicted scour depth.

Calculate value of shear velocity - fall velocity ratio. Then if $\frac{\sqrt{gyS}}{V_f} < \frac{1}{2}$ the mode of sediment movement is primarily as bed load and d_s remains unaltered.

If $\frac{\sqrt{gyS}}{V_f} \geq 1$ the mode of sediment is primarily as suspended load and it is necessary to refer to Figure 2.6 page (19) to obtain factor K_τ.

Then predicted scour depth $d_s = K_\tau \, y_1 \, \frac{d'_s}{y_1}$

(vi) Angle of flow attack. In the case of skewed crossings the abutments will not be normal to the direction of flow and this does affect the predicted scour depth.

Using Figure 2.9 below obtain value of K_θ for each abutment.

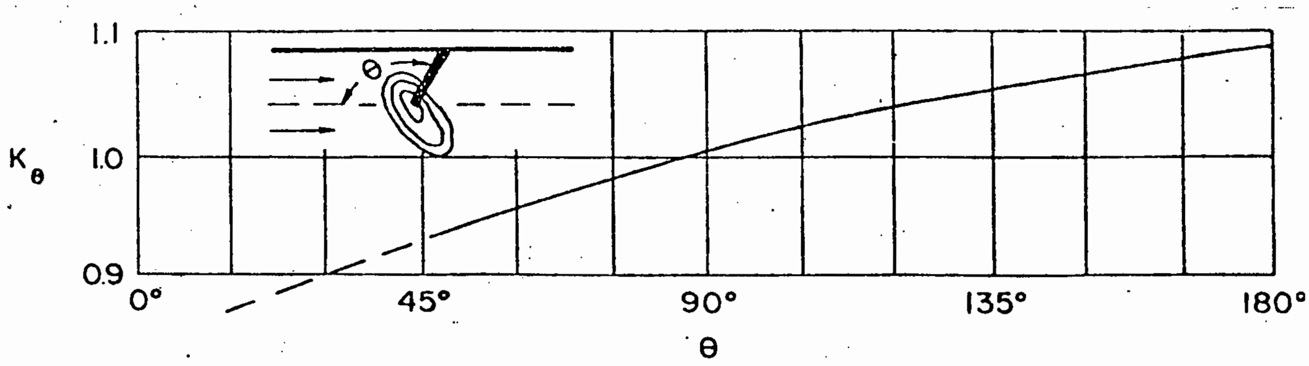


Figure 2.9 Design Factors for the Angle of Incidence on Abutments

(vii) Type of abutment. The shape of the abutment has an effect on scour depth. Figure 2.8 is drawn up assuming a vertical abutment and reduction factors shown in Table 2.3 below can be applied to other types.

Table 2.3 Shape Coefficients for Abutments

Type of Abutment	K_s
Vertical wall	1,00
Wing wall	0,90
Spillthrough	0,80

(viii) Predicted scour depth $d_s = K_\tau \; K_\theta \; K_s \; y_1 \frac{d'_s}{y_1}$ (14)

(ix) The final effect to be considered is that of the scour at the abutment upon the scour at the pier. If there is an overlapping of scour holes the result is cumulative and the scour depth at the pier induced by the scour occurring around the abutment must be added to the local scour that would have occurred at the pier anyway.

If the scour hole breadth ($\simeq 2,75\ d_s$) exceeds the abutment to pier dimension, E, then add to the depth of scour calculated at the pier a further depth equal to $d_s\left(\frac{E - 2,75\ d_s}{E}\right)$. Figure 2.10 can be used to determine the lateral influence of scour holes for abutments that are not constructed at right angles to the flow.

Method IIIa

For abutments that protrude into the main channel the local scour that will occur at the abutments under conditions of general sediment motion must be calculated slightly differently.

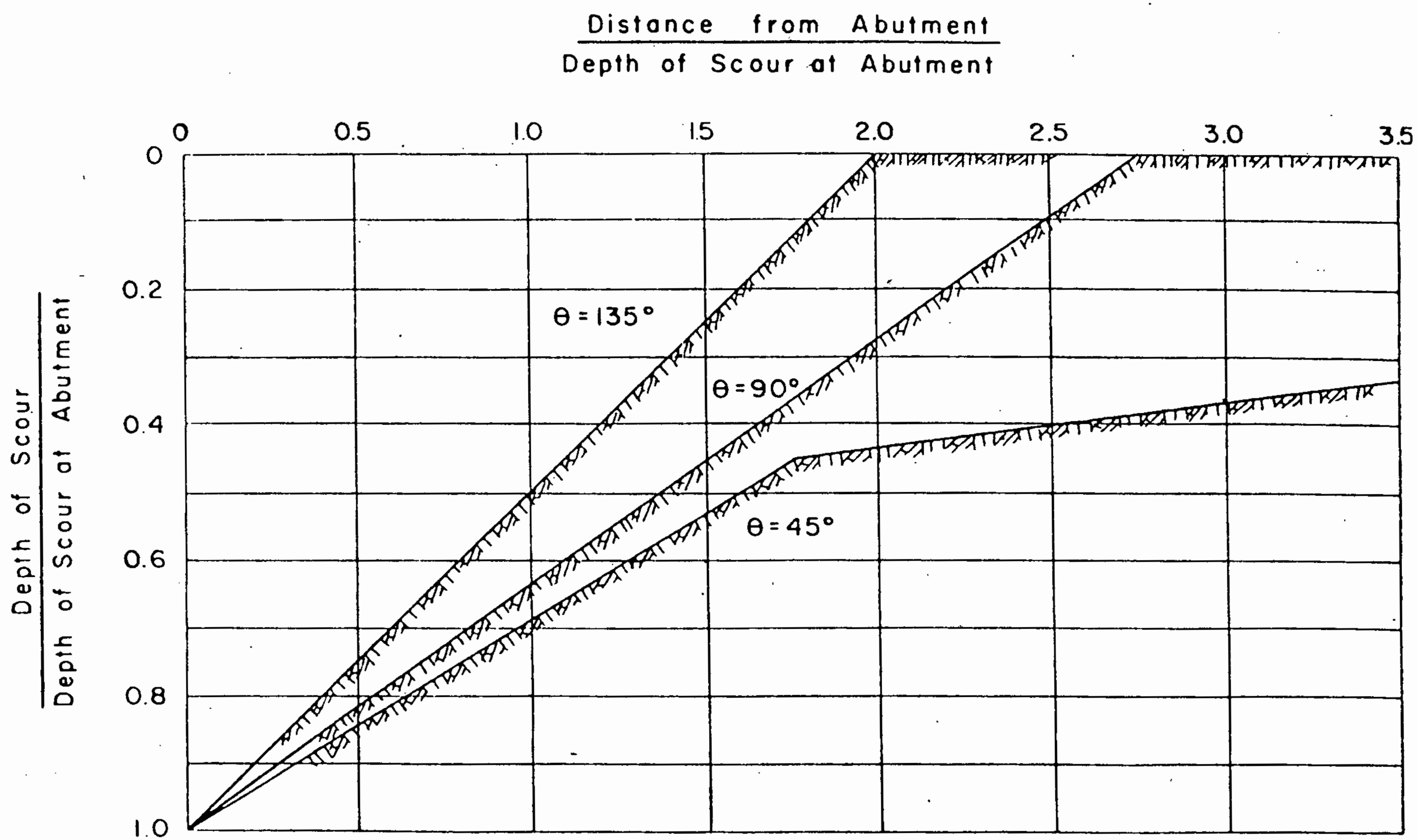


Figure 2.10 Lateral Influence of Scour Holes

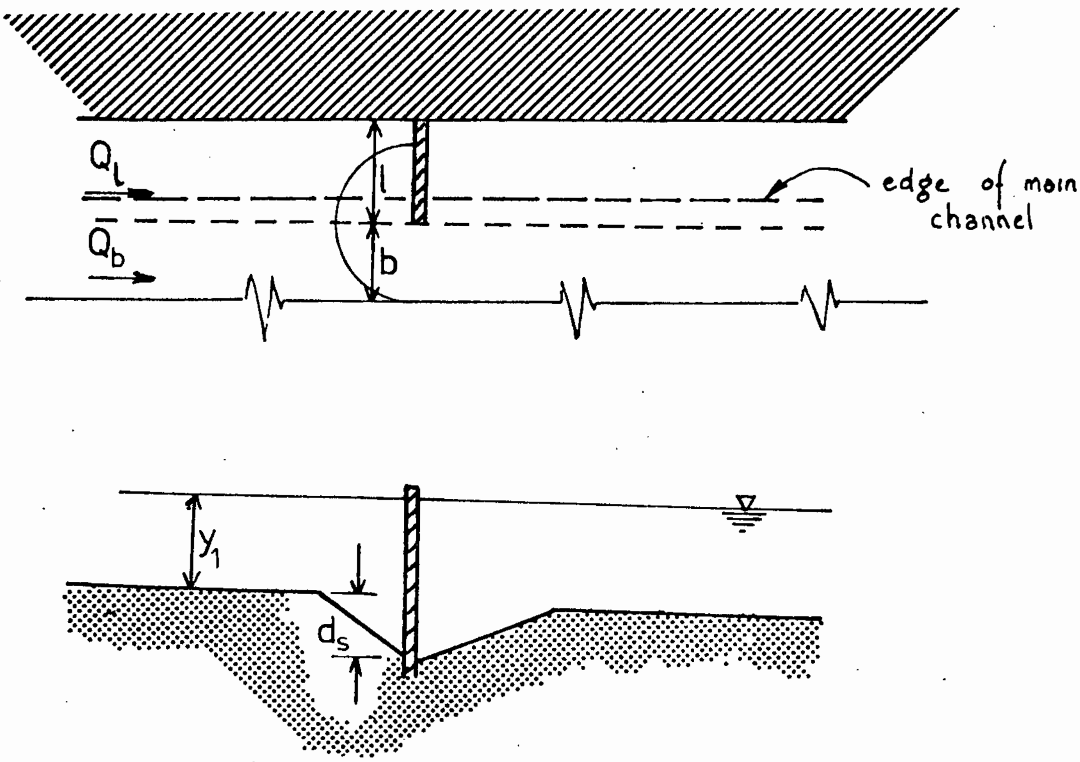


Figure 2.11 Scour at a Protruding Abutment

Figure 2.12 page (28) shows the basic design relationship for scour at an embankment that protrudes into the main river channel.

ℓ_e is the effective length of encroachment and is given by the relationship $\frac{Q_\ell}{\ell_e y_b} = \frac{Q_b}{b y_b}$

where Q_ℓ is the discharge that is obstructed or discharge approaching an encroaching abutment

Q_b discharge over width b. This value must be estimated

b the width of scour. This value must be estimated

y_b average depth over width b at a section upstream of the embankment

The following method may be used to predict scour at an encroaching embankment.

(i) Find value of ratio $\frac{\ell_e}{y_b}$

$$\frac{\ell_e}{y_b} = \frac{Q_\ell b}{Q_b y_b} \tag{15}$$

(ii) From Figure 2.12 below read off value of $\frac{d_s'}{y_b}$ ratio and calculate d_s and b_s

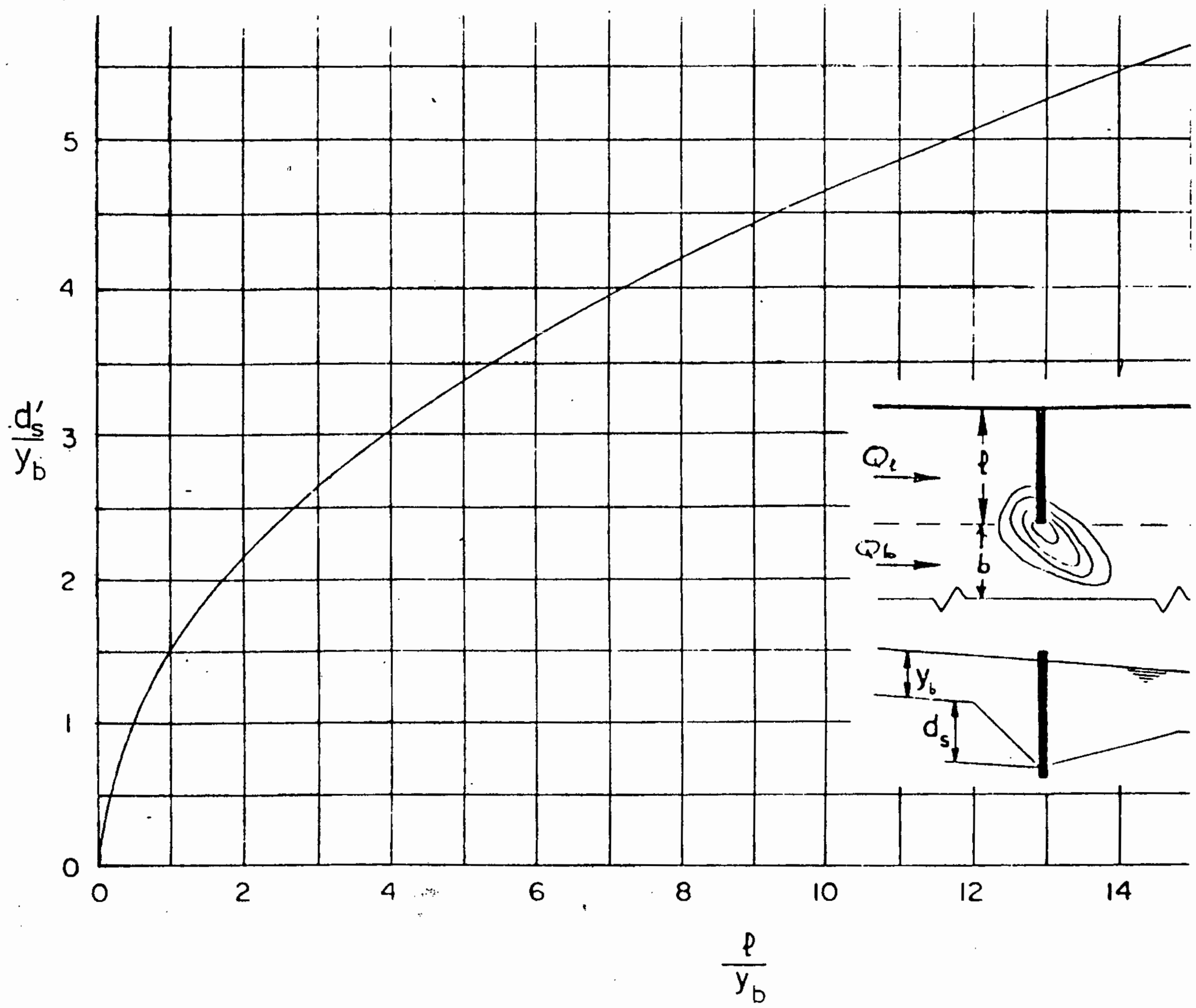


Figure 2.12 Basic Design Curve for Scour at Protruding Abutments

$$d_s = \frac{d'_s}{y_b} y_b$$

$$b_s = 2{,}75\ d_s$$

(iii) Compare b_s (calculated) with b (estimated) - if not approximately equal re-estimate b and Q_b and repeat cycle. If b_s is approximately equal to b, then the corrections to the predicted scour depth to allow for mode of sediment movement, angle of flow attack and abutment shape can be made. These corrections follow the same method as outlined in Method III stages (v), (vi) and (vii).

The following methods (IV, V and VI) of scour prediction are to be used for clear-water scour. This type of scour is likely to occur at a relief bridge where approach velocities are slow and unable to carry sediment, below a dam where there has not been sufficient reach of river to pick up sediment, or at the mouth of a lagoon or lake.

Clear-water scour is liable to be more extensive than scour with general sediment motion. This is because for clear-water scour the scour ceases where the flow has no further capacity to excavate material from the scour hole, but scour with general sediment motion ceases when the rate of excavation equals the rate of deposition of sediment into the scour hole by the flow.

Method IV   To calculate general scour - Clear-water conditions

Again the analysis of the long contraction is used to derive a formula that will predict general scour at a bridge crossing.

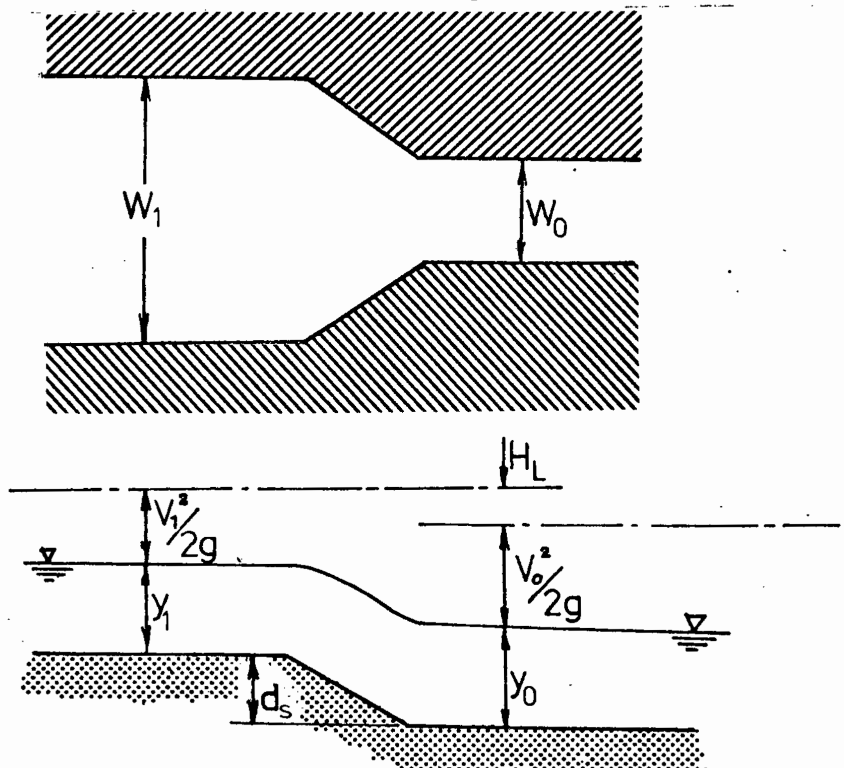


Figure 2.13   Clear-water Scour in a Long Contraction

The flow is continuous therefore

$$Q = V_1 Y_1 W_1 = V_o Y_o W_o \tag{16}$$

and if the head loss is neglected

$$d_s = Y_o - Y_1 + \frac{V_o^2}{2g} - \frac{V_1^2}{2g} \tag{17}$$

or in dimensionless form

$$\frac{d_s}{Y_1} = \frac{Y_o}{Y_1} - 1 + (V_o^2 - V_1^2)\frac{1}{2gY_1} \tag{18}$$

In the contracted section the shear stress on the bed is assumed to be the critical shear stress and is given by

$$\tau_c = 4D = \tau_o \qquad \text{(imperial units)} \tag{19}$$

In the uncontracted section the boundary shear can be obtained from the Manning formula and Stickler's evaluation of n. This gives:

$$\tau_1 = \frac{V_1^2 D^{\frac{1}{3}}}{30\ y_1^{\frac{1}{3}}} \qquad \text{(imperial units)} \tag{20}$$

and the boundary shear in the contracted section can be evaluated similarly

$$\tau_o = \frac{V_o^2 D^{\frac{1}{3}}}{30\ y_o^{\frac{1}{3}}} \qquad \text{(imperial units)} \tag{21}$$

Combining equations (20) and (21)

$$\frac{\tau_1}{\tau_o} = \left(\frac{V_1}{V_o}\right)^2 \left(\frac{y_o}{y_1}\right)^{\frac{1}{3}} \tag{22}$$

and combining equations(19) and (20)

$$\frac{\tau_1}{\tau_o} = \frac{\tau_1}{\tau_o} = \frac{V_1^2}{120\ y_1^{\frac{1}{3}}\ D^{\frac{2}{3}}} \qquad \text{(imperial units)} \tag{23}$$

or $$\left(\frac{y_o}{y_1}\right) = \left(\frac{V_2}{V_1}\right)^6 \left(\frac{\tau_1}{\tau_c}\right)^3 \tag{24}$$

Using equation (16) to eliminate the velocity ratio

$$\frac{y_o}{y_1} = \left(\frac{W_1}{W_o}\right)^{\frac{6}{7}} \left(\frac{\tau_1}{\tau_c}\right)^{\frac{3}{7}} \tag{25}$$

Substituting into equation (18)

$$\frac{d_s}{y_1} = \left(\frac{W_1}{W_o}\right)^{\frac{6}{7}} \left(\frac{\tau_1}{\tau_c}\right)^{\frac{3}{7}} - 1 + \frac{\left(V_o^2 - V_1^2\right)}{2gy_1} \tag{26}$$

and Laursen simplified the equation by neglecting the difference in velocity heads in the two sections. Thus the formula reduces to

$$\frac{d_s}{y_1} = \left(\frac{W_1}{W_o}\right)^{\frac{6}{7}} \left(\frac{\tau_1}{\tau_c}\right)^{\frac{3}{7}} - 1 \tag{27}$$

Figure 2.14 on page (33) presents a graphical solution of this equation. If $\frac{\tau_1}{\tau_c}$ is expressed in metric units the ratio is given by

$$\frac{\tau_1}{\tau_c} = \frac{V_1^2}{36\, D^{\frac{2}{3}} y_1^{\frac{1}{3}}} \tag{28}$$

where V_1 = velocity in uncontracted section (m/s)

D = mean sediment diamter (m)

y_1 = depth in uncontracted section (m)

The following method can be used to estimate general scour under clear-water conditions.

(i) Estimate flood flow Q and associated approach depth y_1

(ii) Calculate ratio $\frac{\tau_1}{\tau_c}$ from equation (28)

(iii) Calculate ratio of uncontracted to contracted waterway widths $\frac{W_1}{W_o}$

(iv) Now the ratio $\frac{d'_s}{y_1}$ can be obtained from Figure 2.14 below and the general scour calculated from $d_s = \frac{d'_s}{y_1} \; y_1$

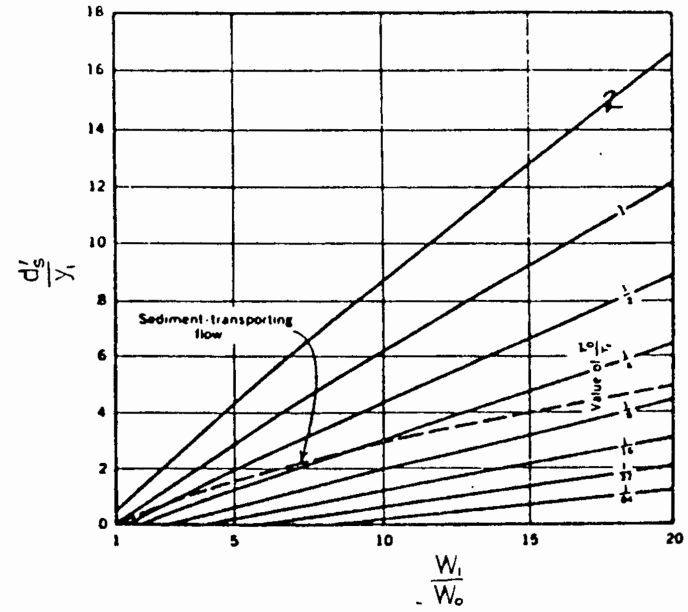


Figure 2.14 Effect of Shear Ratio and Width Ratio on Scour in a Long Contraction

Method V To calculate local scour at a pier - Clear-water scour

The method is an adaptation of the general scour case. A sketch of the model is shown below. The sketch applies also to abutments.

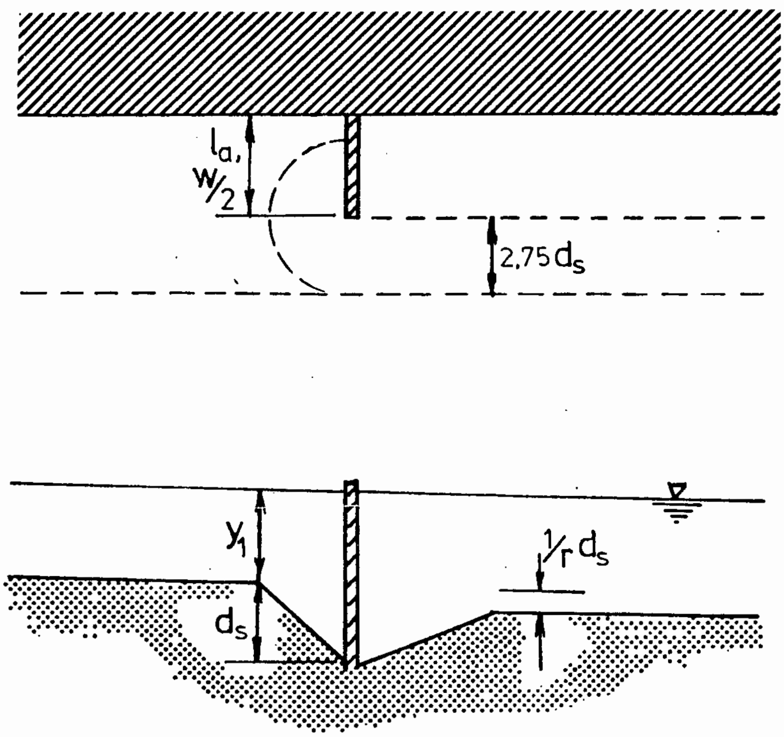


Figure 2.15 Definition Sketch of Clear-water Scour at Piers and Abutments

In the model the width of contraction is taken to be 2,75 d_s and the width of approach flow $\frac{w}{2}$ + 2,75 d_s (for abutments ℓ_a + 2,75 d_s). General scour is assumed to be a fraction $\frac{1}{r}$ of the pier scour.

Then by using equation (27) on page (32)

$$\frac{1}{r}\frac{d_s}{y_1} = \left(\frac{\tau_1}{\tau_c}\right)^{\frac{3}{7}} \left(\frac{\frac{w}{2} + 2{,}75\ d_s}{2{,}75\ d_s}\right)^{\frac{6}{7}} - 1$$

or $$\frac{1}{r}\frac{d_s}{y_1} = \left(\frac{\tau_1}{\tau_c}\right)^{\frac{3}{7}} \left[\frac{\frac{w}{y_1}}{\frac{5{,}5d_s}{y_1}} + 1\right]^{\frac{6}{7}} - 1$$

and $$\frac{w}{y_1} = 5{,}5\ \frac{d_s}{y_1}\left[\frac{\left(\frac{1}{r}\frac{d_s}{y_1} + 1\right)^{\frac{7}{6}}}{\left(\frac{\tau_1}{\tau_c}\right)^{\frac{1}{2}}} - 1\right] \qquad (29)$$

This equation is graphically represented in Figure 2.16 page (36). An empirical value of r = 12 is chosen which is similar to that chosen for scour with general sediment motion. The following method may be used to predict local scour at a pier under clear water scour conditions.

(i) Calculate $\frac{\tau_1}{\tau_c}$ from equation (28)

(ii) Calculate $\frac{y_1}{w}$

(iii) From Figure 2.16 below read off the value of $\frac{d'_s}{w}$ and solve for scour depth $d_s = \frac{d'_s}{w}\ w$

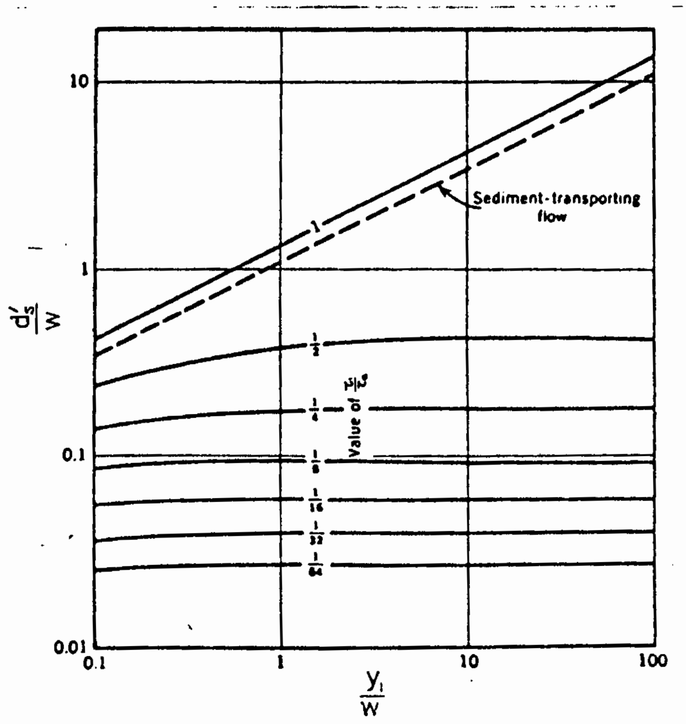


Figure 2.16 Design Curve for Clear-water Scour at a Pier

(iv) Now apply correction factors to adjust the scour depth prediction for the angle of flow attack (Figure 2.5) or pier nose shape (Table 2.2). The method is the same as in Method II stages (iii) to (vi). As scour is under clear-water conditions there will be no adjustment for mode of movement.

As flow direction is more predictable with clear-water scour it will be unusual that the factor for angle of flow attack need be used. The two factors, for flow attack angle and pier nose shape are never used in conjunction. Predicted pier scour depth then is either

$$d_s = K_s \, w \, \frac{d_s'}{w} \tag{30}$$

$$d_s = K_\alpha \, w \, \frac{d_s'}{w} \tag{31}$$

Method VI To calculate local scour at an abutment - Clear-water scour

The derivation of equation (32) below is similar to that used for the pier scour equation (29). The equation derived is

$$\frac{\ell_a}{y_1} = 2{,}75\,\frac{d_s}{y_1}\left[\frac{\left(\frac{1}{r}\frac{d_s}{y_1}+1\right)^{\frac{7}{6}}}{\left(\frac{\tau_1}{\tau_c}\right)^{\frac{1}{2}}} - 1\right] \qquad (32)$$

This equation is graphically represented in Figure 2.17 page (38). Again a value for r is empirically chosen as 12.

Briefly a method that may be used for clear-water abutment scour is as follows

(i) Calculate $\frac{\tau_1}{\tau_c}$ from equation (28)

(ii) Calculate $\frac{\ell_a}{y_o}$

ℓ_a is the nominal length of the abutment and is the width required for the flow approaching the embankment at the nominal velocity and depth of flow on the flood plain.

(iii) By using Figure 2.17 below find $\frac{d'_s}{y_1}$. Then solve for scour by $d_s = \frac{d'_s}{y_1}\, y_1$.

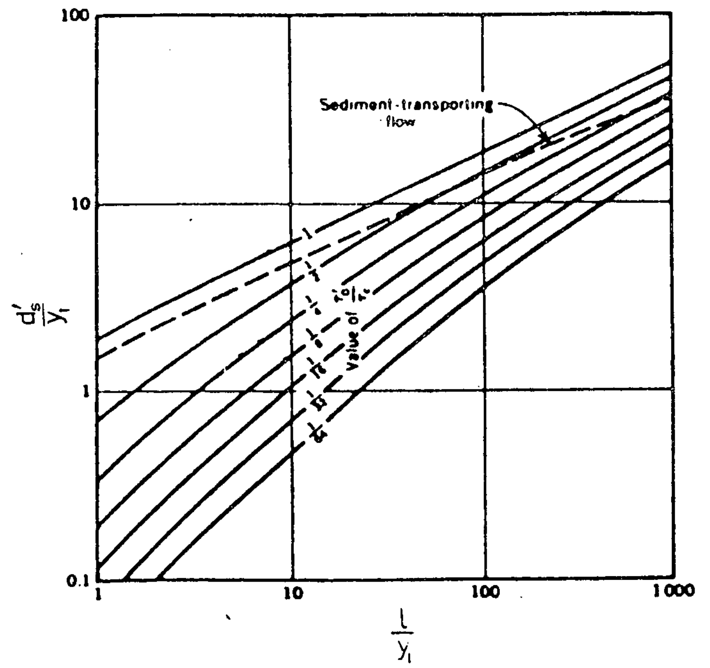


Figure 2.17 Design Curve for Clear-water Scour at an Abutment

(iv) The effect of abutments being skew to the flow and abutment shape can be calculated as in Method II stages (vi) and (vii). The predicted scour depth then is

$$d_s = \frac{d'_s}{y_1} y_1 K_s K_\theta \tag{33}$$

<u>Comment</u> Laursen has produced by far the most complete analysis of the scour problem. No other method of scour prediction has the means of analysis of so many of the variables that may influence the depth of scour. In all, the Laursen method, if it may be so called, considers whether scour is general or local, clear-water scour or scour with general sediment motion, at an abutment or at a pier, and also allows for such effects as pier nose shape, type of abutment, angle of flow attack and mode of sediment movement. The method, with the possible

exception of the determination of proportions of overbank and channel flow, is also simple to apply.

It must be stressed however that the method is derived from a laboratory and, as with all the other methods, has not been verified in the field with the exception of the Skunk River Bridge experiments (42). Several researchers are opposed to the idea that velocity and sediment size do not affect scour depth for scour with general sediment motion. Intuitively they appear to be right and Laursen wrong. However, by imagining a series of experiments in which the sediment size is increased, all other factors remaining constant, and the scour measured for each sediment size it is apparent that for very large particles there will be no scour at all. It is necessary to appreciate however that at this stage scour will be clear-water scour where Laursen admits the importance of sediment size - perhaps a case of intuition leading one astray.

Garde, Subramanya and Nambudripad (21) disagree with the factor 2,75 used by Laursen to relate scour depth to scour hole breadth. They state that in experiments conducted by them values varied from 1,8 to 5,0. Laursen does state however that 2,75 was an average value.

It will be noted that Methods I and IV predict general scour, and Methods II, III, V and VI local scour. Total scour is the sum of general and local scour. In particular cases, the scour at a bridge may be given by the sum of total scour and general scour. For example, consider two adjacent bridges on the same stream, both causing flow contraction. The scour at the first bridge is merely the total scour. The scour at the downstream bridge however is the sum of the general scour caused by the upstream bridge and its own general and local scour effects.

2.2 Holmes

Holmes analysed thirty-six railway bridges in New Zealand whose foundations had undergone scour failure, and from his analysis developed a formula that would have predicted, with reasonable accuracy, the depth of scour attained at each of these sites. More than half (60%) of these bridges were founded on either concrete or timber piles with 35% on footings. In many cases the depth of scour that led to failure had not been measured but had to be estimated at the time of the analysis.

The formula is unusual in that it is the only one derived in the field from actual scour failures. All other formulae derived in the field are based on a general study of river characteristics.

The Figure below describes some of the terms used in his formula.

The formula is

$$y_s = y_{sg} + y_{s\ell} \qquad (34)$$

where y_s = total scoured depth (m)

y_{sg} = scoured depth due to general scour (m)

$y_{s\ell}$ = local scour (m)

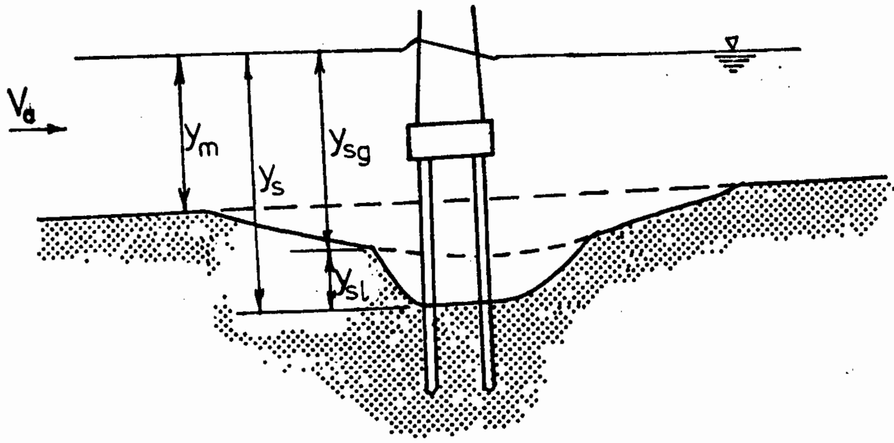


Figure 2.18 Definition Sketch for Holmes Formula

The scoured depth due to general scour can be calculated by the formula

$$y_{sg} = y_r \, V_a \, K \left(\frac{W_o}{A} \right)^{\frac{1}{2}} \tag{35}$$

where y_r = the flood rise: this dimension is measured immediately upstream of the bridge and is taken from normal low water stage (or ordinary low tide level is appropriate) to peak flood level (m)

A = the waterway area. This is the area of occupied waterway measured normal to the flow at the bridge and measured assuming an unscoured channel profile. The water surface level is that which would be achieved immediately upstream of the bridge and usually no deduction is made for the projected area of the piers (m^2)

W_o = waterway width: In his analysis Holmes used the total waterway width at the bridge. For design purposes he recommends that the value used here should be such that $W_o = 1{,}25\ W_{80}$ where W_{80} is that width of waterway carrying 80% of the flow

$$K = \left[\frac{W_o}{4{,}83\ Q^{\frac{1}{2}}} \right]^{\frac{1}{2}} \qquad (36)$$

If K as calculated by equation (36) exceeds 1,0 then it is made equal to 1,0.

Q = flow at peak flood m^3/s

V_a = the approach velocity (m/s). This term is calculated from derived values for mean flow velocity, mean depth and y_m which is defined below. V_a is given by the following formula

$$V_a = V_1 \left(\frac{Y_m}{Y_1} \right)^{\frac{2}{3}} \qquad (37)$$

At bridge sites downstream of two converging flows or with braided or meandering rivers V_a must be increased by 20%

y_m = flood depth. A dry weather bed profile is assumed. The flood depth is measured from upstream flood level down to the lowest bed level which is likely to occur at the pier being considered. If residual scour holes (either local or general) tend to add to the depth of y_m, the increase in depth is discounted (m)

y_1 = the mean depth of flow upstream of the bridge (m)

V_1 = the mean flow velocity upstream of the bridge (m/s)

These terms are illustrated in Figure 2.19 below.

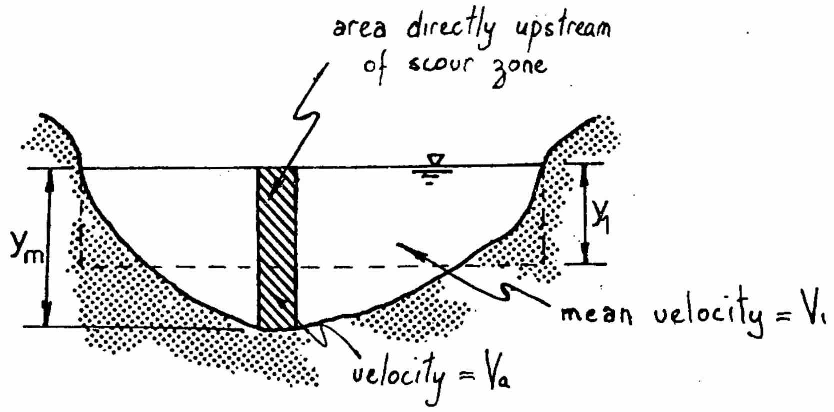


Figure 2.19 Definition Sketch for Holmes Velocity Equation

The local scour can be calculated using the equation

$$y_{s\ell} = 0{,}8 \sqrt{V_a w} \qquad (38)$$

where w = pier width in metres

Where the direction of flow attack is not aligned with the pier axis Holmes suggests that the pier width be increased by the factors given by Laursen which can be obtained by using Figure 2.3 on page (12).

This formula for local scour is an approximation of the one proposed by Shen.

<u>Comment</u> The method presented here provides a quick and simple method of predicting scour and the fact that it has been formulated from actual failures gives it a certain credibility. A comparison of the predicted depths and actual depths on the thirty-six bridges analysed reveals an accuracy of prediction in the range - 20% and + 30% with 70% of the results lying within the range of ± 15%. These figures apply to total scoured depth however. Similar figures for depth of scour would show a decrease of accuracy. Prediction by this method is likely to be conservative. Four of the predictions were dead accurate, 11 underestimated the scour and 21 overestimated it.

The average slope at the sites considered was 0,46% (approx. 1 in 200) and it may be that the formula is particularly applicable to steep fast flowing rivers. There was no obvious trend relating slope and accuracy of prediction though.

It may be inferred that the formula has been worked out from cases where scour has occurred under conditions of general sediment motion which is the more usual type of scour. Two interesting points arise from this. The first is that sediment size is not included in the formula which lends support to Laursen's theory that sediment size does not affect the equilibrium depth of scour under general sediment motion conditions. The second is that the local scour as calculated by the formula $y_{s\ell} = 0{,}8 \sqrt{V_a w}$ which is a simplification of the formula proposed by Shen (which will be discussed later) ($d_s = 0{,}000223 R_e^{0{,}619}$ metres) the same conditions for its application that Shen stipulates are not applied - namely that it is for clear-water scour and so called blunt piers.
By not making these two stipulations the formula is being conservative, and may lead to slight overestimation of the predicted scour depth.

Regime Theory

Several methods of predicting scour which are all similar and related have arisen out of the Regime Theory. Before dealing with the formulae developed by Lacey, Inglis, Blench and others it is necessary to say something about this theory.

The Regime Theory arose from the mass of data collected by Indian-English engineers from the irrigation canals in Northern India and Pakistan which provided the 'laboratory of regime science'. In 1919 the concept of the Regime Theory was stated by E.S. Lindley - 'When an artificial channel is used to convey silty water, both bed and banks scour or fill, changing depth, gradient and width, until a state of balance is attained at which the channel is said to be in regime. These regime dimensions depend on discharge, quantity and nature of bermsilt, and rugosity of the silted section; rugosity is also affected by velocity which determines the size of wavelets into which the silted bed is thrown'.

This concept has been formulated, principally by Lacey and Blench, into the following form:

$$\begin{aligned} \text{mean width} &\propto Q^{\frac{1}{2}} \\ \text{depth} &\propto Q^{\frac{1}{3}} \\ \text{slope} &\propto Q^{\frac{1}{6}} \end{aligned}$$

Lacey and Blench differ on the factor of proportionality and where Lacey used a 'silt factor' Blench preferred to use both a 'bed factor' and a 'side factor'.

The value to be chosen for Q, the discharge, is straightforward for irrigation canals but for rivers presents some difficulty. It is generally taken to be bank-full discharge which on most rivers can be defined within close limits. Another choice has been the discharge of the five year flood which is often similar to the bank-full discharge. However for use in scour problems the value used is the peak flood discharge for which the foundations are to be designed.

When applied to scour prediction the regime method predicts local scour effects, if they do so at all, by assuming that local scour will be some proportion of the general scour. This method has obvious limitations.

2.3 Lacey

Lacey proposed the equation

$$y_L = 0{,}47\ (Q/f)^{\frac{1}{3}} \tag{39}$$

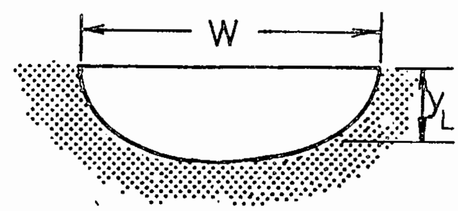


Figure 2.20 Lacey Regime Channel

where y_L = Lacey mean depth equal to cross-sectional area divided by surface width (m)

and f = Lacey silt factor

Q = design flow which in scour calculations will be the flood discharge (m^3/s)

The Lacey silt factor can be calculated from the equation $f = 50\sqrt{D}$ where D is the sediment diameter in metres. In a paper in 1930 (35) Lacey gave a Table showing the variation of silt factor with sediment size. The Table was revised in 1934 (36) and the revised version with sediment size shown in mm is shown in Table 2.4 below. This Table shows the silt factor assigned to various grades of silt by different authorities, and calculated values of the average silt diameter from Lacey's formula for comparison with the description of the bed.

Table 2.4 The Lacey Silt Factor

f	D mm	
24,78	243,8	Large boulders, shingle, sand
21,60	185,4	Large boulders, shingle heavy sand
16,20	104,1	Boulders and gravel
14,05	78,7	Boulders and gravel
11,97	57,7	Boulders and gravel
10,33	42,7	Boulders and gravel
10,10	40,1	Medium boulders, shingle and sand
10,00	39,6	Gravel and coarse bajri
8,67	30,0	Gravel and coarse bajri
6,33	15,9	Small boulders, shingle and sand
4,84	9,30	Large pebbles and coarse gravel
2,98	3,56	Coarse bajri and sand
2,58	2,64	Coarse bajri and sand
2,06	1,70	Heavy sand
1,91	1,45	Fine bajri and sand
1,65	1,09	Fine bajri and sand
1,56	0,27	Coarse sand
1,44	0,81	Coarse sand
1,35	0,74	Medium sand
1,00	0,39	Standard Kennedy sand
0,41	0,066	Fine Egyptian silt (200 mesh approx)

It is apparent from Figure 2.20 above that the Lacey depth or Lacey mean depth is not the maximum depth of the stream. To determine the maximum depth Lacey gave the following factors that give the approximate relationship of mean to maximum depth.

Factor

1,273 straight stable channel (based on elliptical section)

1,5 moderate bend

1,75 severe bend

2,0 right angle bend (based on a triangular section)

Lacey did not predict scour at bridges but equation (39) may be combined with his equation for waterway width

$$W = 4{,}83\ Q^{\frac{1}{2}} \tag{40}$$

where W = waterway width (m)

Q = discharge (m^3/s)

to give $$\frac{y_{Lo}}{y_{L1}} = \left(\frac{W_1}{W_o}\right)^{\frac{2}{3}} \tag{41}$$

if W_o is an imposed width - or the width in the contracted section then W_1 is width in uncontracted section and y_{Lo} and y_{L1} are the Lacey mean depths in the contracted and uncontracted sections respectively.

Then using equations (39), (40) and (41) at a section of imposed width the Lacey mean depth is given by

$$y_L = \frac{1{,}34\ q^{\frac{2}{3}}}{f^{\frac{1}{3}}} \tag{42}$$

where $q = \frac{Q}{W_o}$ the discharge intensity in $m^3/s/m$

W_o is the imposed width or waterway surface width at the bridge in metres. Expressed another way

$$y_s = 1{,}273.1{,}34 \left(\frac{Q}{W_o}\right)^{\frac{2}{3}} f^{-\frac{1}{3}} \quad \text{metres} \tag{43}$$

assuming the constriction or bridge is in a straight reach of the river.

This formula does not predict total scour as there is no component for local scour in the equation. It is a simple method of assessing general scour however.

2.4 Inglis

Inglis measured scour at various sites and by comparing these measured depths with the Lacey depth concluded that the maximum total scoured depth around bridge piers was twice the Lacey depth, or $y_S = 2\ y_L$ and y_L is given in equation (39).

This formula suffers from several obvious limitations. Firstly, there is no constriction effect implicit in the formula. Secondly, although the formula predicts total scoured depth no allowance is made for pier size or angle of inclination to flow which both have real relevance.

2.5 Blench

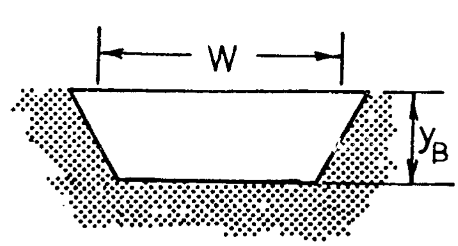


Figure 2.21 Blench Regime Channel

Blench adopted a slightly different approach to that chosen by Lacey when he used a trapezoidal section as his typical section where the defined width is the mean of bed and surface widths or the width at half depth. The depth is the depth across the rectangular section - that is the cross-sectional area divided by width. Blench also used both a 'bed factor' and a 'side factor' instead of the single 'silt factor' used by Lacey. For scour prediction, Blench also introduced a factor called the 'zero-bed factor', F_{bo}, which is a function of the bed sediment properties only. He gave the following formula to predict total scoured depth.

$$y_s \quad = \quad 1,48\ K_B\ q^{\frac{2}{3}}\ F_{bo}^{-\frac{1}{3}} \tag{44}$$

where q = discharge intensity equivalent to design flow divided by the width at half depth ($m^3/s/m$)

K_B = a factor to convert zero flood depth to maximum total scoured depth. The factor is to be used as follows:

at noses of spurs and guide banks	2,0 to 2,75
where flow impinges at right angles on bank	2,25
between and around bridge piers	2,0

F_{bo} = the zero bed factor

A graph showing the variation of F_{bo} with size of bed material was published in 1964 by Blench and Qureshi (9) and is shown here in Figure 2.22page (51). It was published together with a description of the empirical nature in which it was drawn up and a warning from the authors 'lest it may acquire a spurious scientific dignity to which it does not pretend'. In addition the Figure does not apply when the depth is less than thirty times the median sediment diameter (i.e. $y_o/D < 30$).

The method only applies to sand and larger materials and no values for F_{bo} with bed material smaller than 0,1 mm are given. For particles 0,1 to 0,3 F_{bo} can be calculated by the formula $F_{bo} = 1,9\sqrt{D}$, where D is in mm, or taken from Figure 2.22 below.

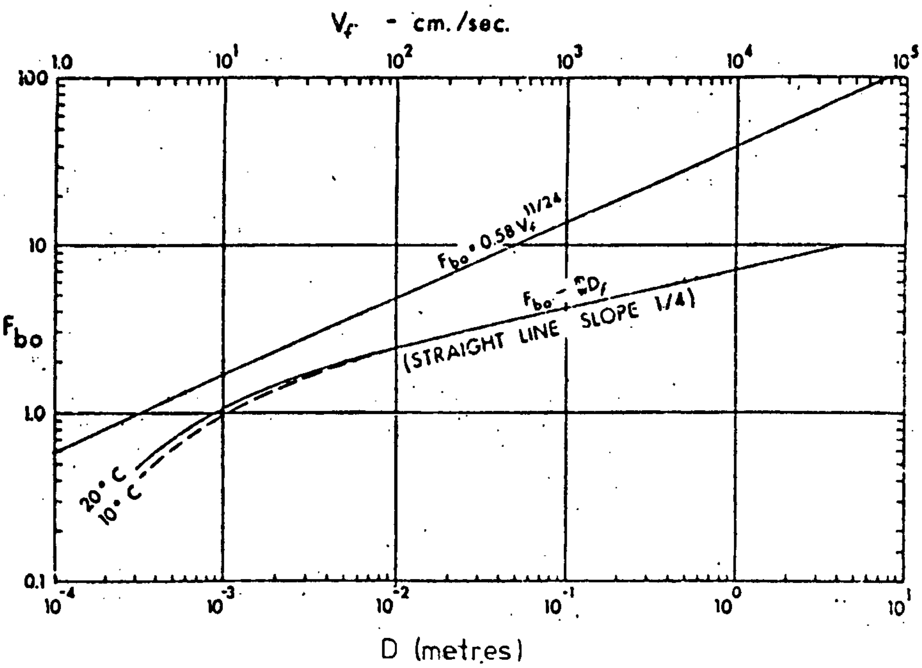


Figure 2.22 Chart Estimating Zero Bed-factor

<u>Comment</u> This formula is an improvement on the one proposed by Inglis in that it does take account of constriction effects by considering discharge intensity rather than simply discharge. However the effects of pier size and shape are again ignored. Blench states though that the values given for K_B 'assume that the river can take on any physically possible angle of attack'.

There is a stipulation of major importance. The formula is to be used 'to obtain suitably designed scoured depths for designing aprons, but not to obtain the scoured depths that may occur without the aprons'. In other words Blench requires the provision of rip-rap.

2.6 Ahmad

From a study of scour at spur dikes and laboratory models of bridges Ahmad proposed that scour can be calculated by the formula below

$$y_s = 1{,}48\ K_A\ q^{\frac{2}{3}} \tag{45}$$

where q is the discharge intensity of the design flood or

$q = \frac{Q}{W}$ in $m^3/s/m$

W is the unobstructed surface waterway width at the bridge (m)

K_A a multiplying factor to be selected as shown in Table 2.5 below. The factor is dependent on the approach condition

Table 2.5 Ahmad's Coefficient

Approach Condition	$V_{\frac{1}{2}}/V_1$	K_A
Scour below a severe bend on the concave side accompanied by a swirl on the convex bend	1,25	2,0 - 2,25
Moderate bends	1,15	1,5 - 1,75
Straight obstruction placed at any angle of 30^o to 90^o to the flow	1,0	1,2 - 1,5
Straight obstruction placed at any angle of 90^o to 150^o to the flow	1,0	1,5 - 1,75

$V_{\frac{1}{2}}/V_1$ is the ratio of mean velocity in half the channel width on the side of the abutment or spur dike to the mean velocity in the full section.

This Table was originally drawn up for calculation of scour at spur dikes. In the model tests completed on bridges K_A varied from 1,1 to 2,0 but in every model but one training works, consisting of a spur upstream of the bridge, were used.

Ahmad recommends that a value of K_A from 1,7 to 2,0 be used and in addition it is necessary to 'provide proper guide bank to keep the road or railway approaches safe from embayment and to keep abnormal scour from the main bridge crossing'.

Though intended for spur dikes the formula might be applied to predict general scour in the type of arrangement shown below in Figure 2.23.

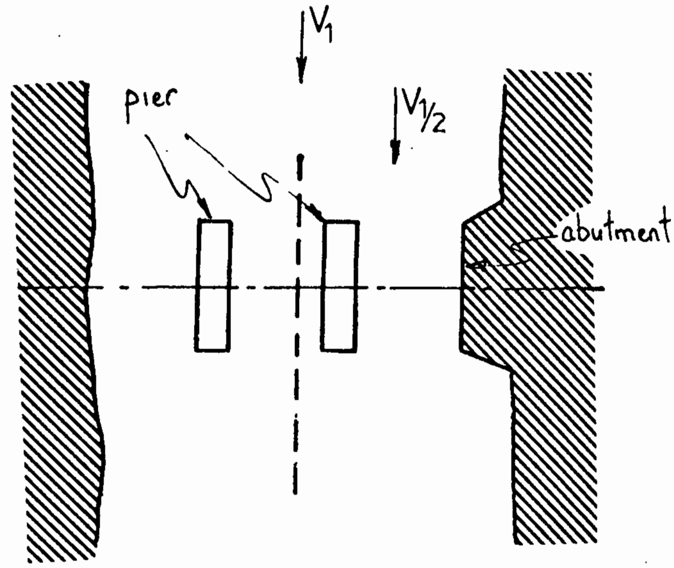


Figure 2.23 Definition Sketch for Ahmad Formula

<u>Comment on Regime Formulae</u> The regime formulae are all similar in that they predict total scoured depth and are all in the form $y_s = y_o + d_s = \text{factor } q^{\frac{2}{3}}$. It is interesting to note that this result is not very different to the one proposed by Laursen.

$$\frac{d_s}{y_1} = \left(\frac{W_1}{W_o}\right)^{0,64} - 1 \quad \text{for} \quad \frac{\sqrt{g y_o S}}{V_f} = 1$$

$$\frac{d_s}{y_1} = \left(\frac{W_1}{W_o}\right)^{0,69} - 1 \quad \text{for} \quad \frac{\sqrt{g y_o S}}{V_f} > 2$$

On average the relation can be written

$$\frac{d_s + y_1}{y_1} = \frac{y_o}{y_1} = \left(\frac{Q/W_o}{Q/W_1}\right)^{0,64-0,69} \approx \left(\frac{q_o}{q_1}\right)^{\frac{2}{3}}$$

and $$\frac{y_o}{q^{\frac{2}{3}}} = \frac{y_1}{q^{\frac{2}{3}}} = K$$

which is very similar to the result obtained by the regime authors.

It is probable that the regime formula do give an accurate prediction of the average scoured depth and the assumption that the maximum scoured depth at a section is some multiple of that average depth appears reasonable where applied to bends or natural river processes. However applying a multiplying factor to establish the scour depth at piers is in conflict with the results established in model experiments where it appears that local scour should be in addition to, rather than a multiple of, the general scour.

The reliability of any of the multiplying factors is uncertain firstly as they are not related to any pier size, shape or alignment and secondly as their derivation appears to be partly from field cases where the difficulty of measuring scour is well known.

To use the regime formulae it is necessary to have data concerning the stage-flood discharge relationship as the formulae define total scoured depth which is measured from the water surface.

2.7 Herbich and Brennan

Another set of formulae, similar to the regime formulae in that they are primarily useful in predicting average scour, were developed in Canada by Herbich and Brennan. They did a thorough study of thirteen bridge sites in Southwestern Ontario and at each site took measurements which included size and grading of bed material, flow velocities and river cross-sections at high and low flows and the discharge. From the cross-section the waterway area, surface width and maximum depth could be obtained.

The formulae were derived principally using dimensional analysis and gave good correlation between predicted general scour and the actual scour measured at the bridge sites.

Scour was found to be a function of the ratios

$$\frac{V_s}{V_f}, \quad \frac{\tau}{\tau_c} \quad \text{and} \quad \frac{V_o}{V_c}$$

where

V_s = the shear or friction velocity

V_f = the fall velocity of the sediment

τ = the average shear stress

τ_c = the critical shear stress

V_o = the average velocity

V_c = the critical velocity

These terms will be more fully explained later.

The formulae vary only in the influence they assign to these ratios. The results of each of these formulae were found to correlate well with the actual scour measured at the thirteen bridge sites. The formulae are:

(1) $$d_s = 0{,}267 + 0{,}57 \left(\frac{V_s}{V_f} \right) \tag{46}$$

(2) $$d_s = 0{,}271 + 0{,}034 \left(\frac{\tau}{\tau_c} \bigg/ \frac{V_o}{V_c} \right) \tag{47}$$

(3) $$d_s = 0{,}332 - 0{,}342 \left(\frac{V_s}{V_f} \right) + 0{,}046 \left(\frac{\tau}{\tau_c} \bigg/ \frac{V_o}{V_c} \right) \tag{48}$$

(4) $$d_s = 0{,}35 - 0{,}17 \left(\frac{V_s}{V_f}\right) + 0{,}027 \left(\frac{\tau}{\tau_c} \Big/ \frac{V_o}{V_c}\right) \qquad (49)$$

This formula is to be applied only to shallower, less turbulent flows (depth < 3 m (10 ft) Reynolds number < 45.10^5)

(5) $$d_s = 0{,}43 - 0{,}44 \left(\frac{V_s}{V_f}\right) + 0{,}047 \left(\frac{\tau}{\tau_c} \Big/ \frac{V_o}{V_c}\right) \qquad (50)$$

This formula is to be applied to deeper, more turbulent flows (depth > 3 m (10 ft) Reynolds number > 45.10^5)

To apply these formulae it is necessary first to be able to calculate the various parameters.

τ - The Average Shear Stress or Average Tractive Force

Herbich and Brennan used the formula:

$$\tau = \rho_w \left(\frac{V_4 - V_3}{5{,}75 \log \frac{y_3}{y_4}}\right)^2 \qquad \text{(imperial units)} \qquad (51)$$

where V_3, V_4 are the velocities measured at depths y_3, y_4

ρ_w is the mass density of the water

For their investigation many velocity measurements were made at different depths and Herbich and Brennan felt that they would obtain a more accurate result by using equation (51) than by using the equation proposed by P.Dubois

$$\tau = \gamma_w \, yS \qquad (52)$$

γ_w = specific weight of water (9,81 kN/m^3)

y = depth to bed (m)

S = slope (m/m)

Another empirical formula is also given:

$$\tau = 0{,}0021\ \rho\ V^2\ R_H^{-\frac{1}{3}} \qquad \text{(imperial units)} \qquad (53)$$

τ_c - Critical Tractive Force or Critical Shear Stress

The critical shear stress is that stress that will cause a particle to move. The value of the stress is dependent on the sediment properties of specific gravity and size. Several researchers have developed formulae for calculating critical shear stress and these include the following:

White (1936) $\tau_c = 0{,}18\ (\gamma_s - \gamma_w)\ D \tan\phi$ (54)

Shields (1936) $\tau_c = 0{,}06\ (\gamma_s - \gamma_w)\ D$ (55)

Tison (1953) $\tau_c = 0{,}3$ to $0{,}7\ (\gamma_s - \gamma_w)\ D \tan\phi$ (56)

and the coefficient varies with Reynolds number

D (Equations (54) to (56)) is the diameter of the particle at 50 percent by weight, in metres.

Stebbings (1963)

$$\tau_c = \frac{mg}{a} \tan\phi\ 10^{-3} \qquad (57)$$

m = mass of sand particle (kg)

a = area of sand particle over which stress acts (m^2)

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

ϕ (equations (54), (56) and (57)) is the natural angle of repose of material under water.

Lane and Carlson (1953)

$$\tau_c = 0{,}075\ D_{25} \tag{58}$$

D_{25} is the diameter of the particle at 25 percent by weight (in metres)

Any of the above formulae will give a good approximation of the critical shear stress.

V_o - The Average Velocity at a section at the Bridge

This is simply $\frac{\text{flood flow}}{\text{waterway area}}$. The waterway area is calculated at the bridge with the projected pier area deducted. For the purposes of their analysis Herbich and Brennan measured this quantity. For design purposes the velocity can be tentatively calculated using the Manning formula.

Several formulae exist which can be used to find Kutter's 'n' which appears in the Manning formula.

Strickler formula

$$n = \frac{(D)^{\frac{1}{6}}}{24{,}07} \tag{59}$$

Keulegan formula

$$n = \frac{(D)^{\frac{1}{6}}}{25{,}66} \tag{60}$$

Irmay formula

$$n = \frac{(D_{10})^{\frac{1}{6}}}{26{,}8} \tag{61}$$

D_{10} is the diameter of particle at 10 percent by weight (in metres)

Lane and Carlson

$$n = \frac{(D_{25})^{\frac{1}{6}}}{21,34} \tag{62}$$

D_{25} is the diameter of particle at 25 percent by weight (in metres)

Lane and Carlson

$$n = \frac{(D_{35})^{\frac{1}{6}}}{R_H \; 46,6} \tag{63}$$

D_{35} is the diameter of particle at 35 percent by weight (in metres)

R_H is the hydraulic radius in metres

Straub

$$n = 0,0526 \; (D)^{\frac{1}{6}} \tag{64}$$

In equations (59), (60) and (64) D is the median sediment diameter (in metres).

These formulae will lead to underestimation if applied to areas with vegetational cover.

The following Table (reproduced from 150 1070 - 1973(E) Liquid Flow Measurement in Open Channels - Slope Area Method) is given to assist in the selection of the correct n value.

Table 2.6 The Manning Coefficient

Coefficients for channels with relatively coarse bed material and not characterized by bed formations

Type of bed material	Size of bed material (mm)	Manning's coefficient n
Gravel	4 to 8	0,019 to 0,020
	8 to 20	0,020 to 0,022
	20 to 60	0,022 to 0,027
Pebbles and shingle	60 to 110	0,027 to 0,030
	110 to 250	0,030 to 0,035

Coefficients for channels other than those with coarse bed material

Type of channel and description	Manning's coefficient n
EXCAVATED OR DREDGED	
a) Earth, straight and uniform	
Clean, recently completed	0,016 to 0,020
Clean, after weathering	0,018 to 0,025
With short grass, few weeds	0,022 to 0,033
b) Rock cuts	
Smooth and uniform	0,025 to 0,040
Jagged and irregular	0,035 to 0,050

Type of channel and description	Manning's coefficient n
NATURAL STREAMS	
Minor streams (top width at flood stage less than 30 m (100 ft))	
a) Streams on plains	
Clean, straight, full stage no rifts or deep pools	0,025 to 0,033
Flood plains	
a) Pasture, no brush	
Short grass	0,025 to 0,035
High grass	0,030 to 0,050
b) Cultivated areas	
No crop	0,020 to 0,040
Mature row crops	0,025 to 0,045
Mature field crops	0,030 to 0,050
c) Brush	
Scattered brush, heavy weeds	0,035 to 0,070
Light brush & trees - without foliage	0,035 to 0,060
Light brush & trees - with foliage	0,040 to 0,080
Medium to dense brush, without foliage	0,045 to 0,110
Medium to dense brush, with foliage	0,070 to 0,160
d) Trees	
Cleared land with tree stumps no sprouts	0,030 to 0,050
Same as above, but with heavy growth of sprouts	0,050 to 0,080
Heavy stand of timber, a few felled trees, little undergrowth, flood-stage below branches	0,080 to 0,120
Same as above, but with flood-stage reaching branches	0,100 to 0,160
Dense willows, in mid-summer	0,110 to 0,200

If a stage discharge relationship has been drawn up for the site the level of the design flood can be ascertained whence waterway area and velocity can be derived.

V_c - <u>The Critical Velocity</u>

Several formula have been developed to predict the flow velocity necessary to cause a particle on a river bed to move. The one used by Herbich and Brennan was derived by Straub for the dredge fill closure of the Missouri River at Fort Randall. It was based on his own formula for n (equation (64)).

$$V_c = 4{,}67\sqrt{\frac{\gamma_s - \gamma_w}{\gamma_w}}\left(\frac{y_o}{D}\right)^{\frac{1}{6}}\sqrt{D} \qquad (65 \qquad (65)$$

where V_c is in m/s

D is the median sediment diameter in metres.

V_s - <u>Shear Velocity</u>

$$\text{Shear or friction velocity } V_s = \sqrt{\frac{\tau}{\rho_w}} \qquad (66)$$

For particles < 10 mm diameter and Reynolds number > 100 (in terms of shear velocity and diameter of particle $R_e = \frac{V_s D}{\nu}$). Liu found that the bed shear velocity was constant.

$$V_s = 0{,}13\ V_f \qquad (67)$$

Hallmark and Smith showed that

$$V_f = 0{,}227\ D^{\frac{1}{2}} \tag{68}$$

with V_f in m/s and D, the median sediment diameter, in mm

By substitution $$V_s = 2{,}94\ D^{\frac{1}{2}} \text{ (for } D > 10 \text{ mm)} \tag{69}$$

V_f - <u>Fall Velocity</u>

The terminal velocity of a particle, for unhindered settling in water can be described by the formula

$$V_f = \sqrt{\frac{4}{3} \frac{gD}{C_D} \frac{\rho_s - \rho_w}{\rho_w}}$$

There are three regions of flow; laminar, transitional and turbulent which are designated by the Reynolds number of the particle. Under conditions of scour flow is likely to be turbulent ($R_e > 1000$) and then the coefficient of drag $C_D = 0{,}44$ and the above equation reduces to

$$V_f = 1{,}73\sqrt{Dg\ (S_s - 1)} \tag{70}$$

V_f = fall velocity in m/s

D = median sediment diameter in m

S_s = relative density of sediment ($= \frac{\rho_s}{\rho_w}$)

A more simple formula has been produced by Hallmark and Smith which Herbich and Brennan found gave good correlation with equation (70). The equation assumes a shape factor of 1 and is

$$V_f = 0{,}227\ D^{\frac{1}{2}} \quad \text{(with D in mm and } V_f \text{ in m/s)}$$

Comment The five formula given in this method can only be used to calculate general scour. It is surprising that in a study as thorough as this one that the effects of piers should be ignored. It is interesting to note though that on the five bridges that have piers (out of the thirteen bridges where measurements were taken) that the formulae lead to underestimation of the average scour. For these five bridges the formula underestimated in 16 cases out of 20 (80%) whereas the average over the thirteen bridges was 54% (or 28 underestimations from 52 estimates). This shows that local scour probably did occur at these piers.

It is not made clear whether scour at these sites occurred under conditions of general sediment motion or clear water. In nine cases the average velocity was greater than the critical velocity, so it seems likely that for the majority of cases scour occurred under general sediment motion conditions. There is therefore disagreement with the Laursen theory in that effects of both velocity and sediment size are considered important.

Measurements were made at the bridges at times of both high and low flow. The high flow did not correspond to floods of the size normally associated with washaways and the average scour depth of the bridges was not great (average 0,6 m, maximum 1,4 m). River gradients were not steep and the flow velocities in flood not high (Average 1,1 m/s, maximum 2,4 m/s). It is possible that the accuracy of the formula may be affected at higher flow velocities.

2.8 Komura

One of the most recent formulae to be developed for scour in a constriction was produced by Komura in 1966. The theoretical layout is identical to that used by Laursen (Figure 2.13) and is shown again below as Figure 2.24.

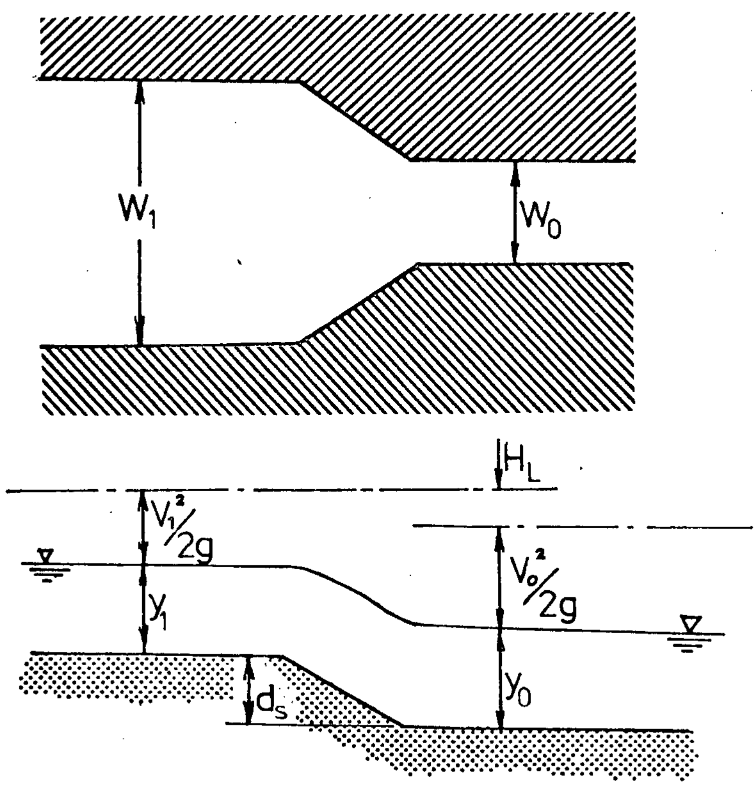


Figure 2.24 Definition Sketch of Scour in a Long Constriction

Formulae were derived for both the dynamic equilibrium state and the static equilibrium state. These states respectively correspond to scour with general sediment motion and clear water scour.

The analysis of the dynamic equilibrium state makes use of the assumptions that both flow and rate of sediment transport are continuous in the unconstricted and constricted sections. The analysis yields the equation

$$\frac{d_s}{y_1} = \left(\frac{\tau_1}{\tau_o}\right)^{\frac{2}{7}} \left(\frac{W_1}{W_o}\right)^{\frac{6}{7}} - 1 \tag{71}$$

where y_1 is the water depth in the unconstricted section and subscripts 1 and O refer to the unconstricted and constricted sections respectively

τ is shear stress on the bed, subscripts refer to the section being considered

W is the surface waterway width, and subscripts refer to the section being considered

Equilibrium depth of scour in the static equilibrium state is derived by assuming that when the scour is fully developed the boundary shear stress in both the unconstricted and constricted sections is equal to the critical tractive force. The scour can be expressed by

$$\frac{d_s}{y_1} = \left(\frac{\tau_{cl}}{\tau_{co}}\right)^{\frac{2}{7}} \left(\frac{W_1}{W_o}\right)^{\frac{6}{7}} - 1 \tag{72}$$

where τ_c is the critical tractive force.

In applying the formula the following method may be used.

(i) Check whether constriction is long or short. A long constriction is defined as one in which the length exceeds or equals the breadth or $\frac{L}{W_o} \geq 1$

If the constriction is short the example is outside the range of the experimental data and the formula may not be used with confidence.

(ii) Check whether equilibrium state is dynamic or static.

If $V_{cl} << V_{sl}$ the equilibrium state will be the dynamic equilibrium state and solution is given by equation (71).

If $V_{cl} \simeq V_{sl}$ the equilibrium state will be the static equilibrium state. Solve by using equation (72).

Methods of determining the values of the critical and shear velocities have been described in the section pertaining to the formulae by Herbich and Brennan.

Komura however uses Yuichi Iwagaki's formula on critical tractive force to find the critical velocity.

$$\frac{\tau_c}{\rho_w} = a_c \, (S_s - 1) \, gD \tag{73}$$

where τ_c = critical tractive force and a_c = a constant

Iwagaki showed that a_c is a function of the critical shear velocity Reynolds number, $\frac{V_c D}{\nu}$, where V_c is the critical velocity. The exact relationship between a_c and V_c is not given however. Similarly Komura does not specify a preferred method of finding τ_c.

To find the shear velocity Komura uses the equation

$$\frac{V_1}{V_{sl}} = 7{,}66 \left(\frac{R_H}{k_s} \right)^{\frac{1}{6}} \tag{74}$$

R_H is the hydraulic radius and k_s the equivalent roughness which is expressed in the function

$$\frac{k_s}{D} = K \left[\frac{V_s^2}{(S_s - 1)\ gD} \right]^m \tag{75}$$

where K = a constant

m = a dimensionless exponent pertaining to the tractive force, whose value depends on the bed configurations but is taken as unity in the constriction problems

S_s = the relative density of the sediment

D = the mean diameter of the bed material

No value is given for the constant K. Komura does state though that 'When $m = 0$, $k_s = KD = D_{65}$ is generally used, in which D_{65} = the particle size of bed material for which 65%, by weight, is finer'.

(iii) If $V_{cl} << V_{sl}$ then solve using equation (71)

$$\frac{d_s}{y_1} = \left(\frac{\tau_1}{\tau_o}\right)^{\frac{2}{7}} \left(\frac{W_1}{W_o}\right)^{\frac{6}{7}} - 1$$

In this formula strictly τ is the shear stress on the bed but the ratio of the shear stresses for the whole channel is approximately equal to the ratio of shear stresses for the bed so either stress may be used.

To obtain the value of the ratio $\frac{\tau_1}{\tau_o}$ Komura uses the greater of the two values obtained from the formulae

$$\left(\frac{\tau_1}{\tau_o}\right) = \left(\frac{W_1}{W_o}\right)^{-\frac{2}{3}} \tag{76}$$

and

$$\frac{\tau_1}{\tau_o} = C_d^{\frac{7}{2}} \; F_1^{\frac{7}{10}} \left(\frac{W_1}{W_o}\right)^{-\frac{2}{3}} \; \sigma_{\phi 1}^{-\frac{7}{4}} \tag{77}$$

where $C_d = 1{,}45 \quad (C_d^{\frac{7}{2}} = 3{,}67)$

F_1 = Froude number in unconstricted region $\left(= \frac{V_1}{\sqrt{gy_1}} \right)$

$\sigma\phi_1$ = the standard deviation of the particle size distribution in the unconstricted region

$$\left(\sigma_\phi = \sqrt{\frac{D_{84}}{D_{16}}}\right)$$

(iv) If $V_{c1} \simeq V_{s1}$ then solve using equation (72)

$$\frac{d_s}{y_1} = \left(\frac{\tau_{c1}}{\tau_{co}}\right)^{\frac{2}{7}} \left(\frac{W_1}{W_o}\right)^{\frac{6}{7}} - 1$$

To find the value of $\frac{\tau_{c1}}{\tau_{co}}$ Komura gives

$$\frac{\tau_{c1}}{\tau_{co}} = C_s^{\frac{7}{2}} \; F_1^{\frac{7}{10}} \left(\frac{W_1}{W_o}\right)^{-\frac{2}{3}} \; \sigma_{\phi 1}^{-\frac{7}{4}} \tag{78}$$

with notation as for equation (77) and $C_s^{\frac{7}{2}} = 5{,}18$.

<u>Comment</u> It is interesting to note the similarity of the equation for static equilibrium of scour to that proposed by Laursen for clear-water scour in equation (16)

$$\frac{d_s}{y_1} = \left(\frac{\tau_1}{\tau_c}\right)^{\frac{3}{7}} \left(\frac{W_1}{W_o}\right)^{\frac{6}{7}} - 1$$

Though similar there is a significant difference in the logic of the derivation in that Laursen assumes that once scour has fully developed the boundary stress in the constricted section only is equal to the critical tractive force. Komura states that in both constricted and unconstricted sections the shear stress has become equal to the critical tractive force.

The equations given are only intended to predict general scour in a long constriction. No local effects due to piers or abutments are considered.

Shen recommends that these formula be used in conjunction with his own formulae that predict local scour to obtain total scour at any site.

Using dimensional analysis Komura developed two further formulae. These are

For dynamic equilibrium depth of scour

$$\frac{d_s}{y_1} = C_d \; F_1^{\frac{1}{5}} \left(\frac{W_1}{W_o}\right)^{\frac{2}{3}} \sigma_{\phi 1}^{-\frac{1}{5}} - 1 \qquad (79)$$

From Komura's experimental data $C_d = 1{,}45$. From Ashida's experimental data, where the constricted region used in the experiments was short $C_d = 1{,}22$. Komura states 'More data are needed to determine the constant C_d'.

For static equilibrium depth of scour

$$\frac{d_s}{y_1} = C_s \; F_1^{\frac{1}{5}} \left(\frac{W_1}{W_o} \right)^{\frac{2}{3}} \sigma_{\phi 1}^{-\frac{1}{2}} - 1 \qquad (80)$$

C_s was found to equal 1,60 in the experimental range investigated.

2.9 Das

From laboratory model experiments and a theoretical consideration of the shear stress on the stream bed, Das proposed the following equation to predict the scour depth at end dump channel constrictions, which might be considered comparable to a spur or bridge abutment

$$1{,}44 \; \frac{(y_1 + d_s)}{y_1} = F_1^{0,85} \left(\frac{y_1}{D} \right)^{0,29} \left(\frac{\rho_w}{\rho_s - \rho_w} \right)^{0,43} 10^{0,43(j_1\beta + j_2)} \qquad (81)$$

y_1 = approach depth in metres

F_1 = upstream Froude number = $V_1/\sqrt{gy_1}$

D = representative size of bed material = median diameter for sizes up to 2 mm, in millimetres

ρ_w = density of water

ρ_s = density of bed material

β = the constriction ratio = W_o/W_1

j_1, j_2 = exponents

Figure 2.25 below shows the configuration being considered.

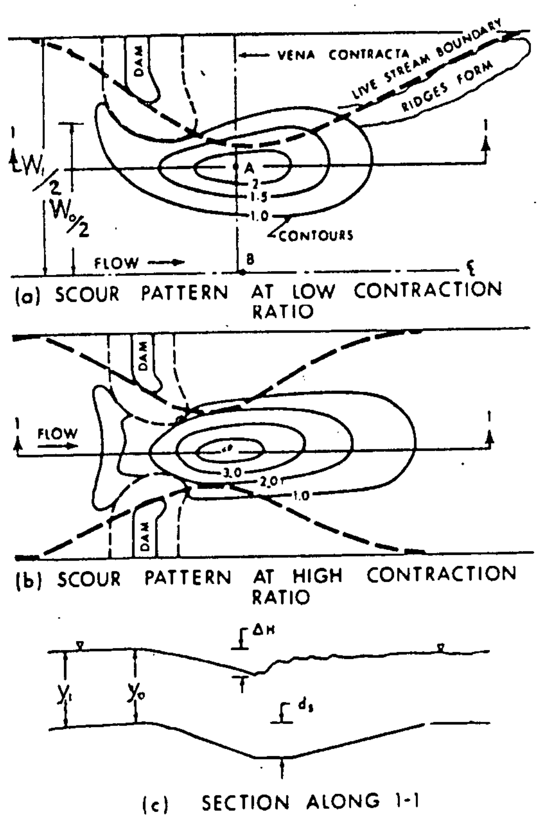


Figure 2.25 Typical Scour Patterns at End-dump Constriction

The formula is not primarily intended for application in bridge design but may be used to obtain an indication of the likely depth of scour that may occur near the abutments. The formula was derived principally to predict the maximum depth of scour occurring during end dump construction. The scour being considered is total scour, that is it has both local and general scour components.

Experiments were conducted in a non-recirculating flume and equation (81) applies therefore to clear-water scour. If used for prediction in cases of scour with general sediment transporation the depth of scour predicted should be slightly conservative. In the experiments three sediments were used the median diameters being 1,20 mm, 0,60 mm and 0,25 mm. In addition experiments were run with both pebble size material (D_{75} = 6,60 mm) and light-weight material (coal with specific gravity 1,25 and median diameter of 1,60 mm).

The following method can be used to solve equation (81) for d_s the depth of scour.

(i) Calculate F_1, the Froude number of the approach flow.

$$F_1 = V_1/\sqrt{gy_1}$$

(ii) Calculate the constriction ratio, β,

$$\beta = W_o/W_1$$

(iii) Using Figure 2.26 below read off value of $\dfrac{y_1+d_s}{y_1} \left(\dfrac{D}{y_1}\right)^{0,29}$

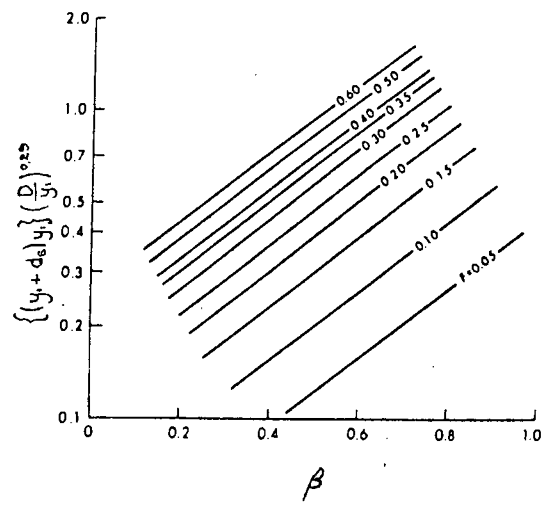


Figure 2.26 Das' Design Curve for Maximum Scour Depth: Clear-water Flow

(iv) Substitute in known values of y_1 and D and solve for d_s. y_1 is in metres and D is measured in millimetres.

2.10 Garde, Subramanya and Nambudripad

Garde et al used dimensional analysis and a series of laboratory model experiments to derive a formula that would predict scour at the nose of a spur dike as shown in Figure 2.27 below. The dimensional analysis revealed the importance of the parameters sediment size or drag coefficient of particles, Froude number and degree of constriction ($\beta = W_o/W_1$), the formula derived was

$$\frac{y_o + d_s}{y_o} = K_G \frac{1}{\beta} F^p$$

or

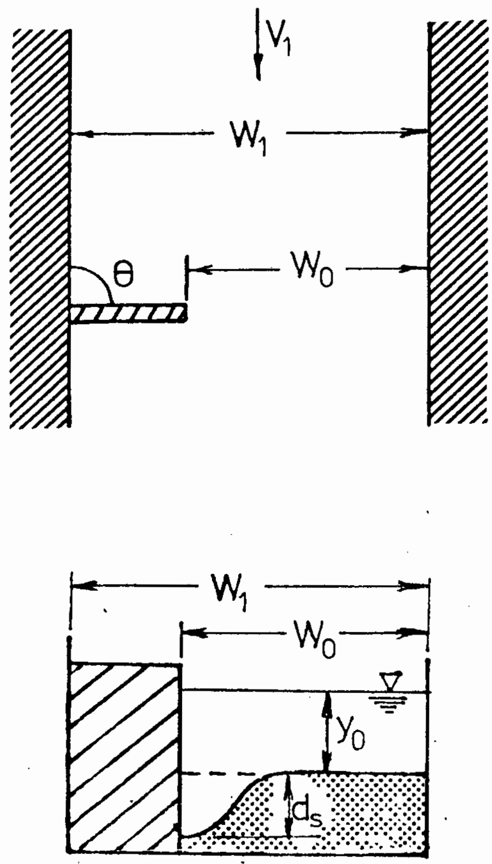


Figure 2.27 Definition Sketch for Garde et al

$$\frac{d_s}{y_o} = K_G \frac{W_1}{W_o} \left(\frac{V_1}{\sqrt{gy_1}} \right)^p - 1 \tag{82}$$

K_G and p are functions of C_D, the coefficient of drag of the sediment particle. Values of K_G and p can be obtained from Figure 2.28.

The coefficient of drag, C_D, can be calculated by the equation

$$C_D = \frac{4}{3} \frac{(\gamma_s - \gamma_w)\ D}{V_f^2\ \rho_w}.$$

where V_f = settling velocity of sediment

Under turbulent conditions (Reynolds number for particle > 1000)

C_D = 0,44.

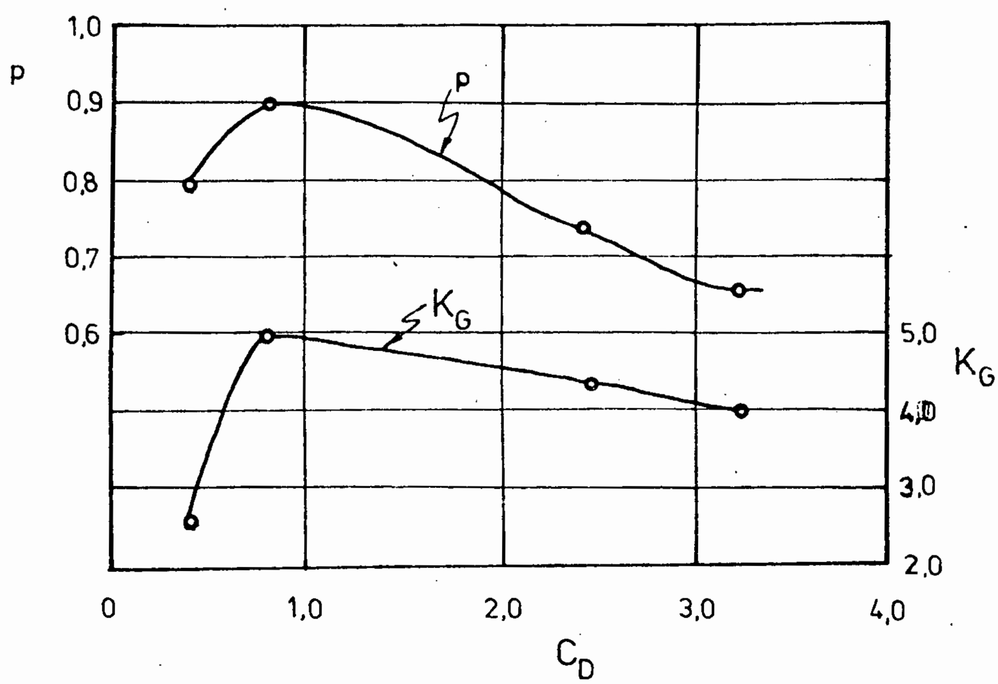


Figure 2.28 Garde et al's Chart Showing Variation of p and K_G

<u>Comment</u> Figure 2.27 shows the type of arrangement for which the formula was derived.

This situation is essentially similar to the one that may occur at bridge abutments. Garde et al also did experiments where the obstruction was placed in the middle of the flume to act like an idealized pier. They state that the data obtained 'show the same tendency as the rest of spur-dike data' and imply that their scour formula can be adapted to pier scour problems.

All experiments were carried out under conditions of general sediment motion and the formula therefore only applies to this condition.

The angle of inclination of the spur dike was kept constant at 90°. Further experiments have shown that for constant Froude number, constriction ratio and sediment size, maximum scour occurs at angle of inclination 90°. Laursen disagrees (see Figure 2.7) and maintains that scour increases with decreasing angle of inclination where the angle is as shown in Figure 2.27.

Two other areas of disagreement emerge between Laursen's theory and the one proposed by Garde et al. Firstly, Garde et al stress the importance of sediment size and its relevance to final scour depth. Secondly, measurements of the breadth and depth of the scour hole showed a breadth/depth ratio varying from 1,8 to 5,0. Laursen uses a figure of 2,75 for this ratio. Garde et al could find no apparent correlation between the breadth/depth ratio and either constriction ratio or sediment diameter.

Neill disagrees with the use of the drag coefficient C_D to measure the sediment characteristics as C_D is independent of sediment size above diameters of 1,5 mm. Scour depth would also then be independent of sediment size above this diameter and rip-rap protection would be ineffective.

From Figure 2.27 it can be seen that the scour depth defined by equation (82) is the local scour depth occurring at the toe of a spur dike. The formula may also be used to predict local scour at a vertical bridge embankment.

2.11 Carstens

From a theoretical consideration of the stability of a sediment particle on a river bed and by using the experiments of Chabert and Engeldinger, Carstens derived a formula to predict local scour at a pier for clear-water conditions.

Carstens showed that the ratio of the sum of all disturbing forces, the drag and lift forces resulting from the flow around the particle, to the resisting forces or the effective weight of the particle, was proportional to the square of a term he called the sediment number or:

$$\frac{\Sigma F_M}{\Sigma F_R} = [f \text{ (sediment-grain geometry)}] N_s^2$$

where ΣF_M = algebraic sum of all disturbing forces

ΣF_R = effective weight of particle

$$N_s = \frac{V_1}{\sqrt{(S_s - 1)gD}} \qquad \text{(Sediment number)} \qquad (84)$$

V_1 = mean velocity of the approaching flow (m/s)

S_s = ratio of solids density to fluid density

g = gravitational constant

D = sediment diameter in metres

Chabert and Engeldigner did extensive experiments using a laboratory recirculating flume, and vertical circular cylinders. Four different sand sizes were used, and scour depth measurements taken at regular time intervals.

Carstens considered the volume of the scour hole and the rate at which the volume increased. When the rate of excavation had reduced to zero scour depth was terminal. The point was given by the empirically based formula.

$$\frac{d_s}{w} = 0{,}546 \left(\frac{N_s^2 - 1{,}64}{N_s^2 - 5{,}02} \right)^{\frac{5}{6}} \qquad (85)$$

w = cylinder diameter

Comment There is doubt in view of the fact that experiments were conducted with a recirculating flume, whether scour was clear-water scour or scour with general sediment motion. In fact Carstens states that for some of the experiments it was necessary in the analysis to deduct the rate of sediment transport into the scour hole. However no experimental data were used where the rate of sediment transport into the scour hole was not either zero or could be given a definite value.

In the experiments only vertical circular cylinders were used. The effect of varying the pier nose shape therefore is not given.

The formula is dimensionless in form and therefore any consistent system of units may be used.

A final point is that the ratio $\Sigma F_M/\Sigma F_R$ would achieve a greater physical meaning if ΣF_M were the vector sum of all motivating forces rather than the algebraic sum.

2.12 Shen, Schneider and Karaki

Shen recognised that the basic mechanism of local scour is the eddies and vortices set up by any obstruction placed in flowing water. Tison showed analytically that as the streamlines bend round an obstruction they achieve a diving component the strength of which is proportional to the curvature of the streamlines. This diving motion was described by Shen as the 'horseshoe vortex' and an analysis of it revealed that its strength was proportional to the Reynolds number of the pier. Shen clearly differentiates between clear-water scour and scour with general sediment motion. The formula given to predict local scour at a pier under clear-water conditions is

$$d_s = 0{,}000223\ R_e^{0{,}619} \text{ metres} \tag{86}$$

R_e is the pier Reynolds number

$$R_e = \frac{V_1 w}{\nu}$$

V_1 = mean upstream undisturbed flow velocity (m/s)

w = pier width in metres

ν = the kinematic viscosity which is temperature dependent

Table 2.7 below shows the relationship between temperature and kinematic viscosity.

Table 2.7 Temperature-kinematic viscosity relationship of water

Temp ^{o}C	ν (mm^2/s)
5	1,519
10	1,308
15	1,141
20	1,007
25	0,897
30	0,804

Using the data published by several researchers Shen found the envelope to all data describing scour with continuous sediment motion was given by both Larras' formula and the one by Breusers.

Larras:

$$d_s = 1{,}05\ K_L\ w^{\,0{,}75} \tag{87}$$

where d_s = depth of scour in metres

w = pier breadth in metres

K_L = pier shape coefficient K_L = 1,0 for cylindrical piers.

K_L = 1,4 for rectangular piers aligned with flow.

Breusers:

$$d_s = 1{,}4\ w \quad \text{Any consistent system of units may be used.} \tag{88}$$

The formula is for circular cylindrical piers.

For larger diameter piers the Breusers formula is more conservative. If bed load sediment transport is fully developed the smaller of the two scour depths from equation (86) and equations (87) and (88) should be used.

In situations where bed load sediment transport is not fully developed a value for equilibrium scour depth must be chosen that is intermediate between the values given by equation (86) and equations (87) and (88). This situation might occur at a bridge whose upstream approaches are armoured.

Shen points out that the depth of scour will vary due to travelling sand dunes on the bed. As the troughs of dunes pass through the scour hole its depth will be increased by one half of the dune height. No method for estimating the probable dune height is put forward.

The strength of the 'horseshoe vortex' is a function of pier shape and pier size. Shen defines 'sharp' piers as those at which, due to their nose shape, no horseshoe vortex develops. At these piers the formulae do not apply. It was found that any pier nose that is wedge shaped with a wedge angle of 30° or less can be regarded as sharp. However flow direction in a turbulent river is not predictable and a sharp nosed pier may be converted to blunt nosed by a change in the angle of attack. Shen recommends for design therefore only that rectangular piers be given a semi-circular front.

The effect of variation of angle of flow attack on the accuracy of equations (86), (87) and (88) is not stated. Shen recommends however that for piers whose axis is not aligned with the flow that the scour be predicted using the method developed by Laursen.

Similarly Shen's experimental work was related to local scour effects. For calculating general scour he recommends either Komura's or Laursen's methods. Komura's analysis is believed by Shen to be an improvement over that developed by Laursen.

2.13 Larras

One of the methods recommended by Shen for the determination of local scour with general sediment motion was that developed by Larras. His formula is given in equation (87)

$$d_s = 1{,}05\ K_L\ w^{0{,}75}$$

d_s = scour depth in metres

w = pier width in metres

K_L = a factor dependent on the pier shape and the alignment of its axis of symmetry to the direction of the current. Values are shown in Table 2.8 below.

Table 2.8 Larras: Variation of Scour Depth with Pier Shape and Alignment

Shape of Pier	Length/ Width	Coefficient K_L					
		0^o	10^o	15^o	20^o	30^o	45^o
Circular		1,1	1,0	1,0	1,0	1,0	1,0
Lenticular	2	0,91				1,13	
	3	0,76	0,98	1,02	1,24		
	4	0,76		1,12		1,50	2,02
Elliptical	2	0,91				1,13	
	3	0,83		1,06	1,24		
Oval	4	0,92		1,18		1,51	
Oblong	2	1,0				1,17	
	3	1,0	1,02	1,13	1,24		
	4	1,0		1,15		1,52	
Rectangular	2	1,11		1,38		1,56	1,65
	4	1,11		1,72w		2,17	2,43
	6	1,11		2,00		2,69	3,05
	8	1,11		2,23		3,03	3,64
	10	1,11		2,48		3,43	4,16

Equation (87) and Table 2.8 were obtained from an analysis of the data obtained in the laboratory experiments of Laursen and Toch, Chabert, Tison and the field measurements of Richard.

2.14 Chitale

From a series of experiments on a model of the Hardinge Bridge, Chitale found a relationship between the upstream Froude number, upstream depth of flow and local scour depth at the nose of a pier.

The equation of this relationship is

$$\frac{d_s}{y_1} = 0{,}51 + 6{,}65\ F_1 - 5{,}49\ F_1^2 \qquad (89)$$

The experiments were done with four sizes of sand varying from 0,16 mm to 1,51 mm in diameter and the results are reproduced in Figure 2.29 below. A plot of upstream flow depth against scour depth showed no correlation.

Comment It is not stated explicitly whether the experiments were carried out under clear-water conditions or conditions of general sediment motion. However, Chitale states'In a few tests in which the upstream depth was less than stable the upstream bed scoured and blanketed the scour pit around the pier. In such cases the maximum depth of scour was measured just before deposition in the scour hole of sand from upstream occurred', and from this it appears that most of the tests were for clear-water scour and for those that weren't measurements were taken in such a way as to simulate clear-water scour. This equation then would not be applicable to scour with general sediment motion.

Although the local scour at the piers was measured no effects for pier size, alignment of pier axis to flow, or shape of nose of pier appears in the formula. It is not certain whether the formula was intended as a design law but it appears that it may be applicable only to bridges geometrically similar to the Hardinge Bridge.

From the description of the experiments it also appears that the scour measured was total scour.

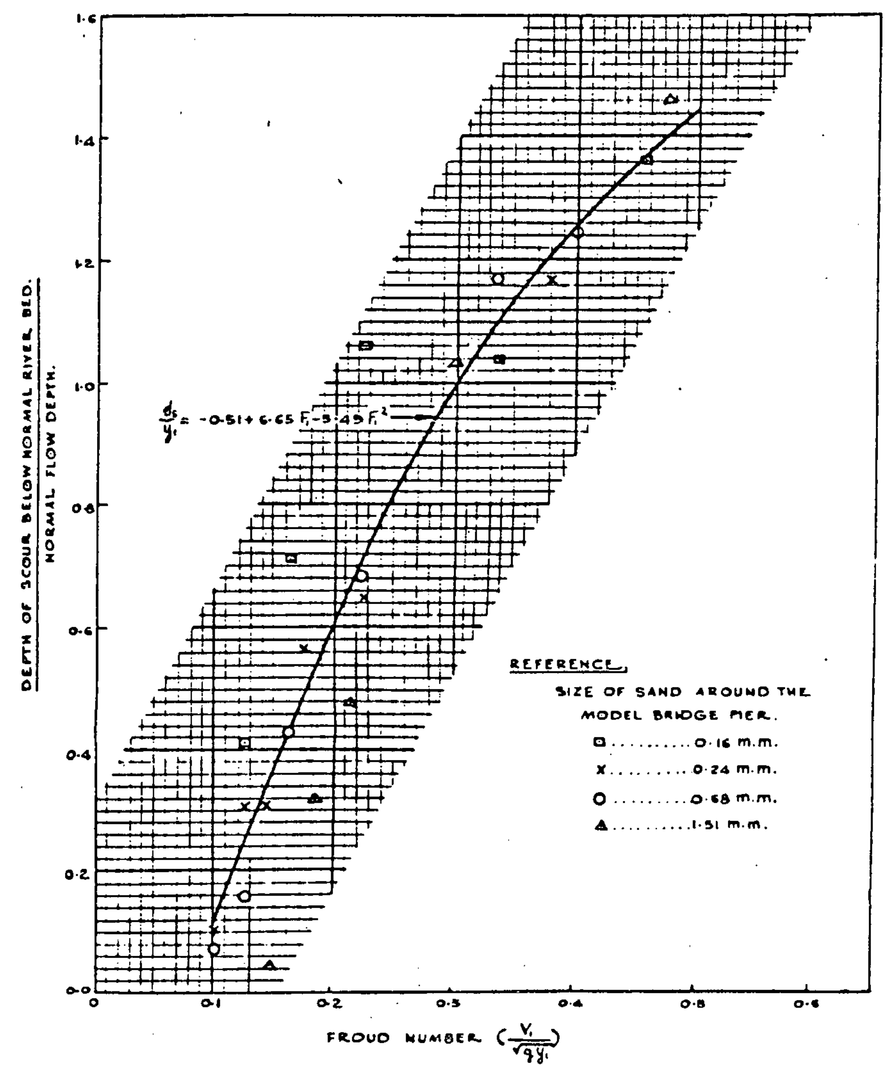


Figure 2.29 Results of Experiments by Chitale

2.15 Coleman

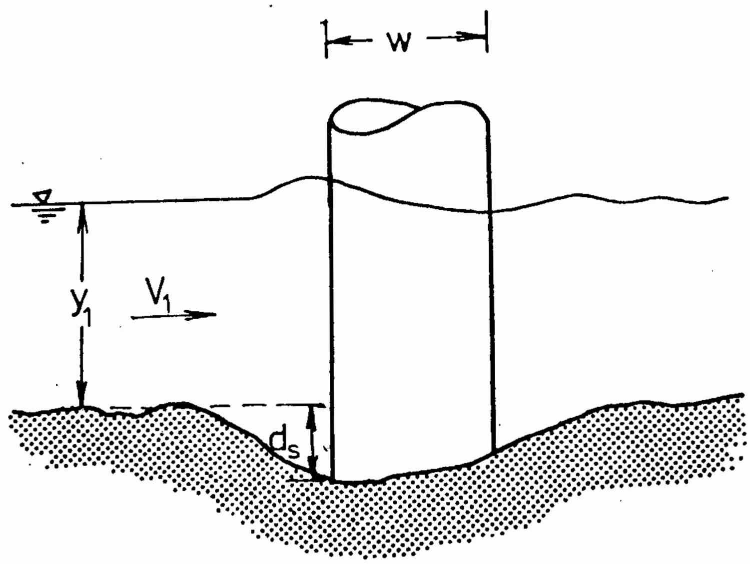


Figure 2.30 Definition Sketch of Local Pier Scour by Coleman

Using the results of his own laboratory experiments, those of Shen et al and dimensional analysis Coleman developed a formula to predict local scour at a bridge pier. Experiments were carried out with a recirculating flume and so were performed under conditions of general sediment motion. Only cylindrical piers were used.

The resulting equation was

$$d_s = 1{,}49\ w^{\frac{9}{10}} \left(\frac{V_1^2}{2g} \right)^{\frac{1}{10}} \tag{90}$$

and stemmed from a graphical examination of the scour Euler number

$\left(\frac{V_1}{\sqrt{2gd_s}}\right)$ and the pier Reynolds number $\left(\frac{V_1 w}{\nu}\right)$.

This relationship is similar to the one found by Breusers, namely that $d_s = 1{,}4\ w$ for circular cylindrical piers.

Once again this is a formula that has been developed purely in a laboratory and the accuracy of its application to prototype conditions is therefore uncertain. Pier diameters of cylinders used in the experiments varied from 0,145 ft to 0,5 ft but two data supplied by Shen relating to a 3 ft diameter pier were completely inconsistent with the other data. Coleman points out though that the formula can perhaps be applied to prototype conditions as the Froude numbers included are of the order of magnitude of prototype Froude numbers that would occur in practice.

For the following scour prediction methods discussion will necessarily be limited as the papers in which they are presented could not be obtained. However they have been described in papers by other researchers and are included here for completion's sake.

2.16 Straub

In 1939 Straub presented the first formula predicting scour in a long constriction and based on geometry alone.

$$\frac{d_s}{y} = \frac{1}{\left(\frac{W_o}{W_1}\right)^{\frac{9}{14}}} - 1 \qquad (91)$$

The formula can be applied to the contraction caused by bridge piers if the degree of contraction is in excess of 10 percent.

2.17 Neill

The following formulae describe the maximum scour that can be obtained at any velocity

$$\frac{d_s}{y_1} = 1{,}5 \left(\frac{w}{y_1} \right)^{0{,}7} \tag{92}$$

or

$$\frac{d_s}{w} = 1{,}5 \left(\frac{y_1}{w} \right)^{0{,}3} \tag{93}$$

These formulae fit the curve given by Laursen (Figure 2.4) for local scour at a pier.

For round nosed piers the coefficient changes to 1,2 if the pier is aligned with the flow. For any unaligned piers use the coefficient 1,5, and w, the pier width, is the width of pier projected normal to the flow direction.

The implication of the term 'maximum scour' is that it is clear-water scour since greater scour depths can be attained under clear-water conditions than under conditions of general sediment motion. Laursen however originally produced the curve for the general sediment motion case.

In an unpublished report (54) Laursen's curves (Figures 2.4 and 2.5) are tentatively suggested for use in design for estimating local scour.

2.18 Maza and Sanchez

Maza and Sanchez studied the criteria given by Laursen and Toch and those of Yaroslavtsiev. After further laboratory experiments of their own they concluded that scour could be predicted by using the lesser value of the solution given by the Laursen and Toch methods (see Laursen Method II) and their own adaptation of the formula used by Yaroslavtsiev.

Maza and Sanchez's formula is:

$$\frac{d_s}{w} = K_1 K_2 \frac{V_1^2}{gw} - \frac{3D}{w} \qquad (94)$$

where d_s = depth of scour in metres

D = sediment diameter in millimetres

w = pier width in metres

V_1 = undisturbed flow velocity in metres per second

K_1 a coefficient depending on pier geometry and angle of flow attack. See Figure 2.31 below

K_2 a coefficient depending on depth of flow to pier width ratio (y/w) and value of V_1^2/gw as shown in Figure 2.32 below

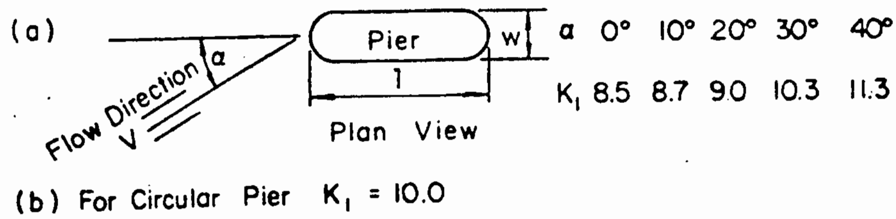


Figure 2.31 Variation of K_1 with flow angle after Maza & Sanchez

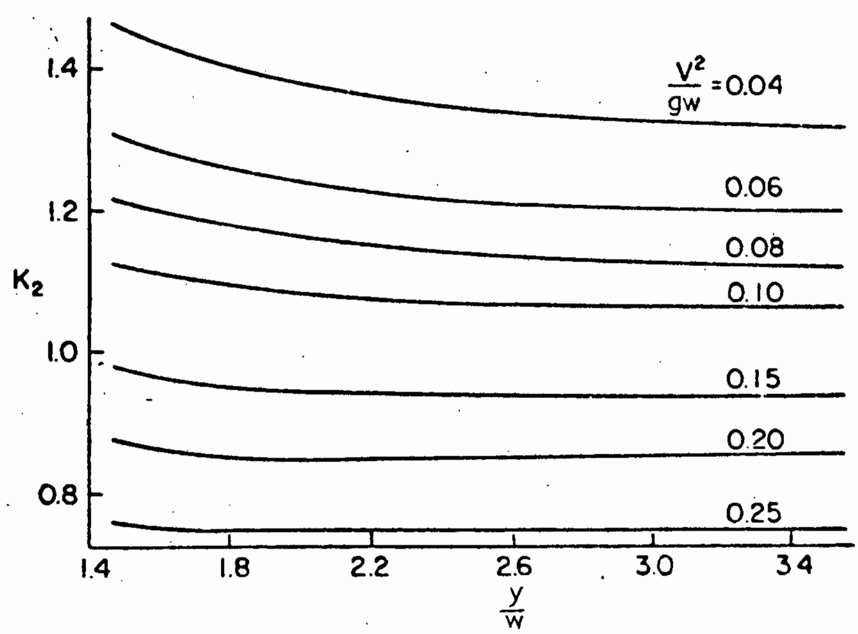


Figure 2.32 Variation of K_2 with flow after Maza & Sanchez

The fact that this formula by Maza and Sanchez is to be used in conjunction with Laursen and Toch's formula which is for the case of local scour at a pier with conditions of general sediment motion implies that it is intended for those same conditions.

2.19 Breusers

From a study of scour around drilling platforms Breusers concluded that for a circular cylindrical pier the local scour was a function only of the pier width or

$$d_s = 1{,}4\ w \tag{95}$$

TABLE 2.9 SUMMARY OF FORMULAE APPLICATION

No	Author	V A R I A B L E S C O N S I D E R E D											
		Flow Characteristics							Sediment Characteristics			Actual Bed Shear Stress	Mode of Sediment Transport
		Magnitude	Depth	Velocity	Waterway Width	Approach Angle	Froude No	Reynolds No	Diameter	Fall Velocity	Critical Shear Stress		
1	Laursen I	x			x				x	x			
2	Laursen II		x			x							x
3	Laursen III	x	x			x							x
4	Laursen IV		x	x	x				x		x	x	
5	Laursen V		x	x		x			x		x	x	
6	Laursen VI		x	x		x			x		x	x	
7	Holmes	x	x	x	x								
8	Lacey	x			x				x				
9	Inglis	x							x				
10	Blench	x			x	x			x				
11	Ahmad	x			x								
12	Herbich & Brennan I								x	x			
13	Herbich & Brennan II			x					x		x	x	
14	Herbich & Brennan III			x					x	x	x	x	
15	Herbich & Brennan IV			x					x	x	x	x	
16	Herbich & Brennan V			x					x	x	x	x	
17	Komura I		x		x							x	
18	Komura II		x		x						x		
19	Komura III		x	x	x		x		x				
20	Komura IV		x	x	x		x		x				
21	Das		x	x			x		x				
22	Garde		x	x	x		x		x				
23	Carstens			x					x				
24	Shen			x				x					
25	Larras					x							
26	Chitale		x	x			x						
27	Coleman			x				x					
28	Straub				x								
29	Neill		x			x							
30	Maza & Sandez		x	x		x			x				
31	Breusers												

TABLE 2.9 SUMMARY OF FORMULAE APPLICATION

No	Author	Formula Based On: Field Results	Formula Based On: Lab Model	Formula Based On: Theory	Formula Based On: Dimensional Analysis	Formula Predicts: General	Formula Predicts: Local Abutment	Formula Predicts: Local Pier	Formula Predicts: Total	Formula for Scour with: Gen. Sed Motion	Formula for Scour with: Clear Water	Variables Considered, Bridge Characteristics: Constriction Ratio	Variables Considered, Bridge Characteristics: Pier Shape	Variables Considered, Bridge Characteristics: Abutment Shape	Variables Considered, Bridge Characteristics: Pier Width
1	Laursen I		x	x		x				x					
2	Laursen II		x					x		x			x		x
3	Laursen III		x				x			x				x	
4	Laursen IV		x	x		x					x	x			
5	Laursen V		x	x				x			x				x
6	Laursen VI		x	x			x				x		x	x	
7	Holmes	x							x	x					x
8	Lacey	x				x				x					
9	Inglis	x							x	x					
10	Blench								x	x					
11	Ahmad	x	x			x				x					
12	Herbich & Brennan I	x			x	x									
13	Herbich & Brennan II	x			x	x									
14	Herbich & Brennan III	x			x	x									
15	Herbich & Brennan IV	x			x	x									
16	Herbich & Brennan V	x			x	x									
17	Komura I		x	x		x				x		x			
18	Komura II		x	x		x					x	x			
19	Komura III		x		x	x				x		x			
20	Komura IV		x		x	x				x	x	x			
21	Das		x	x			x				x				
22	Garde		x		x		x			x		x			
23	Carstens		x	x				x		?	x				x
24	Shen							x			x				x
25	Larras	x	x					x		?	?		x		x
26	Chitale		x						x		x				
27	Coleman		x		x			x		x					x
28	Straub					x				x		x			
29	Neill		x					x		x			x		x
30	Maza & Sanchez		x	x				x		x			x		x
31	Breusers							x							x

CHAPTER 3

Two Case Studies

This chapter examines two bridges in the Cape Province that have failed because of scour. The type of investigation is similar to that carried out by P.S. Holmes on failed railway bridges in New Zealand (27). However, instead of leading to the development of another formula, the information obtained describing conditions at the bridge at the time of the flood is applied and compared to the formulae given in Chapter 2. By analysing several bridges perhaps a formula that would adequately describe the scour in each case could be developed, or greater confidence established in one of the existing formulae.

The case studies here are presented primarily to show the type of research that is possible, rather than as definitive reports of these two particular failures. Accurate data was difficult to obtain and in some cases, particularly with regard to the sediment characteristics, the data had to be assumed - and therefore will not be accurate. A description of how the data was obtained is given together with the data and from this an assessment as to its accuracy may be made.

As some of the data is inaccurate and as only two bridge failures are analysed a degree of discretion should be exercised before rejecting any formulae which show large prediction errors.

All calculations are shown in full in order to demonstrate the methods of formula application described in Chapter 2.

3.1 Case Study No 1. Boy Retief Bridge

Location: Great Fish River Mouth, Eastern Province

Date of Failure: 6 March, 1974

General Description of Bridge

Pictures of the bridge taken at the time of the flood that caused the bridge to fail are included in this document and are shown on pages (98) and (99).

The bridge was intended for road traffic but was in fact in the process of being replaced when it was washed away. The foreground and background of some of the pictures show the construction of the new bridge.

Figure 3.1 page (100) shows the general layout of the bridge including the approach embankment and the area of overflow over this embankment.

Overall the bridge was 215 metres long consisting of eleven 19,12 metre spans. It crossed the main channel of the Great Fish River and had a long embankment approach stretching across the mud flats overbank area on the right bank.

Two piers failed in the flood. They were the two central piers - or piers numbers 5 and 6, looking downstream and commencing to count at the left.

Pier and Foundation Details

Piers were constructed on caissons of dimensions shown in Figure 3.2. Local scour will be determined by the dimensions and

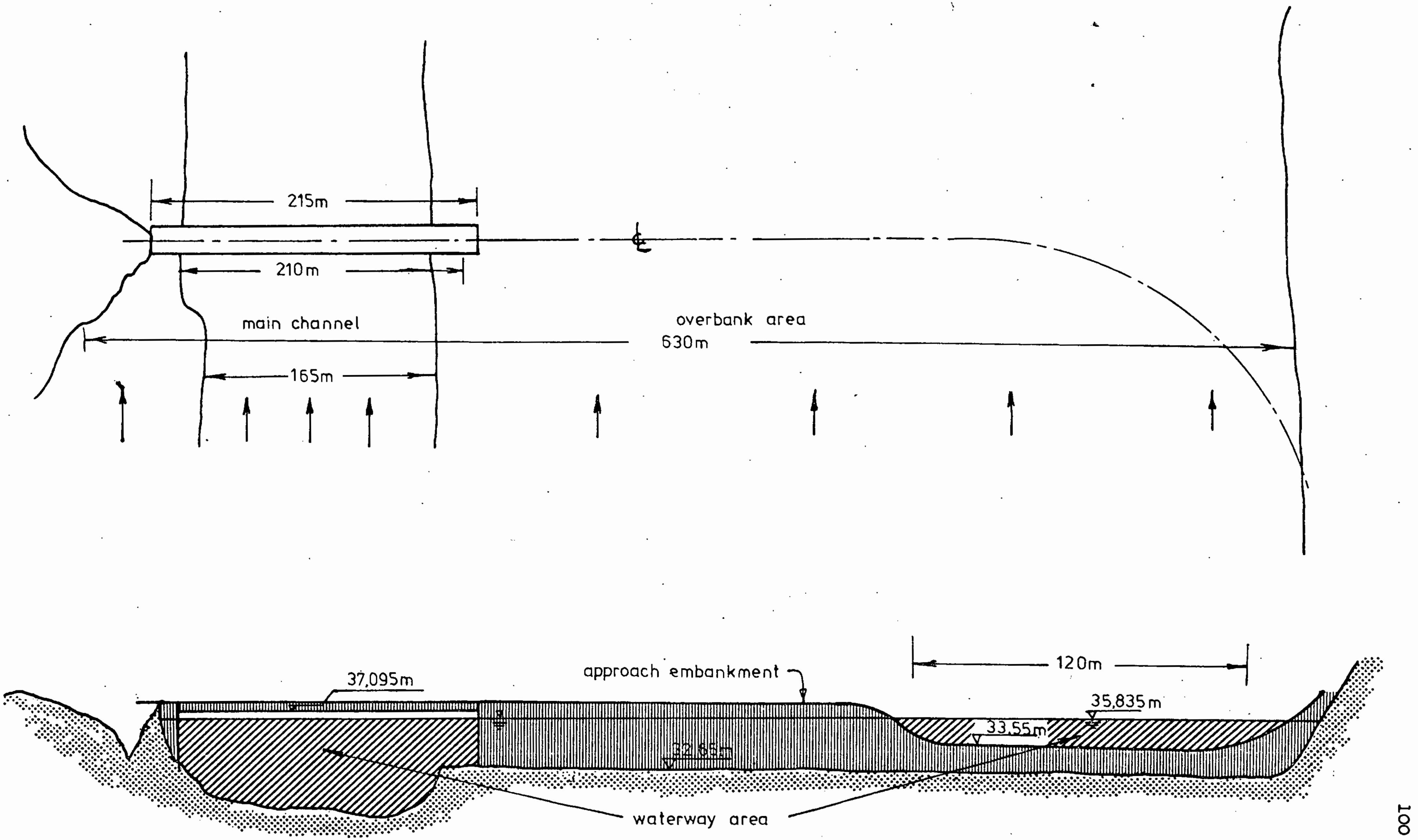


Figure 3.1 Boy Retief Bridge - General Layout

not to scale

shape of the caissons rather than the piers as the tops of the caissons are immediately below, or in some cases are above, normal bed level.

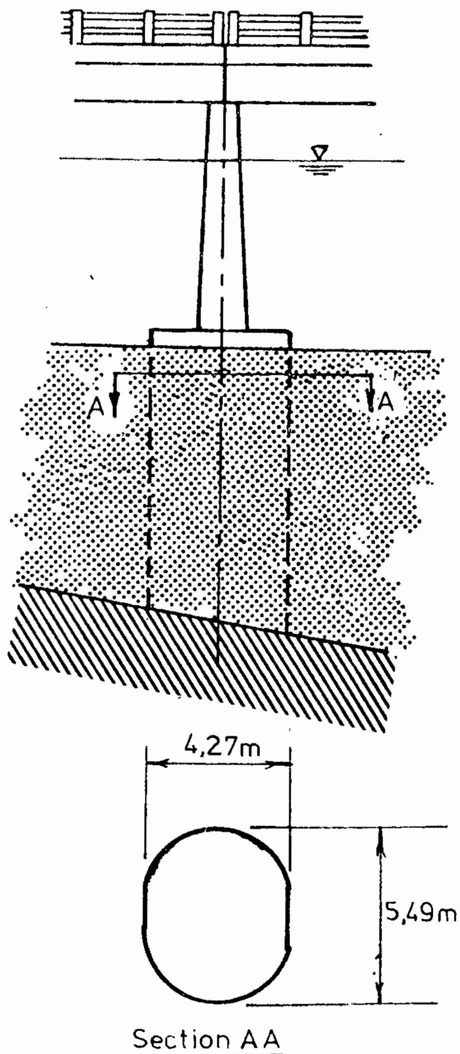


Figure 3.2 Boy Retief Bridge - Pier & Foundation Details

Foundation levels of the caissons are shown in Table 3.1 below. Piers are numbered from left to right looking downstream. The caissons to piers 1 to 7 were founded on a rock shelf but the caissons at piers 8 to 10 and at the right abutment are stopped off in the mud and sand.

Table 3.1 Foundation Depths - Boy Retief Bridge

Pier No	Foundation Level (m)	Bed Level	Foundation Depth	Foundation Condition
1	31,72	31,72	0	Rock
2	27,45	30,32	2,87	Rock
3	25,92	30,29	4,37	Rock
4	24,80	30,26	5,46	Rock
5*	20,50	30,07	9,57	Rock
6*	17,11	29,89	12,78	Rock
7	13,85	29,78	15,93	Rock
8	14,18	30,54	16,36	Mud and Sand
9	18,05	32,43	14,38	Mud and Sand
10	18,49	32,45	13,96	Mud and Sand

* indicates pier that failed

Both of the piers that failed were, according to a plan of the bridge, founded on rock. The photographs show that the turbulence was greatest at piers 4, 5 and 6. It is surprising that pier 4 did not fail along with piers 5 and 6, especially in view of its lesser foundation depth. Pier 7 may have been saved on account of its greater foundation depth and by the fact that the turbulence around this pier appears to be less than that which is occurring at the more central piers.

Flood Magnitude

3 400 cubic metres per second. This figure was obtained from the Department of Water Affairs who maintain a series of recording weirs and gauging stations around South Africa. The gauging station from which this figure was obtained is one hundred kilometres upstream from station Q9M18 which was the station nearest the bridge. Unfortunately station Q9M18 was put out of commission in February 1973, to allow a bridge to be built over it.

The figure of 3 400 m^3/s was recorded at the station on the day preceding the failure of the bridge and represents the peak value of the flood to pass over the weir. The figure is approximate as the gauging station is not calibrated for very high flows.

Maximum Flood Level

The maximum flood level at the bridge was recorded at 35,835 m. This level is 1,26 m below the soffit of the bridge deck.

Flow under Bridge at Peak Flood

Although the peak river discharge was measured at approximately 3 400 m^3/s a portion of this flow passed over the approach embankment to the bridge on the right bank of the river. This embankment runs for about 120 metres at a level of 33,55 m which is 2,28 metres below the peak flood level. The flow actually passing under the bridge was calculated assuming that the flows would split in proportion to the respective values of $AR_H^{\frac{2}{3}}$ at the bridge and over the embankment.

Considering an unscoured profile and making no reduction for the area of the piers the waterway area under the bridge at peak flood was 1020 m^2. Assuming a slight increase in respect of the scour area, say waterway area is equal to 1 100 m^2.

Then at the bridge $R_H = \frac{1100}{250} = 4,4$

and $AR_H^{\frac{2}{3}} = 1100\ (4,4)^{\frac{2}{3}} = 2954$

The waterway area over the embankment = 376 m^2.

Wetted perimeter at embankment $\approx$ 290 m

$$AR_H^{\frac{2}{3}} = 376 \left(\frac{376}{290}\right)^{\frac{2}{3}} = 448$$

and the flow splits approximately in proportions 2950:450.

The flow passing under the bridge $= 3400 \left(\frac{2950}{2950 + 450}\right) = 2950\ m^3/s.$

That portion of the overbank flow passing under the bridge is approximately 400 m^3/s.

Bed Slope

The gradient of the bed was taken from contours on a plan of the site. Bed slope = 0,0018.

Flow Velocity

The velocity can be tentatively calculated either from the Manning formula or by dividing the flow by the known waterway area.

The Manning formula gives $V = \dfrac{R_H^{\frac{2}{3}}\ S^{\frac{1}{2}}}{n}$

Say n = 0,05 (Table 2.6); S = 0,0018 and R_H = 4,4

Then V = 2,28 m/s

and Velocity $= \frac{\text{Flow}}{\text{Area}} = \frac{2950}{1100} = 2{,}68$ m/s

The discrepancy between the two values can be decreased by assuming a larger waterway area under the bridge. For example if this area is assumed to be 1200 m^2 (i.e. the increase in area due to scour is 1200 - 1020 = 180 m^2) then the velocities become 2,41 m/s (Manning) and 2,5 m/s.

For the purposes of the following calculations the velocity at peak flood condition at the bridge was assumed to be 2,5 m/s - which is a value intermediate to 2,28 m/s and 2,68 m/s.

The approach velocity of the flow upstream of the bridge in the main channel is also approximately 2,5 m/s. If the overbank flow is included the approach velocity of the flow reduces to 1,4 m/s.

Flow Depth

The following information was obtained from correspondence with the contractor responsible for the construction of the new bridge - which was under construction at the time of the flood.

'One week before the flood the depth was approximately 4,2 m. About one week after the flood the depth was found to be 15 metres'. Unfortunately the point on the bridge at which these measurements were taken was not recorded but it can be assumed to be near the centre of the river at approximately the maximum depth.

No direct measurement was taken at peak flood. However 'eye witnesses recall that when the bridge was washed away the edge spans fell vertically in the water, and disappeared without showing signs of having hit the bottom of the river bed. The length of the span was 19,12 metres and it would appear as if the depth at flood time was in excess of this'.

If a rectangular flow section is assumed the mean depth of flow at the bridge is equal to 5,24 metres.

Similarly, upstream of the bridge, the mean depth of flow can be worked out at 6,7 metres. Assuming the maximum depth occurring at mid-stream, directly upstream of the failed piers, to be 30 percent greater (Lacey uses mean to maximum depth ratio of 1,273) this maximum depth is then 8,7 metres.

If the normal flow depth is taken at 4,2 metres then the rise in water level due to the flood was 1,3 x 5,24 - 4,2 = 2,6 m. The flow depth at piers 5 and 6 assuming an unscoured bed is approximately 35,835 - 30,00 = 5,83 m.

Waterway Width

The waterway width at the bridge is virtually constant with depth and can be taken at 210 metres. Of this width the piers occupy 16 metres ($\approx$ 8%) and once scour has exposed the caissons their constriction effect totals 37 metres ($\approx$ 18%) and the effective waterway width is reduced to 173 m. As the top of the caissons are at approximately normal bed level they will be rapidly exposed by a flood.

The width of the main channel upstream of the bridge is 165 metres - which is narrower than the waterway width at the bridge.

Overbank flow commences as the flood level at the bridge exceeds 32,65 m. The total width of the stream then is 630 metres.

After the flood exceeds a level of 33,55 m a portion of the flow passes over the approach embankment to the bridge.

Water Temperature

As the flood occurred in the late summer the water temperature was probably in the vicinity of 20°C.

Probable Flow Alignment

The following was assumed from a study of the general layout of the site.

While the flow is contained in the main channel flow is aligned with the piers.

The degree of swirl and misalignment of flow around the abutment and piers near the right bank will increase as the overbank flow increases. Piers 5 and 6, the failed piers which will be considered in this case study, should not be affected by this overbank flow because of their distance from the right bank. The angle between the direction of flow and pier axis will therefore be considered to be zero.

Pier Shape

As shown in Figure 3.2 the caissons have semi-circular fronts.

Sediment Size and Grading

Very little definite information could be obtained as to the sediment size and grading.

The general layout plan of the bridge describes the bed material as 'mud and sand'. A wet grading analysis showed that all material passed a 0,075 mm sieve.

For the purposes of the following calculations the median sediment diameter was therefore assumed to be 0,04 mm.

Depth of Scour

The scour at pier number 6 will be assumed to have taken place to a depth at least equal to the foundation depth of the pier. This represents a minimum depth of scour at pier number 6 of 12,8 metres.

It seems likely that the scour did not exceed the foundation depth of pier number 7 - as it did not fail. This pier was founded at a reduced level of 13,85 m. The scour depth at pier 6 probably therefore would not have exceeded 15,9 metres.

The total scoured depth corresponding to a 12,8 metre scour depth, is 18,7 metres. Eye witnesses believed the depth of flow (which at the bridge is equal to the total scoured depth) to be in excess of 19,12 metres - corresponding to a depth of scour of 13,5 metres.

Type of Scour

The scour causing the piers to fail was scour with general sediment motion. It has both general and local components.

Symbols and their Values

In the following calculations the following symbols will be assigned the values given below:

$A = 1020\ m^2$	$V_o = 2,5\ m/s$	$W_o = 173\ m$
$D = 0,04\ mm$	$V_1 = 2,5\ m/s$	$W_1 = 630\ m$
$g = 9,8\ m/s^2$	$Q = 2950 m^3/s$	$W_2 = 165\ m$
$S = 0,0018$	$Q_a = 400\ m^3/s$	$w = 4,27\ m$
$S_s = 2,65$	$Q_c = 2550\ m^3/s$	$y_o = 5,24\ m$
Depth of flow at piers		$y_1 = 6,7\ m$
5 and 6 = 5,83 m		$y_m = 8,7\ m$
		$y_r = 2,6\ m$
		$\alpha = 0^o$

Laursen

The total scour at a bridge pier for scour with general sediment motion is given by the sum of the values for scour worked out by Methods I and II.

Method I - To calculate the General scour

(i) Find value of $\frac{\sqrt{gyS}}{V_f}$

$$V_f = 1,73\ \sqrt{Dg(S_s - 1)}$$

$$\frac{\sqrt{gyS}}{V_f} = \frac{\sqrt{9,81 \times 6,7 \times 0,0018}}{1,73\ \sqrt{4 \times 10^{-5} \times 9,81(2,65 - 1)}}$$

$$= 7,8$$

(ii) 7,8 > 2 so use equation (8)

$$\frac{d_s}{y_1} = \left(\frac{Q}{Q_c}\right)^{\frac{6}{7}} \left(\frac{W_2}{W_o}\right)^{0,69} - 1$$

$$= \left(\frac{2950}{2550}\right)^{\frac{6}{7}} \left(\frac{165}{173}\right)^{0,69} - 1$$

$$= 0,1$$

$$d_s = 0,1 \times 6,7 = 0,68 \text{ m}$$

<u>Method II - To calculate the Local Scour</u>

(i) $\frac{y_o}{w} = \frac{5,24 \times 1,3}{4,27} = 1,59$

y_o is made equal to 5,24 x 1,3 because the piers being considered (numbers 5 and 6) are at mid-stream. Lacey uses a factor of 1,273 to convert mean depth to maximum depth.

(ii) From Figure 2.4 page (16) $\frac{d'_s}{w} = 1,75$

(iii) $\alpha = 0^o$

(iv) $K_\alpha = 1,0$

(v) From Table 2.2 page (18) $K_s = 0{,}9$

Scour depth $d_s = K_s\, w \dfrac{d_s}{w} = 0{,}9 \times 4{,}27 \times 1{,}75$

$= 6{,}73$ m

(vi) $\dfrac{\sqrt{gyS}}{V_f} = 7{,}8 > 2$

Using Figure 2.6 page (19) $\dfrac{d_s}{y_1} = \dfrac{6{,}73}{6{,}7} \approx 1{,}0$

$K_\tau = 1{,}25$

$d_s = 1{,}25 \times 6{,}73 = 8{,}41$ m

Total Scour $= 8{,}41 + 0{,}68 = 9{,}09$ m

Holmes

$$y_s = y_{sg} + y_{s\ell}$$

$$y_{sg} = y_r\, V_a\, K \left(\frac{W_o}{A}\right)^{\frac{1}{2}}$$

$$y_r = 2{,}6 \text{ m}$$

$$V_a = V_1 \left(\frac{y_m}{y_1}\right)^{\frac{2}{3}} = 2{,}5 \left(\frac{8{,}7}{6{,}7}\right)^{\frac{2}{3}} = 3{,}0$$

$$K = \left[\frac{W_o}{4{,}83\, Q^{\frac{1}{2}}}\right]^{\frac{1}{2}} = \left[\frac{173}{4{,}83 \times 2950^{\frac{1}{2}}}\right] = 0{,}81$$

$$\left(\frac{W_o}{A}\right)^{\frac{1}{2}} = \left(\frac{173}{1020}\right)^{\frac{1}{2}} = 0{,}41$$

$$y_{sg} = 2,6 \times 3,0 \times 0,81 \times 0,41 = 2,59 \text{ m}$$

$$\therefore \quad y_{sg} = y_m = 8,7 \text{ m (as } 2,59 < y_m)$$

$$y_{s\ell} = 0,8 \sqrt{V_a\ w}$$

$$= 0,8 \sqrt{3,0 \times 4,27} = 2,86 \text{ m}$$

$$y_s = 8,7 + 2,86 = 11,56 \text{ m}$$

$$d_s = 11,56 - 5,83 = 5,73 \text{ m}$$

Lacey

$$y_s = 1,273 \times 1,34 \left(\frac{Q}{W_o} \right)^{\frac{2}{3}} f^{-\frac{1}{3}}$$

$$f = 50 \sqrt{D} = 50 \sqrt{4 \times 10^{-5}} = 0,32$$

$$y_s = 1,7 \left(\frac{2950}{173} \right)^{\frac{2}{3}} \times 0,32^{-\frac{1}{3}}$$

$$= 16,47$$

$$d_s = 16,47 - 5,83 = 10,64 \text{ m}$$

Inglis

$$y_s = 2 \times y_L$$

$$y_L = 0,47 \left(\frac{Q}{f} \right)^{\frac{1}{3}}$$

$$= 0,47 \left(\frac{2950}{0,32} \right)^{\frac{1}{3}}$$

$$= 9,85 \text{ m}$$

$$y_s = 2 \times 9,85 = 19,7$$

$$d_s = 19,7 - 5,83 = 13,87 \text{ m}$$

<u>Blench</u>

$$y_s = 1,48\ K_b\ q^{\frac{2}{3}}\ F_{bo}^{-\frac{1}{3}}$$

Formula really only applicable to sand beds as F_{bo} not obtainable for $D < 0,1$ mm

Assuming $D = 0,1$ mm then $F_{bo} = 0,6$ (Figure 2.22)

$$K_b = 2,0$$

$$q = \frac{Q}{W_o} = \frac{2950}{173}$$

$$y_s = 1,48 \times 2,0 \left(\frac{2950}{183} \right)^{\frac{2}{3}} \times 0,6^{-\frac{1}{3}}$$

$$= 23,25 \text{ m}$$

$$d_s = 23,25 - 5,83 = 17,42 \text{ m}$$

<u>Ahmad</u>

$$y_s = 1,48\ K_A\ q^{\frac{2}{3}}$$

$K_A = 1,5$ (Table 2.5) $\qquad q = \frac{2950}{173}$

$$y_s = 1,48 \times 1,5 \left(\frac{2950}{173}\right)^{\frac{2}{3}} = 14,71 \text{ m}$$

$$d_s = 14,71 - 5,83 = 8,88 \text{ m}$$

<u>Herbich and Brennan</u>

Equation (46) $\quad d_s = 0,267 + 0,57 \left(\frac{V_s}{V_f}\right)$

Equation (47) $\quad d_s = 0,271 + 0,034 \left[\frac{\tau}{\tau_c} \Big/ \frac{V_o}{V_c}\right]$

Equation (48) $\quad d_s = 0,322 - 0,342 \left(\frac{V_s}{V_f}\right) + 0,046 \left[\frac{\tau}{\tau_c} \Big/ \frac{V_o}{V_c}\right]$

Depth of flow > 3 m

Reynolds number $= \dfrac{V_o\, y_o}{\nu} = \dfrac{2,5 \times 5,24}{1 \times 10^{-6}} = 131 \times 10^5 > 45 \times 10^5$

Therefore use equation (50)

$$d_s = 0,43 - 0,44 \left(\frac{V_s}{V_f}\right) + 0,047 \left[\frac{\tau}{\tau_c} \Big/ \frac{V_o}{V_c}\right]$$

Now calculating the various parameters in turn.

$\tau = \gamma_w\, y_o\, S = 9,81 \times 5,24 \times 0,0018 = 0,093 \text{ kN/m}^2$

$\tau_c = 0,06\ (\gamma_s - \gamma_w)\, D = 0,06\ (26 - 9,81)\, 4 \times 10^{-5}$

$= 3,9 \times 10^{-5} \text{ kN/m}^2$

$$V_o = 2{,}5 \text{ m/s}$$

$$V_c = 4{,}67 \sqrt{\frac{\gamma_s - \gamma_w}{\gamma_w}} \left(\frac{y_o}{D} \right)^{\frac{1}{2}} \sqrt{D}$$

$$= 4{,}67 \; (1{,}65)^{\frac{1}{2}} \left(\frac{5{,}24}{4 \times 10^{-5}} \right)^{\frac{1}{6}} (4 \times 10^{-5})^{\frac{1}{2}}$$

$$= 0{,}27 \text{ m/s}$$

$$V_s = \sqrt{\frac{\tau}{\rho_w}} = \sqrt{0{,}093} = 0{,}30 \text{ m/s}$$

$$V_f = 1{,}73 \sqrt{Dg(S_s - 1)}$$

$$= 1{,}73 \sqrt{4 \times 10^{-5} \times 9{,}81 \times 1{,}65}$$

$$= 0{,}044 \text{ m/s}$$

Hence $$\frac{V_o}{V_c} = \frac{2{,}5}{0{,}27} = 9{,}26$$

$$\frac{\tau}{\tau_c} = \frac{0{,}093}{3{,}9 \times 10^{-5}} = 2385$$

$$\frac{V_s}{V_f} = \frac{0{,}3}{0{,}044} = 6{,}82$$

$$\frac{\tau}{\tau_c} \bigg/ \frac{V_o}{V_c} = \frac{2385}{9{,}26} = 257{,}6$$

Equation (46) $\quad d_s = 0{,}267 + 0{,}57 \times 6{,}82 = 4{,}15 \text{ m}$

Equation (47) $\quad d_s = 0{,}271 + 0{,}034 \times 257{,}6 = 9{,}03 \text{ m}$

Equation (48) $\quad d_s = 0{,}322 - 0{,}342 \times 6{,}82 + 0{,}046 \times 257{,}6 = 9{,}84$ m

Equation (50) $\quad d_s = 0{,}43 - 0{,}44 \times 6{,}82 + 0{,}047 \times 257{,}6 = 9{,}54$ m

Komura

(i) $\frac{L}{W_o} \ll 1{,}0$ so this is a short constriction and Komura's formula is not strictly applicable.

(ii) This example is almost certainly one of scour with general sediment motion, that is $V_{cl} \ll V_{sl}$. The solution therefore is that for the dynamic equilibruum state and equation (71) should be used.

$$\frac{d_s}{y_1} = \left(\frac{\tau_1}{\tau_o}\right)^{\frac{2}{7}} \left(\frac{W_1}{W_o}\right)^{\frac{6}{7}} - 1$$

$$\left(\frac{\tau_1}{\tau_o}\right) = \left(\frac{W_1}{W_o}\right)^{-\frac{2}{3}} = \left(\frac{630}{173}\right)^{-\frac{2}{3}} = 0{,}42$$

or $$\left(\frac{\tau_1}{\tau_o}\right) = C_d^{\frac{7}{2}} \; F_1^{\frac{7}{10}} \left(\frac{W_1}{W_o}\right)^{-\frac{2}{3}} \sigma_{\phi 1}^{-\frac{7}{10}}$$

The value of $\sigma_{\phi 1}$ is not known. However sediment is probably very uniform and $\sigma_{\phi 1}$ therefore close to unity.

Say $\sigma_{\phi 1}^{-\frac{7}{10}} = 1{,}05$

$$\left(\frac{\tau_1}{\tau_o} \right) = 1{,}45^{\frac{7}{2}} \left(\frac{2{,}5}{\sqrt{9{,}81 \times 6{,}7}} \right)^{\frac{7}{10}} \times 0{,}42 \times 1{,}05 = 0{,}71$$

Hence $\left(\frac{\tau_1}{\tau_o} \right) = 0{,}71$

$$\therefore \quad \frac{d_s}{y_1} = (0{,}71)^{\frac{2}{7}} \left(\frac{630}{173} \right)^{\frac{6}{7}} - 1$$

$$= 1{,}74$$

$$d_s = 1{,}74 \times 6{,}7 = 11{,}69 \text{ m}$$

Using equation (79), developed by Komura from dimensional analysis

$$\frac{d_s}{y_1} = C_d \; F_1^{\frac{1}{5}} \left(\frac{W_1}{W_o} \right)^{\frac{2}{3}} \sigma_{\phi 1}^{-\frac{1}{5}} - 1$$

$$C_d = 1{,}45$$

If $\sigma_{\phi 1}^{-\frac{7}{10}} = 1{,}05$

then $\sigma_{\phi 1}^{-\frac{1}{5}} = 1{,}01$

$$\frac{d_s}{y_1} = 1{,}45 \left(\frac{2{,}5}{\sqrt{9{,}8 \times 6{,}7}} \right)^{\frac{1}{5}} \left(\frac{630}{173} \right)^{\frac{2}{3}} 1{,}01 - 1$$

$$= 1{,}74$$

$$d_s = 1{,}74 \times 6{,}7 = 11{,}69 \text{ m}$$

Das

Theoretically this prediction should give a slightly conservative result as the formula is intended for clear-water scour.

(i) $$F_1 = \frac{V_1}{\sqrt{gy_1}} = \frac{2,5}{\sqrt{9,8 \times 6,7}} = 0,31$$

(ii) $$\beta = \frac{W_o}{W_1} = \frac{173}{630} = 0,27$$

From Figure 2.26 $$\left(\frac{y_1 + d_s}{y_1}\right) \left(\frac{D}{y_1}\right)^{0,29} = 0,31$$

$$y_1 = 6,7 \text{ m}$$

$$D = 0,04 \text{ mm}$$

$$\left(\frac{6,7 + d_s}{6,7}\right) \left(\frac{0,04}{6,7}\right)^{0,29} = 0,31$$

$$d_s = 2,47$$

Garde, Subramanya and Nambudripad

This formula is not applicable here as it is for scour at abutments only.

Carstens

$$\frac{d_s}{w} = 0,546 \left(\frac{N_s^2 - 1,64}{N_s^2 - 5,02}\right)^{\frac{5}{6}}$$

$$N_s = \frac{V_o}{\sqrt{gD(S_s - 1)}} = \frac{2,5}{\sqrt{4 \times 10^{-5} \times 9,81 \times 1,65}}$$

$$= 98,3$$

$$\frac{d_s}{w} = 0,546 \ (1)^{\frac{5}{6}}$$

$$d_s = 0,546 \times 4,27$$

$$= 2,37 \text{ m}$$

<u>Shen, Schneider and Karaki</u>

$$d_s = 0,000223 \ R_e^{0,619}$$

$$= 0,000223 \left(\frac{2,5 \times 4,27}{1,0 \times 10^{-6}} \right)^{0,619} = 5,00 \text{ m}$$

Larras $d_s = 1,05 \ K_L w^{0,75}$

$$K_L = 1,0$$

$$d_s = 1,05 \times 1,0 \times 4,27^{0,75}$$

$$= 3,12 \text{ m}$$

Breusers

$$d_s = 1,4 \ w$$

$$= 1,4 \times 4,27$$

$$= 5,98 \text{ m}$$

Shen recommends the smallest solution

$$d_s = 3,12 \text{ m}$$

<u>Larras</u>

$$d_s = 1{,}05\ K_L w^{0{,}75}$$

$$= 1{,}05 \times 1{,}0 \times 4{,}27^{0{,}75}$$

$$= 3{,}12\ \text{m}$$

The value of K_L is established from Table 2.8 page (85).

<u>Chitale</u>

$$\frac{d_s}{y_1} = 0{,}51 + 6{,}65F_1 - 5{,}49F_1^2$$

$$F_1 = \frac{V_1}{\sqrt{gy_1}} = \frac{2{,}5}{\sqrt{9{,}8 \times 6{,}7}} = 0{,}31$$

$$\frac{d_s}{y_1} = 0{,}51 + 6{,}65 \times 0{,}31 - 5{,}49(0{,}31)^2$$

$$= 2{,}04$$

$$d_s = 2{,}04 \times 6{,}7 = 13{,}66\ \text{m}$$

<u>Coleman</u>

$$d_s = 1{,}49\ w^{\frac{9}{10}} \left(\frac{V_1}{2g} \right)^{\frac{1}{10}}$$

$$= 1{,}49 \times 4{,}27^{\frac{9}{10}} \left(\frac{2{,}5}{19{,}6} \right)^{\frac{1}{10}} = 4{,}91\ \text{m}$$

Straub

$$\frac{d_s}{y_1} = \frac{1}{\left(\frac{W_o}{W_1}\right)^{\frac{9}{14}}} - 1$$

$$= \left(\frac{173}{630}\right)^{-\frac{9}{14}} - 1$$

$$= 1,29$$

$$d_s = 1,29 \times 6,7 = 8,68 \text{ m}$$

Neill

$$\frac{d_s}{y_o} = 1,2 \left(\frac{w}{y_o}\right)^{0,7}$$

$$\frac{d_s}{5,24} = 1,2 \left(\frac{4,27}{5,24}\right)^{0,7}$$

$$d_s = 5,45 \text{ m}$$

Maza and Sanchez

$$\frac{d_s}{w} = K_1 K_2 \frac{V_1^2}{gw} - \frac{3D}{w}$$

From Figure 2.31 $\quad K_1 = 8,5$

and using Figure 2.32 $\quad \frac{V_1^2}{gw} = \frac{2,5^2}{4,27 \times 9,8} = 0,15$

$$\frac{y_1}{w} = \frac{6,7}{4,27} = 1,57$$

$$K_2 = 0,96$$

$$\frac{d_s}{4,27} = 8,5 \times 0,96 \times 0,15 - \frac{3 \times 4 \times 10^{-2}}{4,27}$$

$$= 1,2$$

$$d_s = 1,2 \times 4,27 = 5,12 \text{ m}$$

<u>Breusers</u>

$$d_s = 1,4 \times w$$

$$1,4 \times 4,27$$

$$d_s = 5,98 \text{ m}$$

The following Table summarises the results obtained.

Table 3.2 The Prediction Results - Boy Retief Bridge

No	Formula	Type of Scour		
		General	Local	Total
1	Laursen I	0,68		
2	Laursen II		8,41	
	Laursen I & II			9,09
7	Holmes			5,73
8	Lacey	10,64		
9	Inglis			13,87
10	Blench			17,42
11	Ahmad	8,88		
12	Herbich & Brennan I	4,15		
13	Herbich & Brennan II	9,03		
14	Herbich & Brennan III	9,84		
16	Herbich & Brennan V	9,54		
17	Komura I	11,69		
19	Komura III	11,69		
21	Das			2,47
23	Carstens		2,37	
24	Shen et al		3,12	
25	Larras		3,12	
26	Chitale			13,66
27	Coleman		4,91	
28	Straub	8,68		
29	Neill		5,45	
30	Maza and Sanchez		5,12	
31	Breusers		5,98	

Actual Depth of Scour ≈ 13,5 m (Total Scour)

The numbers used in the first column are the same as those given in Table 2.9 page (94).

3.2 Case Study No 2. Niven's Drift Bridge

Location: Zwartkops River at Uitenhage, Eastern Province

Date of Failure: 22 August, 1971

General Description of Bridge

A sketch showing the general layout of the bridge is given in Figure 3.3 page (125).

This is a road bridge consisting of twenty four spans each 9,65 metres long. The overall length of the bridge is 231,9 metres.

Two piers failed in the flood. They were adjacent piers approximately at mid-stream. Looking downstream and counting from left to right the failed piers were numbers 13 and 14. When the bridge was restored these two piers were replaced by a single centrally placed pier.

Pier and Foundation Details

Figure 3.4 is a sketch showing the relevant pier and foundation details. In a flood the piers would rapidly be excavated to their foundations whose shape and size would determine the local scour depth. The foundations were rectangular 0,79 m wide and 8,66 m long.

Foundations were onto 'hard purple shale'. Table 3.3 below gives the foundation depths.

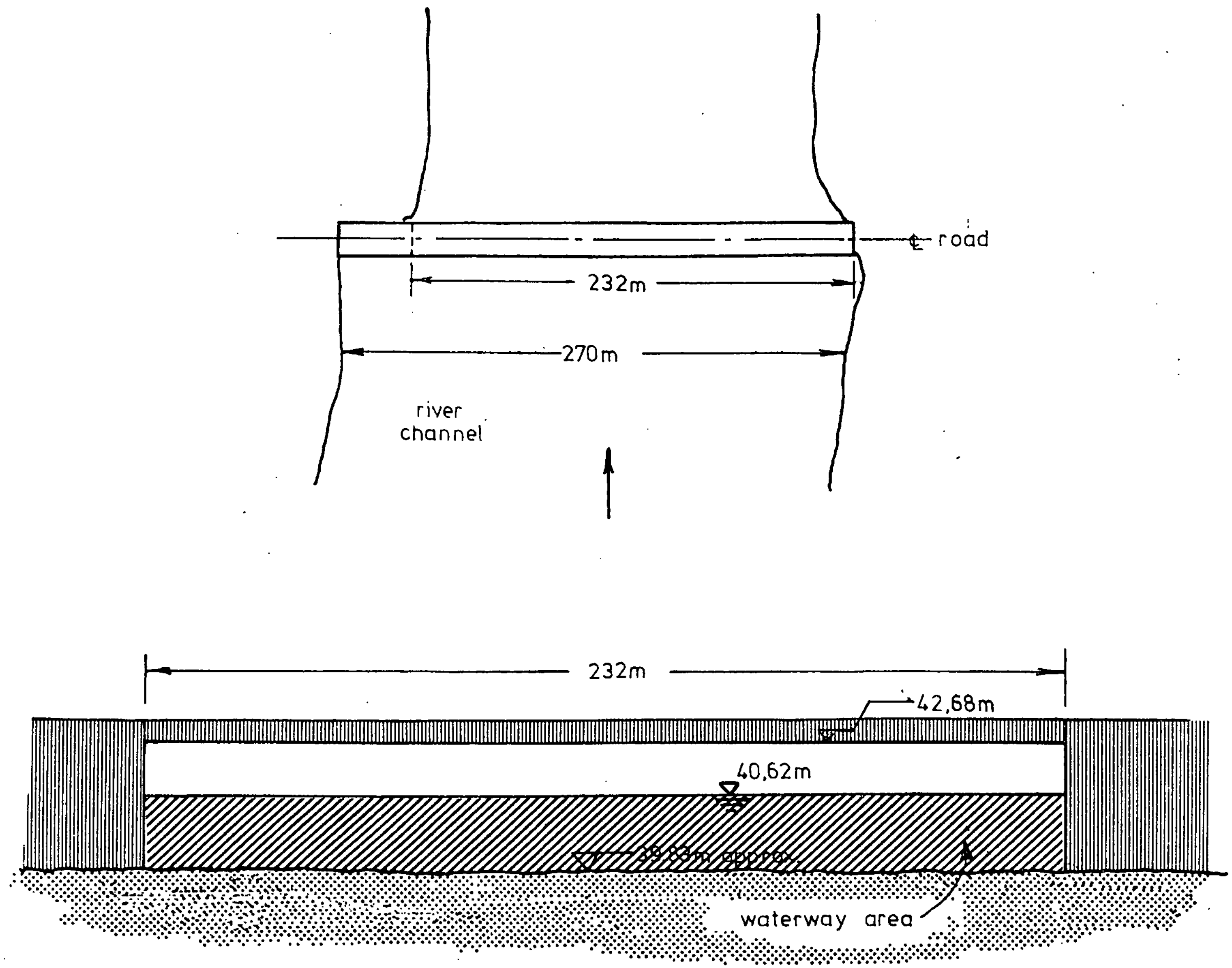


Figure 3.3 Niven's Drift Bridge - General Layout

not to scale

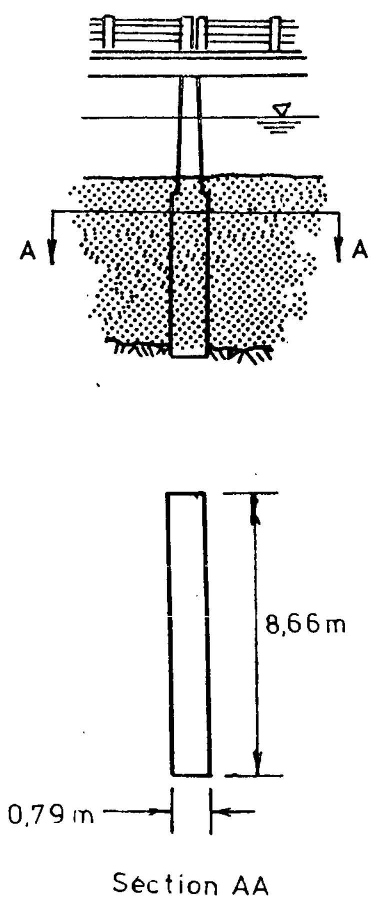


Figure 3.4 Niven's Drift Bridge - Pier & Foundation Details

Table 3.3 Foundation Depths - Niven's Drift Bridge

Pier No	Foundation Level (m)	Bed Level (m)	Depth of Foundation Below Bed Level (m)
1	36,75	40,23	3,48
2	35,45	40,15	3,70
3	36,56	40,08	3,52
4	36,72	40,02	3,30
5	36,25	39,97	3,45
6	36,34	39,91	3,57
7	35,95	39,88	3,93
8	35,67	39,84	4,17
9	34,45	39,78	5,33
10	34,40	39,72	5,32
11	34,41	39,65	5,24
12	34,08	39,67	5,59
13*	35,57	39,68	4,11
14*	35,66	39,69	4,03
15	34,38	39,69	5,31
16	34,59	39,70	5,11
17	34,11	39,71	5,60
18	34,20	39,71	5,51
19	36,15	39,75	3,60
20	34,45	39,78	5,33
21	34,34	39,80	5,46
22	34,35	39,82	5,47
23	34,23	39,83	5,60

* indicates pier that failed.

The profile of the river bed at the bridge is remarkably level.

The mean bed level is 39,83 m.

Maximum Flood Level

The maximum flood level was measured and found to be 2,057 metres below the soffit of the bridge.

The flood level was therefore 40,62 m.

Bed Slope

The gradient of the bed was taken from contours on a plan of the site.

Bed slope = 0,0038

Depth of Scour

From Table 3.3 it is apparent that the depth of scour occurring at the piers near mid stream was between 4,113 metres and 5,31 metres.

Interestingly, piers 1 to 6 and 19 which were founded only 3,6 metres below bed level did not fail. This can probably be explained by the reduced flow velocities near the banks.

Waterway Width

The waterway width at the bridge does not vary with flood level once the full waterway width is being utilised. The width then is 231,8 m of which 25,1 m is occupied by piers. The constriction effect of the piers is therefore approximately 11% and the effective waterway width is 207 metres.

The waterway width upstream of the bridge is 270 m. In times of flood therefore the bridge does constrict the waterway width by approximately 60 metres.

Probable Flow Alignment

The contours of the river bed are not parallel and a degree of skewness of flow direction to pier axis direction is therefore likely. The angle between the flow direction and the pier axis will therefore be assumed to be 5^{o}.

Pier Shape

The shape that will influence the depth of scour is the shape of the foundation which is rectangular.

Flow Velocity, Flow Depth, Waterway Area

The above factors are interdependent. To calculate them the following table was drawn up in which the velocities as calculated by the Manning equation, and as recalculated by dividing the flow by the waterway area could be compared.

Maximum Flood level = 40,62 m

Average Bed Level = 39,83 m

Minimum possible depth of flow = 40,62 - 39,83 m = 0,79 m

Flow = 422 m^3/s; S = bed slope = 0,038; From Table 2.6 n = 0,06

Waterway area occupied by piers = 25,1 x depth

Wetted perimeter of piers = 23 x 2 xdepth

Considering a section at the bridge:-

Flood Magnitude

The nearest flow measuring station to the bridge on the Zwartkops River is the Groendal Dam, station M1R01, where the flow is recorded daily at 8 a.m. The maximum flow recorded in this way during the flood period was 422 m^3/s.

The following flow recordings were obtained.

Date	Flow (m^3/s)
August 21	14
August 22	422
August 23	75

The value of 422 m^3/s is only the minimum value of the flood magnitude, therefore, especially in view of the flashy nature of this flood. The surprisingly low level of the maximum recorded flood level at the bridge and the foregoing calculations indicate that the actual peak value of the flood was not greatly in excess of 422 m^3/s. This value is therefore used in all the following scour calculations.

Sediment Size

No analysis of the bed material giving size and grading could be obtained. The following is the results of a borehole drilled between piers 13 and 14 subsequent to their failure. Bed level at the time of the drilling was 38,94 m.

Depth (m)	Level (m)	Description of Material
Bed level - 2,44	38,94 - 36,50	sand, pebbles, boulders
2,44 - 3,88	36,50 - 35,06	soft mudstone
3,88 - 6,1	35,06 - 32,84	medium hard mudstone

In the calculations that follow the sediment size was assumed to be 0,6 mm.

Water Temperature

The flood occurred in late winter. The water temperature was therefore probably between 15°C and 20°C.

Type of Scour

The scour causing the piers to fail was scour with general sediment motion. It had both local and general components.

Symbols and their Values

In the calculations that follow the symbols shown will be assigned the values given below.

A	=	$185\ m^2$	S	=	0,0038	w	=	0,79 m
D	=	0,6 mm	S_s	=	2,65	y_o	=	1,8 m
g	=	$9,81\ m/s^2$	V_o	=	1,13 m/s	y_1	=	1,3 m
ℓ	=	8,66 m	V_1	=	1,2 m/s	y_m	=	1,69 m
Q	=	$422\ m^3/s$	W_o	=	207 m	y_r	=	1,0 m
Q_c	=	$422\ m^3/s$	W_1	=	270 m	α	=	5^o
Q_a	=	$0\ m^3/s$	W_2	=	270 m			

Depth of flow at piers 13 and 14 = 1,8 m

In the following calculations those that would be a repeat of the calculation shown fully in Case Study 1 are not shown in full and only the result is given.

Laursen

The total scour at a bridge pier for scour with general sediment motion is given by the sum of the values for the scour worked out by Methods I and II.

Method I - To calculate the general scour

$$d_s = 0{,}24 \text{ m}$$

Method II - To calculate the local scour

$$d_s = 5{,}85 \text{ m}$$

Total Scour = 5,85 + 0,24 = 6,09 m

Holmes

$$y_s = y_{sg} + y_{s\ell}$$

$$y_{sg} = y_r \, V_a K \left(\frac{W_o}{A} \right)^{\frac{1}{2}} \qquad \text{or } y_m$$

$$\therefore \quad y_{sg} = y_m = 1{,}69$$

$$y_{s\ell} = 0{,}8 \sqrt{V_a \, w} = 0{,}85 \text{ m}$$

$$y_s = 2{,}54 \text{ m}$$

$$d_s = 2{,}54 - 1{,}8 = 0{,}74 \text{ m}$$

Lacey

$$y_s = 1{,}273 \times 1{,}34 \left(\frac{Q}{W_o} \right)^{\frac{2}{3}} f^{-\frac{1}{3}} = 2{,}57 \text{ m}$$

$$d_s = 2{,}57 - 1{,}8 = 0{,}77 \text{ m}$$

Inglis

$$y_s = 2\ y_L$$

$$y_s = 6{,}6$$

$$d_s = 6{,}6 - 1{,}8 = 4{,}8 \text{ m}$$

Blench

$$y_s = 1{,}48\ K_b\ q^{\frac{2}{3}}\ F_{bo}^{-\frac{1}{3}} = 4{,}25 \text{ m}$$

$$d_s = 4{,}25 - 1{,}8 = 2{,}45 \text{ m}$$

Ahmad

$$y_s = 1{,}48\ K_A\ q^{\frac{2}{3}} = 3{,}57$$

$$d_s = 3{,}57 - 1{,}8 = 1{,}77 \text{ m}$$

Herbich and Brennan

Equation (46)

$$d_s = 0{,}267 + 0{,}57 \left(\frac{V_s}{V_f} \right)$$

Equation (47)

$$d_s = 0{,}271 + 0{,}034 \left(\frac{\tau}{\tau_c} \bigg/ \frac{V_o}{V_c} \right)$$

Equation (48)

$$d_s = 0{,}322 - 0{,}342 \left(\frac{V_s}{V_f}\right) + 0{,}046 \left(\frac{\tau}{\tau_c} \Big/ \frac{V_o}{V_c}\right)$$

Depth of flow < 3 m

$$\text{Reynolds number} = \frac{V_o\,y_o}{\nu} = \frac{1{,}13 \times 1{,}8}{1 \times 10^{-6}} = 20 \times 10^5 < 45 \times 10^5$$

Therefore use equation (49)

$$d_s = 0{,}35 - 0{,}17 \left(\frac{V_s}{V_f}\right) + 0{,}027 \left(\frac{\tau}{\tau_c} \Big/ \frac{V_o}{V_c}\right)$$

Equation (46)

$$d_s = 0{,}267 + 0{,}57(1{,}53) = 1{,}14 \text{ m}$$

Equation (47)

$$d_s = 0{,}271 + 0{,}034(57{,}7) = 2{,}23 \text{ m}$$

Equation (48)

$$d_s = 0{,}322 - 0{,}342(1{,}53) + 0{,}046(57{,}7) = 2{,}45 \text{ m}$$

Equation (49)

$$d_s = 0{,}35 - 0{,}17(1{,}53) + 0{,}027(57{,}7) = 1{,}65 \text{ m}$$

<u>Komura</u>

(i) $\frac{L}{W_o} \ll 1{,}0$ so this is a short constriction and Komura's formula is not strictly applicable.

(ii) From previous calculations (Herbich and Brennan)

$V_c = 0{,}56$ m/s

$V_s = 0{,}26$ m/s

Therefore $V_c > V_s$ and equilibrium state will be the static equilibrium state and equation (72) must be used.

$$\frac{d_s}{y_1} = \left(\frac{\tau_{cl}}{\tau_{co}}\right) \left(\frac{W_1}{W_o}\right)^{\frac{6}{7}} - 1$$

$$\left(\frac{\tau_{cl}}{\tau_{co}}\right) = C_s^{\frac{7}{2}} \; F_1^{\frac{7}{10}} \left(\frac{W_1}{W_o}\right)^{-\frac{2}{3}} \sigma_\phi^{-\frac{7}{4}}$$

The value of $\sigma_{\phi 1}$ is not known. However the description of the bed material as 'sand, pebbles, boulders' indicates a high standard deviation - say $\sigma_{\phi 1} = 1{,}5$

$$C_s^{\frac{7}{2}} = 5{,}18$$

$$F_1 = \frac{V_1}{\sqrt{gy_1}} = \frac{1{,}2}{\sqrt{9{,}8 \times 1{,}3}} = 0{,}34$$

$$\left(\frac{\tau_{cl}}{\tau_{co}}\right) = 5{,}18 \times 0{,}34^{\frac{7}{10}} \left(\frac{270}{207}\right)^{-\frac{2}{3}} 1{,}5^{-\frac{7}{4}} = 1{,}00$$

$$\frac{d_s}{y_1} = (1)^{\frac{2}{7}} \left(\frac{270}{207}\right)^{\frac{6}{7}} - 1 = 0{,}26$$

$$d_s = 1{,}3 \times 0{,}26 = 0{,}33 \text{ m}$$

Using equation (80) developed by Komura using dimensional analysis

$$\frac{d_s}{y_1} = C_s \; F_1^{\frac{1}{5}} \left(\frac{W_1}{W_o}\right)^{\frac{2}{3}} \sigma_{\phi 1}^{-\frac{1}{2}} - 1$$

$$C_s = 1{,}60 \quad \text{and} \quad \sigma_{\phi 1}^{-\frac{1}{2}} = 0{,}82$$

$$\frac{d_s}{y_1} = 1,6 \left(\frac{1,2}{\sqrt{9,8 \times 1,3}} \right)^{\frac{1}{5}} \left(\frac{270}{207} \right)^{\frac{2}{3}} 0,82 - 1$$

$$\frac{d_s}{y_1} = 0,26$$

$$d_s = 0,26 \times 1,3 = 0,34 \text{ m}$$

Das

(i) $$F_1 = \frac{V_1}{\sqrt{gy_1}} = 0,34$$

(ii) $$\beta = \frac{W_o}{W_1} = 0,77$$

(iii) From Figure 2.26 $$\left(\frac{y_1 + d_s}{y_1} \right) \left(\frac{D}{y_1} \right)^{0,29} = 1,2$$

$$d_s = 0,65 \text{ m}$$

Garde, Subramanya and Nambudripad

This formula is not applicable here as it is for scour at abutments only.

Carstens

$$\frac{d_s}{w} = 0,546 \left(\frac{N_s^2 - 1,64}{N_s^2 - 5,02} \right)^{\frac{5}{6}}$$

$$d_s = 0,44 \text{ m}$$

Shen, Schneider and Karaki

$$d_s = 0{,}000223\ R_e^{0,619} = 1{,}12\ m$$

or

Larras $d_s = 1{,}05\ K_L w^{0,75} = 1{,}23\ m$

or

Brausers $d_s = 1{,}4 \times w = 1{,}11\ m$

Shen recommends the smallest solution

$$d_s = 1{,}11 m$$

Larras

$$d_s = 1{,}05\ K_L\ w^{0,75}$$

Using Table 2.8 to find K_L

The angularity of the flow with respect to the pier axis is 5°. The pier length/breadth ratio is greater than 10.

K_L therefore has a value between 1,11 and 2,48 say $K_L = 1{,}8$

$$d_s = 1{,}58\ m$$

Chitale

$$\frac{d_s}{y_1} = 0{,}51 + 6{,}65\ F_1 - 5{,}49\ F_1^2$$

$$d_s = 2{,}78\ m$$

Coleman

$$d_s = 1{,}49\ w^{\frac{9}{10}} \left(\frac{v_1}{2g} \right)^{\frac{1}{10}}$$

$$d_s = 0{,}93\ m$$

Straub

$$\frac{d_s}{y_1} = \frac{1}{\left(\frac{W_o}{W_1}\right)^{\frac{9}{14}}} - 1$$

$$d_s = 0{,}24 \text{ m}$$

Neill

$$\frac{d_s}{y_o} = 1{,}5 \left(\frac{w}{y_o}\right)^{0{,}7}$$

$$d_s = 1{,}52 \text{ m}$$

Maza and Sanchez

$$d_s = K_1 K_2 \frac{V_1^2}{gw} - \frac{3D}{w}$$

$$d_s = 0{,}70 \text{ m}$$

Breusers

$$d_s = 1{,}4 \text{ w}$$

This formula is intended only for cylindrical piers.

$$d_s = 1{,}11 \text{ m}$$

Table 3.4 The Prediction Results - Niven's Drift Bridge

No	Formula	Type of Scour General	Local	Total
1	Laursen I	0,24		
2	Laursen II		5,85	
	Laursen I and II			6,09
7	Holmes			0,74
8	Lacey	0,77		
9	Inglis			4,8
10	Blench			2,45
11	Ahmad	1,77		
12	Herbich & Brennan I	1,14		
13	Herbich & Brennan II	2,23		
14	Herbich & Brennan III	2,45		
15	Herbich & Brennan IV	1,65		
18	Komura	0,33		
20	Komura	0,34		
21	Das			0,65
23	Carstens		0,44	
24	Shen et al		1,11	
25	Larras		1,58	
26	Chitale			2,78
27	Coleman		0,93	
28	Straub	0,24		
29	Neill		1,52	
30	Maza and Sanchez		- 0,70	
31	Breusers		1,11	

Actual Depth of Scour ≈ 4,2 m (Total Scour)

The numbers used in the first column are the same as those given in Table 2.9 page (94)

Table 3.5 RESULT ANALYSIS

No.	Combination	Case Study 1	Case Study 2	% Error 1	% Error 2	Rank 1	Rank 2
100	7	5,73	0,74	- 57,6	- 82,4		
101	9	13,87	4,80	2,7	- 14,13	7	5
102	10	17,42	2,45	29,0	- 41,7		
103	21	2,47	0,65	- 81,7	- 84,5		
104	26	13,66	2,78	1,2	- 33,8	2	
105	1 + 2	9,09	6,09	- 32,7	45,0		
106	1 + 23	3,05	0,68	- 77,4	- 83,8		
107	1 + 24	3,80	1,35	- 71,9	- 67,9		
108	1 + 25	3,80	1,82	- 71,9	- 56,7		
109	1 + 27	5,59	1,17	- 58,6	- 72,1		
110	1 + 29	6,13	1,76	- 54,6	- 58,1		
111	1 + 30	5,80	- 0,46	- 57,0	-111,0		
112	1 + 31	6,66	1,35	- 50,7	- 67,9		
113	8 + 2	19,05	6,62	41,1	57,6		
114	8 + 23	13,01	1,21	- 3,6	- 71,2	9	
115	8 + 24	13,76	1,88	1,9	- 55,2	3	
116	8 + 25	13,76	2,35	1,9	- 44,0	3	
117	8 + 27	15,55	1,70	15,2	- 59,5		
118	8 + 29	16,09	2,29	19,2	- 45,5		
119	8 + 30	15,76	0,07	16,7	- 98,3		
120	8 + 31	16,62	1,88	23,1	- 55,2		
121	11 + 2	17,29	7,66	28,1	81,4		
122	11 + 23	11,25	2,21	- 16,7	- 47,4		
123	11 + 24	12,00	2,88	- 11,1	- 31,4		
124	11 + 25	12,00	3,35	- 11,1	- 20,2		9
125	11 + 27	13,79	2,70	2,1	- 35,7	5	
126	11 + 29	14,33	3,29	6,1	- 21,7		12
127	11 + 30	14,00	1,07	3,7	- 74,5	10	
128	11 + 31	14,86	2,88	10,1	- 31,4		
129	12 + 2	12,56	6,99	- 7,0	66,4		
130	12 + 23	6,52	1,58	- 51,7	- 62,4		
131	12 + 24	7,27	2,25	- 46,1	- 46,4		
132	12 + 25	7,27	2,72	- 46,1	- 35,2		
133	12 + 27	9,06	2,07	- 32,9	- 50,7		
134	12 + 29	9,60	2,66	- 28,9	- 36,7		
135	12 + 30	9,27	0,44	- 31,3	- 89,5		
136	12 + 31	10,13	2,25	- 25,0	- 46,4		
137	13 + 2	17,44	8,08	29,2	92,4		
138	13 + 23	11,40	2,67	- 15,6	- 36,4		
139	13 + 24	12,15	3,34	- 10,0	- 20,5		10
140	13 + 25	12,15	3,81	- 10,0	- 9,3		3
141	13 + 27	13,94	3,16	3,3	- 24,8	8	15
142	13 + 29	14,48	3,75	7,3	- 10,7		4
143	13 + 30	14,14	1,53	4,8	- 63,6	16	

No.	Combi-nation	Case Study 1	Case Study 2	% Error 1	% Error 2	Rank 1	Rank 2
144	13 + 31	15,01	3,34	11,2	- 20,5		10
145	14 + 2	18,25	8,30	35,2	97,6		
146	14 + 23	12,21	2,89	- 9,6	- 31,2		
147	14 + 24	12,96	3,56	- 4,0	- 15,2	11	6
148	14 + 25	12,96	4,03	- 4,0	- 4,0	11	1
149	14 + 27	14,75	3,38	9,3	- 19,5		8
150	14 + 29	15,29	3,97	13,3	- 5,5		2
151	14 + 30	14,96	1,75	10,8	- 58,3		
152	14 + 31	15,82	3,56	17,2	- 15,2		6
153	15 + 2		7,45		77,4		
154	15 + 23		2,09		- 50,2		
155	15 + 24		2,76		- 34,3		
156	15 + 25		3,23		- 23,1		13
157	15 + 27		2,53		- 39,8		
158	15 + 29		3,17		- 24,5		14
159	15 + 30		0,95		- 77,4		
160	15 + 31		2,76		- 34,3		
161	16 + 2	17,95		33,0			
162	16 + 23	11,91		- 11,8			
163	16 + 24	12,66		- 6,2			
164	16 + 25	12,66		- 6,2			
165	16 + 27	14,45		7,0			
166	16 + 29	14,99		11,0			
167	16 + 30	14,66		8,6			
168	16 + 31	15,52		15,0			
169	17 + 2	20,10		48,9			
170	17 + 23	14,06		4,1		13	
171	17 + 24	14,81		9,7			
172	17 + 25	14,81		9,7			
173	17 + 27	16,60		23,0			
174	17 + 29	17,14		27,0			
175	17 + 30	16,81		24,5			
176	17 + 31	17,67		30,9			
177	18 + 2		6,18		47,1		
178	18 + 23		0,77		- 81,7		
179	18 + 24		1,44		- 65,7		
180	18 + 25		1,91		- 54,5		
181	18 + 27		1,26		- 70,0		
182	18 + 29		1,85		- 56,0		
183	18 + 30		- 0,37		-108,8		
184	18 + 31		1,44		- 65,7		
185	19 + 2	20,10		48,9			
186	19 + 23	14,06		4,1		13	
187	19 + 24	14,81		9,7			
188	19 + 25	14,81		9,7			
189	19 + 27	16,60		23,0			
190	19 + 29	17,14		27,0			
191	19 + 30	16,81		24,5			
192	19 + 31	17,67		30,9			
193	20 + 2		6,19		47,4		
194	20 + 23		0,78		- 81,4		
195	20 + 24		1,45		- 65,4		

No.	Combination	Case Study 1	Case Study 2	% Error 1	% Error 2	Rank 1	Rank 2
196	20 + 25		1,92		- 54,2		
197	20 + 27		1,27		- 69,7		
198	20 + 29		1,86		- 55,7		
199	20 + 30		- 0,36		-108,5		
200	20 + 31		1,45		- 65,4		
201	28 + 2	17,09	6,09	26,6	45,0		
202	28 + 23	11,05	0,68	- 18,1	- 83,8		
203	28 + 24	11,80	1,35	- 12,6	- 67,9		
204	28 + 25	11,80	1,82	- 12,6	- 56,7		
205	28 + 27	13,59	1,17	0,7	- 72,1	1	
206	28 + 29	14,13	1,76	4,7	- 58,1	15	
207	28 + 30	13,80	- 0,46	2,2	-111,0	6	
208	28 + 31	14,66	1,35	8,6	- 67,9		

In Table 3.5 numbers start at 100 for convenience. The numbers used in the column headed 'Combination' refer to the numbers in Tables 3.2 and 3.4

Table 3.5 gives all possible combinations of the sums of the estimated general and local scour for both bridges.

The % Error is calculated according to the formula:

$$\% \text{ Error} = \frac{\text{Estimated Scour} - \text{Actual Scour}}{\text{Actual Scour}} \times 100$$

A negative % Error therefore results from an understimate of the true total scour, a positive % Error from an overestimate.

The results were ranked according to their accuracy, with the most accurate prediction receiving the highest rank.

Any underestimate of the scour will lead to failure of the bridge. Therefore perhaps in any ranking process overestimates of the scour should be assigned a higher value than underestimates. In this study however as the object was to find an accurate formula rather than a safe one, positive and negative error values were assigned equal status.

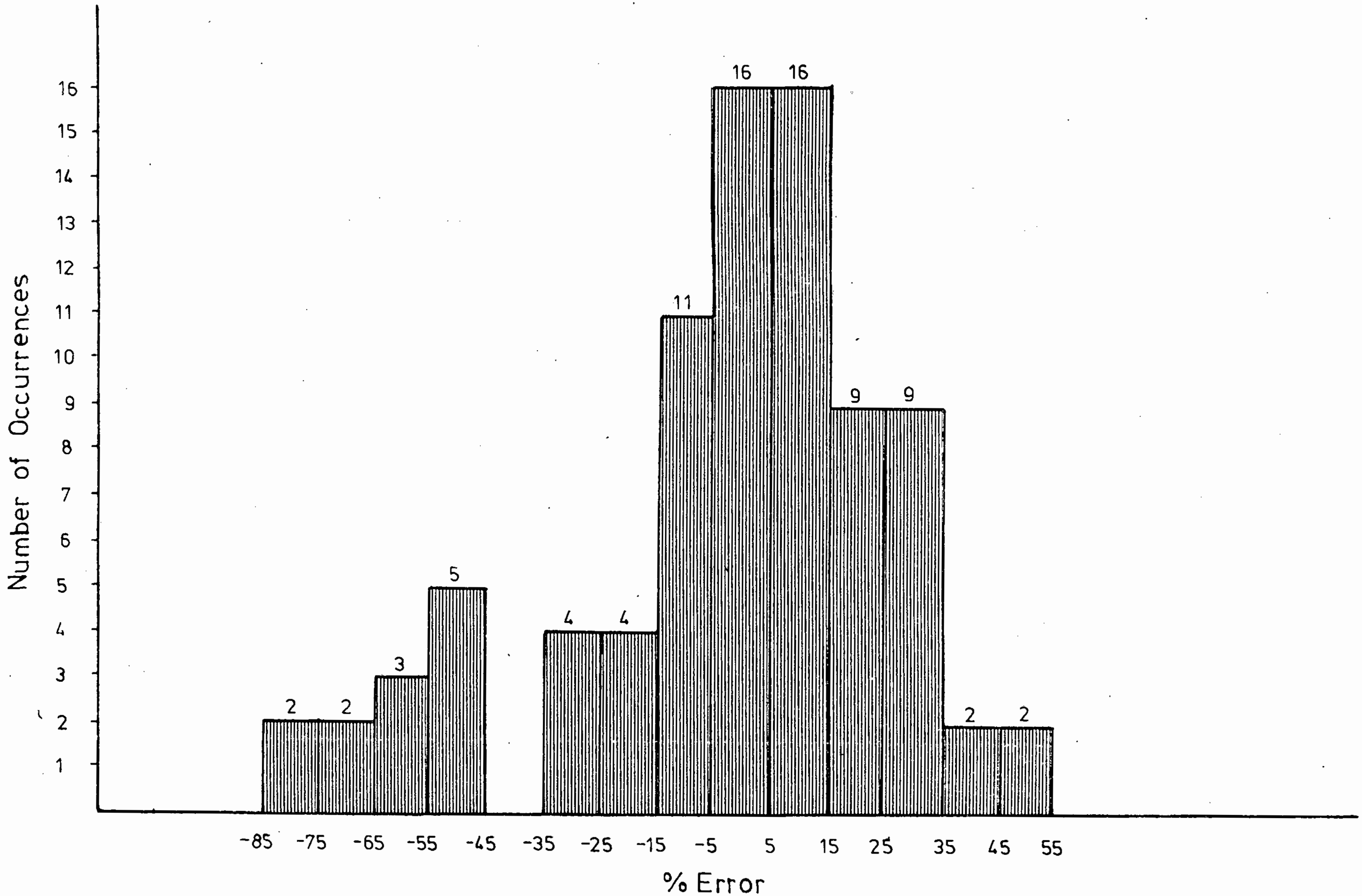


Figure 3.5 Boy Retief Bridge - Distribution of % Error of Predictions

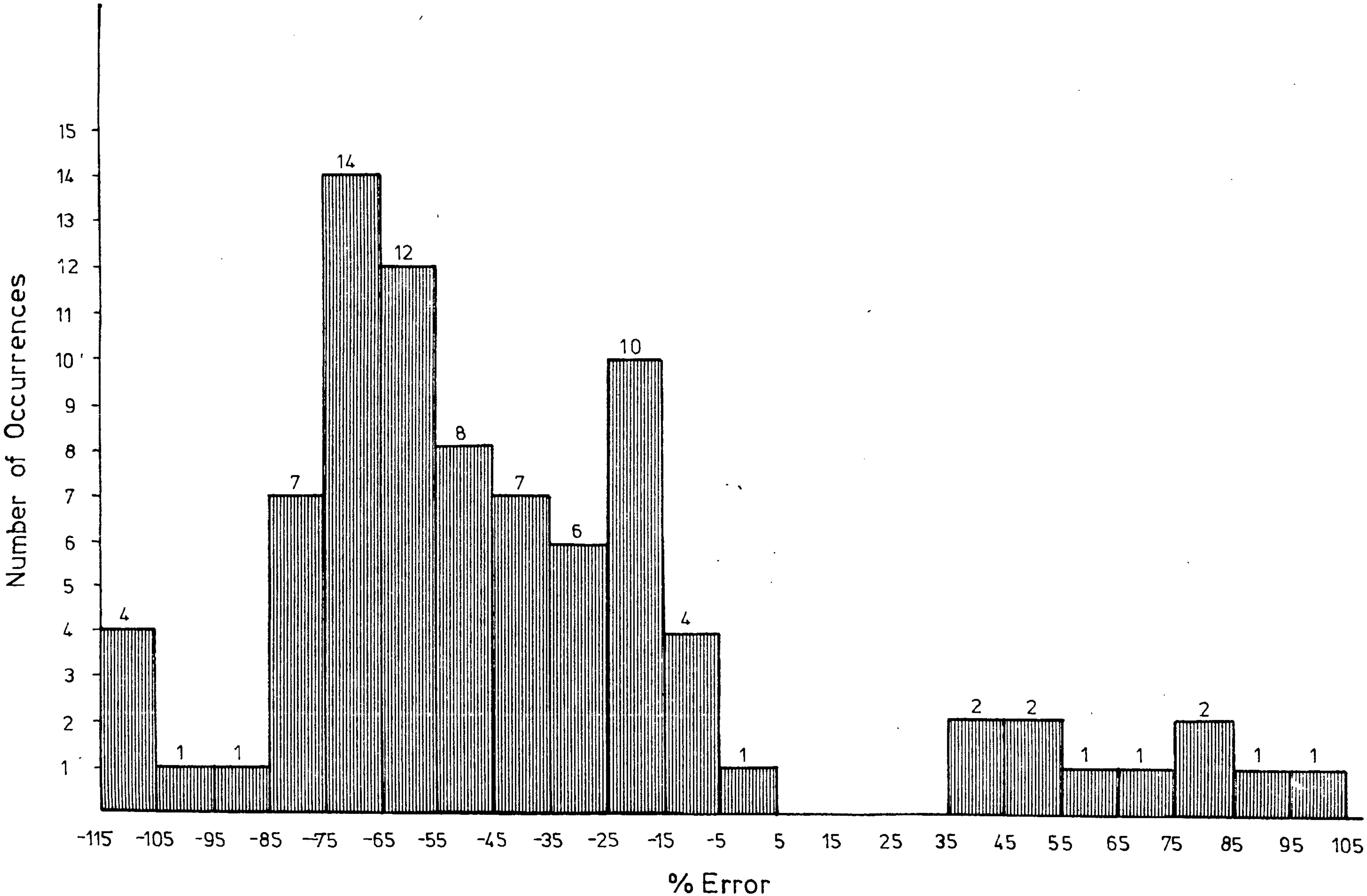


Figure 3.6 Niven's Drift Bridge - Distribution of % Error of Predictions

Discussion of Results

On the basis of only two case studies no truly meaningful discussion is possible but the following points can be made.

i) The Inglis formula gives surprisingly good results (refer to previous discussion on this formula in Chapter 2).

ii) The combination achieving the best result was the general scour equation of Herbich and Brennan III linked to Larras' local scour equation. A number of the local scour equations in fact when linked with the Herbich and Brennan equations gave reasonably accurate results.

iii) A glance at Figures 3.5 and 3.6 raises several questions regarding differences between them

a) Figure 3.6 has considerably greater dispersion - from 115% to + 105 % (220%) compared to the dispersion in Figure 3.5, from - 85 % to + 55 % (140 %). It must be remembered though that a one metre prediction error of the actual scour depth is represented in Figure 3.6 by roughly 25 % Error, whereas an equal error of one metre would be represented by only a 7½% Error in Figure 3.5 where the actual scour depth was 13,5 m.

b) In Figure 3.5 the median value of the results is very close to the actual scour depth but in Figure 3.6 the median value shows a negative error of approximately 50%.

c) The results of Case Study 1 show that 51 out of 85 predictions overestimate the actual scour whereas the comparable figure for Case Study 2 is 8 out of 85.

These factors, and other differences in the shape of the two distributions, must be related to physical factors, such as bridge and waterway geometry or sediment size, in a manner that as yet cannot be explained.

iv) The following Table can be drawn up giving, in real terms, some of the details for the two case studies.

	Case Study 1	Case Study 2
Actual Scour Depth (m)	13,5	4,2
Mean of all predicted values (m)	13,1	2,2
Minimum value predicted (m)	0,65	- 0,46
Maximum value predicted (m)	20,10	8,30
Dispersion = Max - Min (m)	19,45	8,76

CHAPTER 4

Designing to Minimise Scour

It is apparent from the great number of methods that have been proposed to predict scour depth that as yet none of these methods supplies a completely satisfactory solution to the problem. Establishing pier or embankment foundations below these predicted depths will not definitely insure against the bridge failing and being washed away. The use of these prediction methods, besides indicating a possible maximum scour depth also indicate the seriousness of the problem at the particular site and the need for further consideration of precautions that may be taken to protect the bridge in the event of a major flood. From the point of view of scour the following factors should be considered at any bridge site.

4.1 Bridge Location and Geometry

4.1.1 Bridge Location

A thorough site investigation may remove the scour problem altogether or confine it to a few piers by revealing solid foundation conditions at an alternative site.

In addition if it can be avoided bridges should not be located on river bends as the outer edge of the bend will scour at high stages. The sharper the bend the greater will be the scour and hence the undesirability of locating a bridge there. Writers of the regime school generally anticipate depths of up to twice the regime depth at the outer edge of sharp bends.

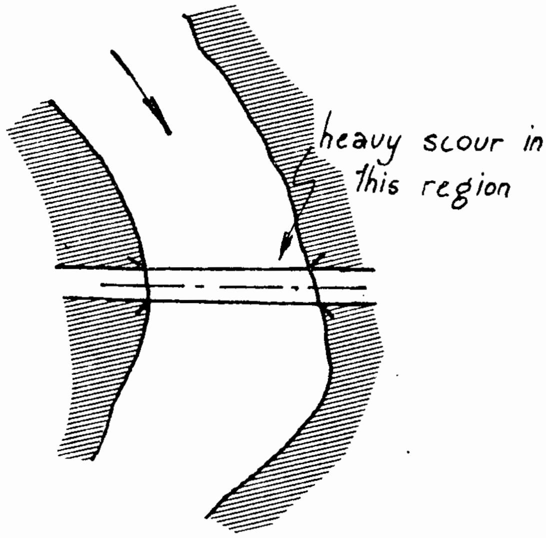


Figure 4.1 Locating Bridges on Bends

Similarly at bridges located at the junction of two converging streams scour 20% in excess of normal may be expected (27).

4.1.2 Angle of Flow Attack

Possibly the greatest single factor serving to aggravate scour problems is angularity of flow to pier and abutment alignment. There is no objection to skewed or curved bridges but the axes of piers and the faces of abutments should be aligned in the direction of the flow to minimise scour.

Figure 2.5 presented by Laursen, and Larras' Table 2.8 show the increase in scour with increasing angle of attack. P.L. Romita (66) has completed laboratory experiments on this problem and was able to draw up the following Table.

Table 4.1 Romita: Scour Depth and Flow Alignment Relationship

Angle of Flow Attack	Scour Depth Increase Relative to Scour at 0^{o}
0	1,0
15	1,9
30	2,6
45	3,0
60	3,3
60 - 90	3,3

Flow direction is often unpredictable and in many cases it may be advisable to consider upstream training works or spur dikes which will direct the flow uniformly. Piers then may be aligned with the flow with the assurance that flow will remain aligned at all flood stages.

4.1.3 Waterway Area

In their study reporting on the bridges damaged in the August 1955 floods in Connecticut, Moulton et al (51) described 'the inadequacy of waterway opening' as 'an important factor contributing to severe scour'.

The design rule generally followed is that the bridge opening should be adequate to maintain flow velocities below 1,5 m/s. If it is found necessary to increase the possible waterway area of the bridge it is preferable to increase the bridge span rather than the bridge height as scour increases with increasing flow depth. The data is not conclusive on this point and several researchers maintain flow depth does not affect scour depth.

Solid railings and utility pipes suspended beneath bridges may reduce the waterway area when it is most needed. They also tend to collect debris which may further reduce the waterway area. Bridge maintenance should include the removal of any debris that may impede the flow.

4.2 Pier and Abutment Geometry

4.2.1 Pier Width

All formulae predicting local scour stress the importance of pier width and show that the local scour depth is directly proportional to pier width. In addition general scour may be generated if piers constrict the waterway width by 10 percent or more. To minimise scour then pier width should be reduced as much as possible.

4.2.2 Pier Nose Shape

Table 2.8 produced by Larras and Table 2.2 by Laursen show how the shape of upstream face of the pier can encourage or

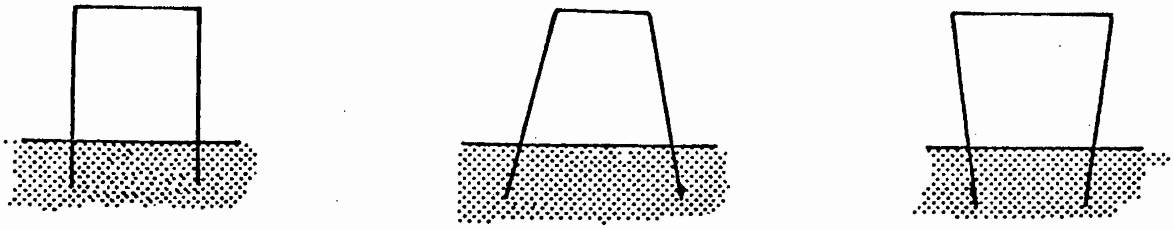


Figure 4.2 Rake Angles of Piers

Piers with negative rake more easily accumulate debris and should therefore be avoided. The negative rake also encourages downward movement of the flow and therefore will enhance local scour. Positively raked piers will have the opposite effect. The effect of pier rake on scour is small however.

4.2.5 Use of a Pier Lip

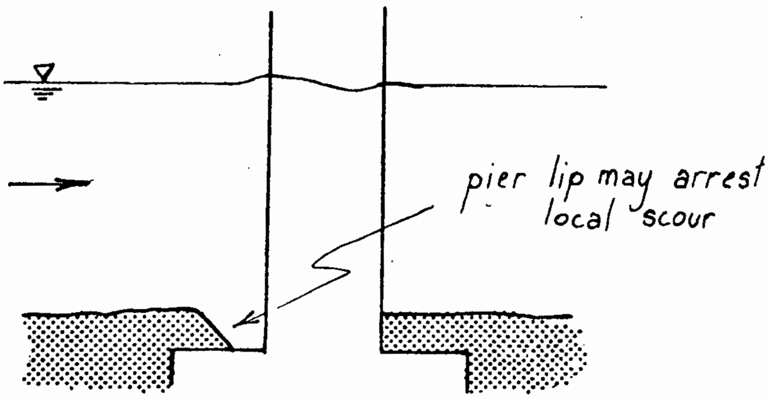


Figure 4.3 Pier Lip as Scour Arrester

A cheap and effective method of reducing local scour can be incorporated into the pier shape by the provision of a lip or 'scour arrester' near the base of the pier. The function of the lip is to divert the local downward current at the upstream face of the pier (or abutment) in order that it impinges less directly on the material around the base of the pier.

Several researchers have done work on this aspect of scour prevettion.

Shen and Schneider (70) found that with a rectangular pier on a footing supported by piles a lip on the footing at the optimum depth reduced scour by about 50 percent. If the lip is too high it loses its effectiveness but there is no danger in placing it too low. The size of the lip required is not specified.

Thomas (80) did a series of experiments to evaluate the effect of this type of shield using a cylindrical pier and circular shield. The arrangement used is shown in Figure 4.4 and the results in Table 4.2

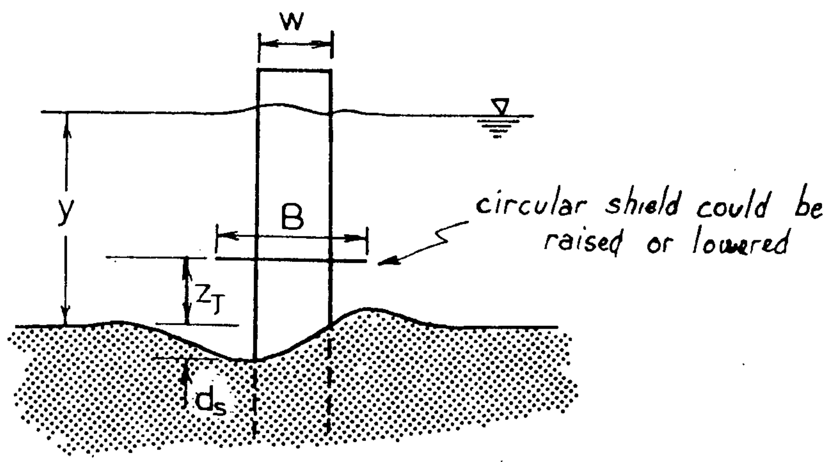


Figure 4.4 Thomas' Arrangement for Pier Lip Experiments

Table 4.2 Results of Thomas's Pier Lip Experiments

	$\frac{d_s(\text{lip})}{d_s(\text{no lip})}$ 100			
	z/y_o			
B/w	1,0	0,73	0,20	0,00
2	100	91,4	81,5	69,8
3	100	89,0	72,4	24,6

The figures given are the percentage scour that occurred for the given B/w and z/y_o values related to the scour that occurred when no shield was used. A graphical representation of these figures can be seen in Figure 4.5 below.

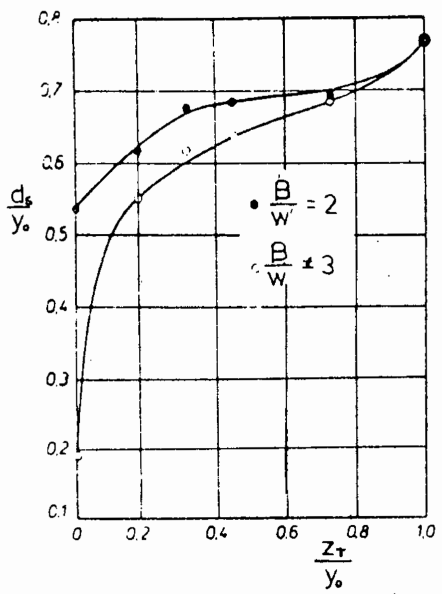


Figure 4.5 Graphical Results of Thomas Pier Lip Experiments

The graph shows that scour decreases as the shield increases in diameter and as it gets closer to the river bed. No combination of shield diameter and elevation causes an increase in scour depth.

No experiments were done by Thomas with the shield buried in the river bed.

Laursen (43) published some of the results of experimental work done by D.E. Schneible in this field. Schneible used plates at the base of the pier and determined the area required necessary to arrest scour. The plates were buried at different depths in each experiment, as shown in Figure 4.6 below, and the area of plate exposed by scouring action measured.

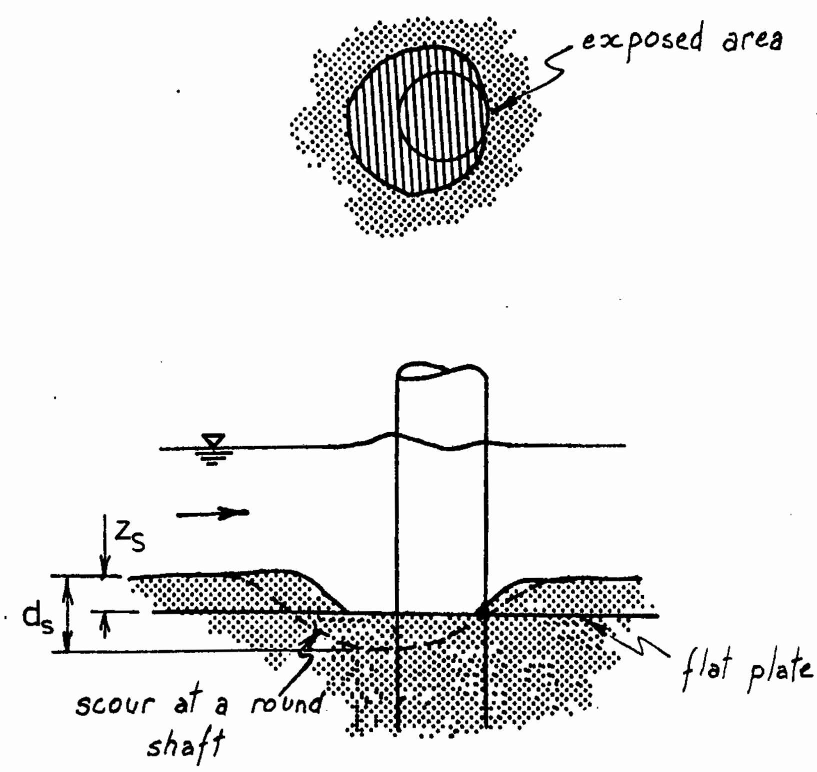


Figure 4.6 Sketch of Schneible's Plate Scour Arrester Experiments

The results are shown below in Table 4.3.

Table 4.3 Schneible's Scour Arrester Experimental Results

Relative Elevation of Flat Plate $= \frac{z_s}{d_s}$	Relative Area of Plate Exposed
0,18	21
0,37	14
0,56	6
0,74	5
1,00	1

The elevation of the flat plate is relative to the depth of scour at a single round shaft (0,061 m diameter). The exposed area is relative to the area of a single round shaft.

A series of experiments similar to those done by Thomas in which a disc of variable diameter which could be fixed around the pier at different elevations, as illustrated in Figure 4.7 and the scour measured for each variation were also done.

Table 4.4 below gives the results. The experiments are complementary to those of Thomas in that the disc is set at or below the stream bed. Thomas set the disc above the bed.

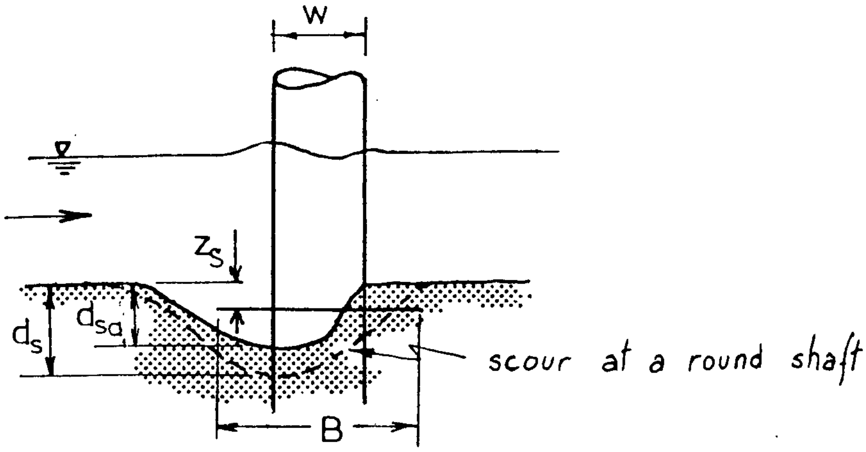


Figure 4.7 Sketch of Schneible's Disc Scour Arrester Experiments

Table 4.4 Schneible's Experimental Results Using Disc as Scour Arrester

Relative Elevation of Disc $= \frac{z_s}{d_s}$	Relative Diameter (B/w)	Relative Scour Depth $= \frac{d_{sa}}{d_s}$
0,18	2,0	0,85
0,37	2,0	0,70
0,56	2,0	0,59
0,37	2,0	0,59
0,18	1,5	0,78
0	2,5	0,52

The elevation and the scour depth are relative to the scour depth at a single 0,061 m diameter round shaft. The diameter of the disc is given relative to the diameter of the shaft.

Schneible's experiments allow a loose formula to be derived that predicts the lateral extent of the scour arrester, x_a, necessary to arrest scour at depth d_a below the bed.

$$x_a = 1{,}375 \frac{d_s^2}{d_a} - 1{,}375\, d_a \qquad (96)$$

where d_s is the depth of scour that would occur without an arrester

In cases where the arrester used was too small, and scour caused undermining of the arrester, the final depth of scour never equalled or exceeded that which would have occurred with no arrester present.

4.2.6 Abutment Shape

The type of abutment used will affect the scour depth at the abutment base. Table 2.3 page (25) shows the coefficients derived by Laursen which describe this effect.

4.3 Anti-Scour Measures

4.3.1 The Use of Rip-Rap

Perhaps the most commonly used anti-scour measure is the placing of rip-rap in vulnerable areas where scour is expected to occur. Often its use is haphazard and bridges fail in spite of its provision. There are a few simple rules that should be followed when rip-rap is provided as a scour preventive measure.

a) Unless the use of rip-rap is envisioned on a very large scale it will be used to prevent local scour and not general scour. Therefore it should be placed below the level to which general scour might occur as otherwise undermining of the rip-rap from its outer edges will lead to its progressive failure.

b) The principal area of attack for local scour is at the base of the upstream face of the pier or abutment. This is where the rip-rap protection should be centred.

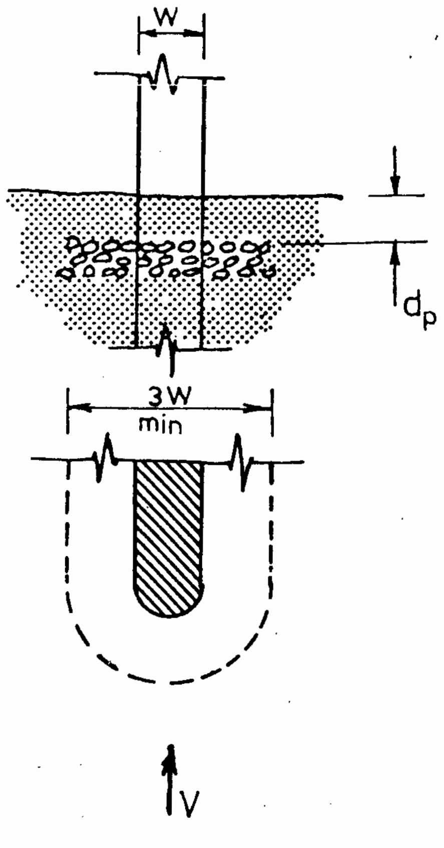


Figure 4.8 The Provision of Rip-rap

c) The size of rip-rap required to resist scour is govered by the flow velocity likely to be encountered at the bed. This velocity influences the hydrodynamic drag and lift forces that will be exerted on the rip-rap elements. Their stability is governed by their submerged weight and angle of repose. Formulae have been developed by the U.S. Army, the Californian Division of Highways, Isbash and others which can be used to estimate the minimum size of rip-rap necessary. In their analysis of these various formulae Stevens, Simons and Lewis (75) found that the highest factor of safety was obtained by the Isbash formula which is also the one recommended by the ASCE Task Committee on Preparation of Sedimentation Manual.

This formula for flow on a horizontal bed reduces to

$$D_r = \frac{0{,}347\ V^2}{(S_r - 1)g} \tag{97}$$

where D_r = diameter of rock in metres

V = flow velocity in metres per second

S_r = relative density of rock

g = gravitational constant (9,81 m/s^2)

This formula does not apply specifically to scour at bridge sites which may be caused by turbulence induced by piers as well as the local stream velocity.

Laursen gives the following formula for rip-rap size in clear-water scour.

$$V = 5{,}96\ D_r^{\frac{1}{3}}\ y^{\frac{1}{6}} \tag{98}$$

V = flow velocity in metres/sec

D_r = rock diameter in metres

y = depth in metres

From the results of a flume study Maza and Sanchez produced the following Table (70).

Table 4.5 Maza and Sanchez Rip-rap Requirements

Mean Flow Velocity (m/s)	Size of Rip-rap Required (m)
0,915	0,061
1,37	0,10
1,83	0,12
2,29	0,18
2,74	0,26
3,2	0,37
3,66	0,46
4,12	0,58

d) The rip-rap should extend at least one pier width from the pier. De Sousa, Pinto and Nelson gave the formula

$$\frac{x_a}{w} = 1,8 \frac{d_a - d_p}{w} \qquad (99)$$

where x_a = lateral extent of rip-rap

d_a = arrested scour depth

d_p = depth of protective layer below bed level

w = pier diameter

e) Maza and Sanchez stipulate that rip-rap should be placed to at least the thickness of the pier width dimension or three rip-rap diameters whichever dimension is the greater.

f) On no account should stone be piled around the base of a pier to protect it. The pile of stones effectively increases the pier width and will aggravate the scour problem.

4.3.2 The Terzaghi Vicksburg Inverted Filter (63)

Terzaghi originally proposed his successful inverted filter to prevent dam seepage causing leaching or the removal of fine material.

Experiments at the U.S. Waterways Experiment Station at Vicksburg, adopting the theory that local scour was the product of upward flow through the bed material created by the stream obstruction, did tests on inverted filters using them as a form of scour protection. They slightly altered the Terzaghi specification when they found that the following specifications produced a leach-proof filter.

$$D_r15 < 5\ D85$$

$$4\ D15 < D_r15 < 20\ D15$$

$$D_r50 < 25\ D50$$

where D_r15 is the diameter of particle in the upper protective layer of which 15 percent by weight is finer

D85 is the diameter of particle in the layer being protected of which 85 percent by weight is finer

D_r50, D15 and D50 can be similarly defined.

The filter increases upwards in particle diameter, layer upon layer, until at the surface the material is large enough to resist removal. Each layer can be the minimum thickness that can be placed with the equipment available as the absolute minimum thickness is probably less than a dozen times the maximum grain size.

Posey, who has written several papers about the filter, states that this method 'has been found not only to give complete protection to fine grained material in the bed of a flowing stream, but also to be more resistant to erosion than uniform material of comparable size'.

The filter suffers from the disadvantage, compared to rip-rap protection, that it requires more extensive preparation. However rip-rap failures have been known to occur due to excessive leaching of fine material between the rip-rap elements and this type of failure will be prevented if a Terzaghi-Vicksburg Filter is used.

Posey does not specify the size of material needed at the surface of the filter. Hallmark and Smith (23) did a study of sediment characteristics, and the forces that produce scour, in order to determine the characteristics of a sediment that would resist scour. They studied also the effectiveness of graded materials as an armourplate. The Terzaghi-Vicksburg filter was used. Sediment characteristics included in the study were particle size gradation, fall velocity of the sedimentary particle, shape factor of the particle and the size necessary to resist removal.

The following method may be used to determine the size of material necessary at the surface of the filter.

(i) Determine velocity at bed. If this velocity is assumed to be the average flow velocity it will allow a factor of safety.

(ii) Use Figure 4.9 below to determine D. Figure 4.9 can be entered on the left hand side using the flow velocity in m/s. D can be read off against the curve for $V = 0{,}227\ D^{\frac{1}{2}}$ which is drawn for values of relative density of rock $S_r = 2{,}65$; shape factor = 1 (spheres); particle drag coefficient = 0,42.

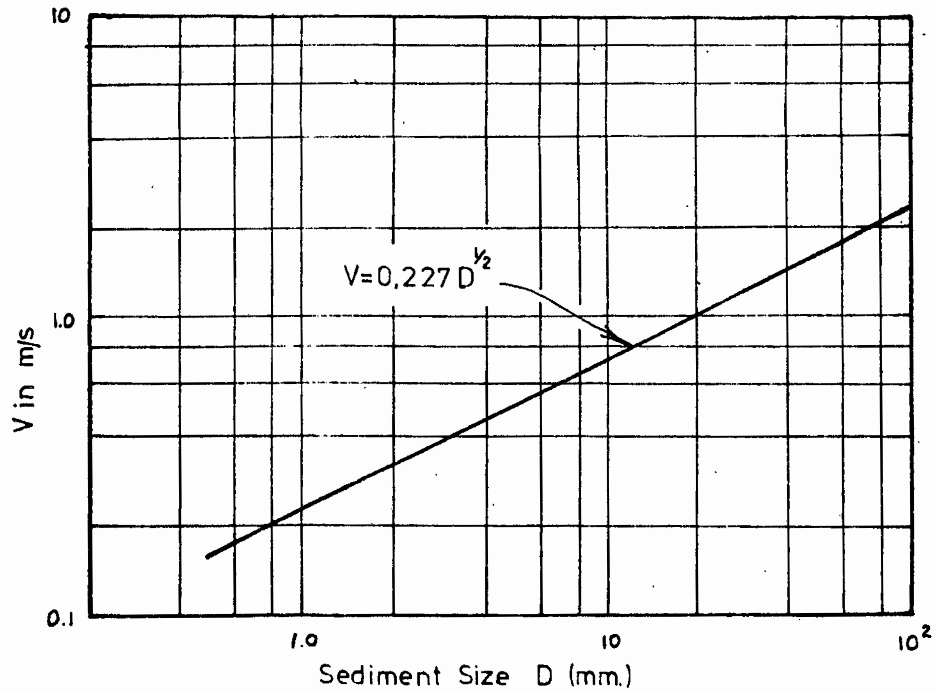


Figure 4.9 Sediment Diameter versus Fall Velocity
(Hallmark and Smith)

(iii) The minimum size necessary to be placed on the surface of the Terzaghi Vicksburg filter then is twice the diameter D found in (ii) above.

Hallmark and Smith suggest that the minimum depth of filter material be three times the diameter of the surface material or

$$\text{Minimum filter depth} = 3(2D) \qquad (100)$$

The area covered by the filter should at least equal the anticipated scour zone.

The principal advantage of this method over rip-rap protection is that leaching is prevented. Used on a small scale through the extensive preparations required to install the filter may make its cost comparable to concrete.

4.3.3 The Use of Protective Piles

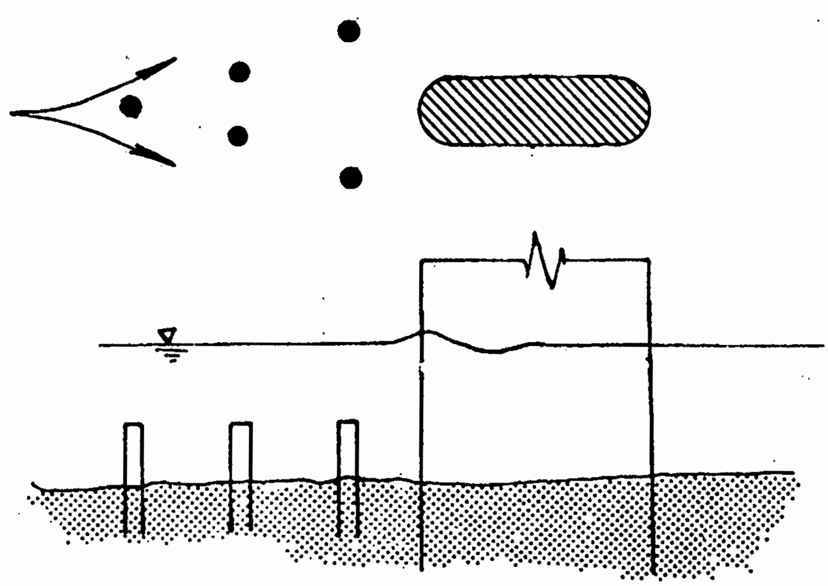


Figure 4.10 Using Piles to Reduce Scour

Piles can be placed on the stream bed upstream of bridge piers which reduce scour considerably by retarding the flow in the vicinity of the piers.

From their laboratory experiments Chang and Karim (Appendix A, Ref. 3) found that by using piles the scour at the pier was reduced by at least 20 percent. The reduction rate is dependent on the number of piles used and their arrangement. Generally a wedge-shaped arrangement is used.

4.3.4 Spur Dikes

Spur dikes are used principally to protect bridge abutments but also serve to control and direct the flow such that piers and abutments may be confidently aligned with the flow and scour reduced. The full waterway area of the bridge is then utilized.

Abutments will be protected as the main area of scour moves from the foot of the abutment to the nose of the spur dike.

The shape and size of spur dikes has not been precisely defined in the literature. Figure 4.11 below shows the type employed by the Mississippi State Highway Department which is credited with saving several bridges from destruction by scour.

4.3.5 Sheet Piling

While not inhibiting scour sheet piling will preserve pier and abutment foundations. Particularly on pier foundations a useful anti-scour measure may be incorporated into the design by allowing the sheet piling to project slightly above the footing and provide a lip as in Figure 4.12 below. The lip is useful in redirecting upwards any locally induced currents.

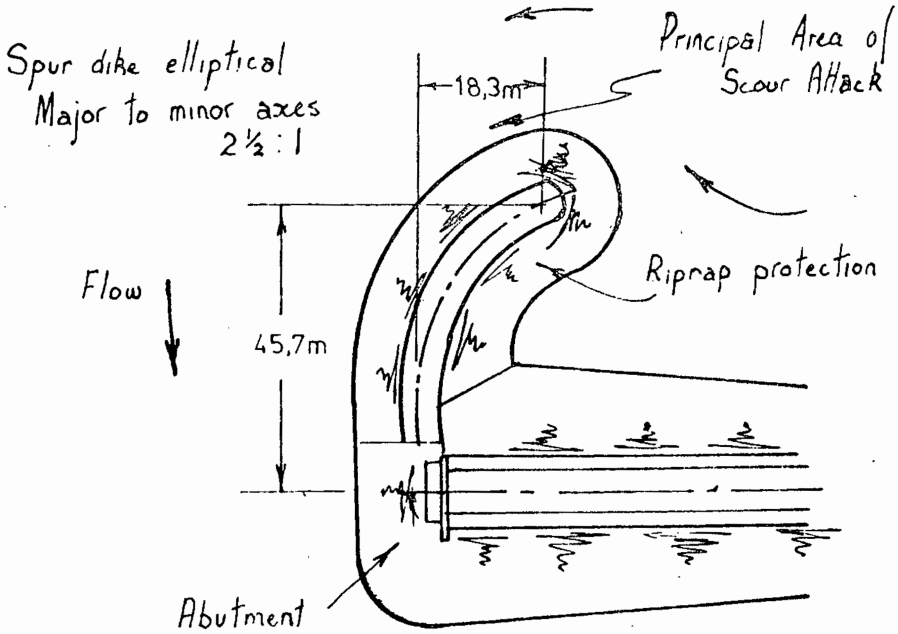


Figure 4.11 Spier Dike Detail

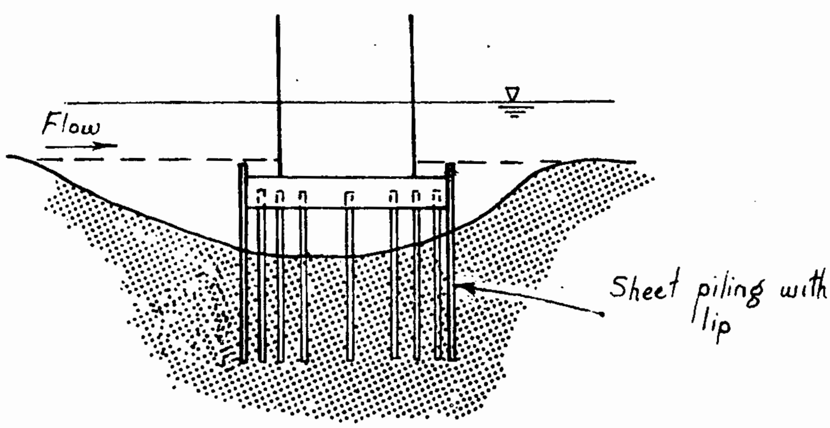


Figure 4.12 Sheet Piling Design

4.3.6 Gabions and Sand-filled Nylon Sausages

Gabions can be used very effectively to protect stream banks and abutments. The suppliers can provide designs of arrangements that have been used.

Posey (62) points out that it is often futile to dump rock over the edges of bridges in the hope that any scour holes will be filled. Instead he proposes that large sand-filled nylon sausages be used which will not wash away.

4.4 Foundations

Many of the points relating to pier geometry can also be applied to foundations. Foundations should present as small a width as possible to the flow, be aligned with the flow, and have a front face that is streamlined.

Foundations of the massive caisson type should not project above the bed level of the approach flow as large scale local scour may be generated.

It is worth noting in connection with piled foundations that if long piles are likely to be exposed by scour they must possess the structural stability to resist the stream forces under the scoured condition.

4.5 Construction Methods

Work carried out during construction of the bridge may stimulate scour at its piers and abutments.

Excessive excavation for pier and footing construction should be avoided. Similarly the area upstream of embankments should not be cleared as it will lead to an increased proportion of overbank flow in times of flood, and the abutments may be threatened by scour. The actual waterway area of the bridge should be maintained as obstruction free as possible though.

Excavations in the river bed downstream of the bridge are not desirable as they will in time lead to a wholesale lowering of the river bed upstream.

Any excess rock should not be dumped at the bases of piers in piles as the heaps of stone obstruct the flow and encourage local scour.

CHAPTER 5

The Need for Further Research

In 1894 H. Engels published the results of experiments that probably constituted the beginning of systematic research into scour. The experiments revealed that the principal scour area was at the upstream rather than downstream face of a pier. The fact that so simple a discovery should be made only after a great deal of money had been wasted protecting the downstream toes of piers demonstrates one of the most salient features connected with the scour problem - that it is almost impossible to observe under field conditions.

The introduction of laboratory research was, in a sense, a breakthrough in that scour could be made to occur under controlled conditions and observed and measured but in the eighty years and more since these first laboratory experiments not a great deal has been added to our knowledge and scour depth can still not be predicted with any confidence. Some of the factors that influence scour are known and these include flow velocity, pier and bridge geometry. Other factors are stressed by some investigators and ignored by others and these include upstream Froude number, pier Reynolds number, bed sediment size and grading, depth of flow, bed slope, and others. Principally because the means of scaling from laboratory experiments to prototype conditions do not as yet exist the value of laboratory experiments is limited.

The primary need in research is therefore the collection of prototype scour data. In order that it may be compared with the laboratory data and, if possible, a relationship derived between the two.

Collecting data from actual bridge sites is not easy mainly because it is desireable to monitor a bridge at the time of a large flood which may not occur in the lifetime of an investigator let alone in the relatively short period that is desireable in an investigation. An investigation therefore intending to monitor a bridge in the time of say a fifty year flood would require that a large number of widely dispersed bridge sites be monitored in order to produce any results within a reasonable period. The literature does not reveal that any investigation of this sort has been attempted.

Laursen and Hubbard monitored a bridge on the Skunk River continuously for over a year. A 1:12 model of the bridge was built and model to prototype comparisons were made. It was found in this case that the scour depth relationship was approximately related to the geometric scale ratio. Data from other sites is necessary before this role can be generally applied however.

This type of investigation is extremely useful but requires careful selection of the bridge site. The site should be chosen for its simplicity. The Skunk River bridge site possessed the following characteristics: a single pier centrally placed in the river and aligned with the flow, a straight uniform river approach with steep sides, sandy bed material extending beyond the probable scour depth; and a nearby recording guage. Shen in addition adds that there should be no riprap or armouring effect at the site and that the pier nose should be 'blunt'. Until experience has been accumulated only geometrically simple bridge sites should be chosen and in this way the study can be confined to local scour. Until local scour can be adequately described to solve problems involving both general and local scour will be extremely difficult. For this reason at the bridge sites initially a flow contraction in excess of ten percent due to either abutments or piers is not desirable.

Once a suitable bridge has been selected the taking of measurements can commence. Site, including slope, bridge and pier geometry can be easily obtained. Flow magnitude is recorded on the nearby recording gauge and flow velocity should be measured just upstream of the bridge but outside the zone of local turbulence caused by the piers. The larger distance of the following is recommended - either three to four pier widths upstream or one pier width plus two scour depths. The water temperature should also be monitored. Measurements of flow depth and direction are necessary. An analysis of the bed material will give the median size and grading. The material transported by the flow should also be analysed.

Finally, of course the scour depth sould be monitored. A few methods have been developed for doing this including electrical (Using the principle that the resistance between two electrodes is dependent on the material between them) and acoustical (the elapsed time between transmission and reception of a signal is converted into depth) methods have been used.

Any instrumentation at the site should not depend on electricity as a power source, but be independent, as the electricity will most likely fail at the time of a flood when the instruments are most needed.

A second type of field investigation has been undertaken by P.S. Holmes in New Zealand and consists of the examination of bridges that have failed due to scour. Though a lot of useful information can be extracted from this type of investigation, it is the author's experience that the data necessary to do it is often difficult or impossible to obtain. The data must be assumed or gaps left in the analysis. However, it is usually possible to determine the size of the flood, the bridge and pier geometry; at a site visit the river bed slope can be measured and sediment samples taken away for analysis for size and grading. The foundation depths of the failed piers can be read off plans of the bridge. Unfortunately the exact depth of scour cannot be found but can be presumed to be in the vicinity of the depths of the foundations. If neighbouring piers did not fail this may give some clue as to the scour depth. P.S. Holmes used the following convention in cases where the scour depths had not been measured and recorded: 'Where footings were completely washed out, scour down to 0,5 m below founding level was generally assumed. At piled piers, scour depths were assessed as follows: to the point of an individual pile when this pile, being one of a group, was known to have been loose; to an adjudged distance above the longest pile in a group where a whole pier was known to have been unstable or sunk; to a short distance above the point of the longest pile when a single pier washed out; to the point of the longest pile when several adjacent piers were washed out'. Cases where the scour depth has been measured and recorded should be regarded with suspicion until the actual method of measurement is known. Scour depth measurement is extremely difficult as the scour hole will refill as the flood recedes and any measurement taken after the flood will not reflect the true scour depth.

In cases where the size of the flood has not been recorded it may be possible to determine its magnitude if the maximum flood level is known and the bed slope and roughness measured. The velocity can be tentatively calculated using the Manning formula or if the size of flood and maximum flood level are known by calculating the waterway area.

Holmes' analysis led to the development of a formula to predict scour. In this thesis no formula has been put forward but instead a comparison of the probable scour depth with the depths calculated using the various formulae.

A third type of field investigation may be possible on bridges downstream of dams where the flow in the river can be controlled and perhaps even floods allowed to occur. In this way a full size model under controlled conditions could be studied.

Laboratory Studies

A great deal of laboratory work has been done which will become more useful once the model to prototype scale relationship is known. Laboratory work in this respect should be in association with field studies.

Further laboratory work to follow up that done by Z. Thomas, D.E. Schneible and Shen and Schneider on the effect of the anti-scour lip at pier footings, would be useful. Very little work appears to have been done on scour in cohesive materials.

Another aspect of the scour problem that does not appear to have been adequately researched is consideration of the flood hydrograph shape. Unless limiting scour is reached very quickly the duration of the maximum flood size will be a factor that may dictate the final depth of scour.

In the research of the scour problem it is now apparent, however, that laboratory studies have only a limited value and resources should now primarily be applied to the collection of data from the field.

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APPENDIX A List of Unobtainable References

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APPENDIX B - A List of Bridge Scour Failures in South Africa

1. Boy Retief Bridge - Great Fish River Mouth - Cape

2. Jack Hattingh Bridge - Nahoon River, East London - Cape

3. Illovo River Bridge - National Road Bridge near the river mouth - Natal
The bridge was founded on caissons taken down to 9,76 m below bed level - but not to rock. Scour occurred under one caisson which sank 2,7 m and tilted. The flood level was 1,22 m below girder level.
Another report (Civil Engineering in South Africa, Dec 1962) says that the caisson for pier number 3 was founded at 10,67 m below bed level and scour effects reached a depth of 11,28 m at this point.

4. Ixopo River Bridge - Road Bridge on Richmond-Highflats Road - Natal
Two spans, one pier and an abutment collapsed.

5. Niven's Drift Bridge - Zwartkops River near Uitenhage - Cape

6. Pondoland Bridge - Umzimvubu River approximately 4 km from river mouth - Natal
The flood overtopped the bridge road level by 0,61 m at its peak. This road level is itself 7,6 m above average river bed level.
The bridge consisted of ten 22,8 m spans with piers founded on concrete caissons. Failures occurred at the fourth pier from the left bank. The deepest foundation level for the caissons was 27,5 m below normal river bed level.

7. Punzi Bridge - Umtamvuna River - Road bridge on Izoingoleweni-Bizana Road. One concrete pier and two spans were swept away.

8. Umzimhlava River (tributary of Umzimvubu River). Low-level concrete bridge in Flagstaff district. The river was 7,6 m above normal.

9. Umzimkulu River - Combined road and rail bridge at the river mouth. The bridge was undamaged but scour occurred to 9,15 m which was enough to expose the rock on which the caissons were founded.

APPENDIX C Examinations Written to Complete the Requirements of the Degree

Examination		Credit Rating
CE 513	Wastewater Treatment	10
CE 522	Aquatic Chemistry	10
CE 504	Probability and Engineering Statistics	4
CE 525	Coastal Engineering	5
CE 526	Coastal Engineering Practice	5
Thesis:	Scour at Bridge Piers and Abutments	10

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

UNIVERSITY EXAMINATION - JUNE, 1975

CE 513 - WASTEWATER TREATMENT

Q.1

(a) An activated sludge plant designed for nitrification and denitrification has been constructed. At the time in question only the reactors are complete. The volume of the main aeration reactor is 15 000 m^3. Eight mechanical aerators have been supplied with a guaranteed oxygen transfer rate of 2,4 kg O_2 $(kWh)^{-1}$ under standard conditions.

An unsteady state test was carried out to determine whether the manufacturers' guarantee figure is acceptable. On the day of the test the saturation concentration of oxygen in the clean tap water contained in the aeration basin was 9,2 mg ℓ^{-1}. Atmospheric pressure was 760 mm Hg. Average power drawn by each aerator was 198 kW. As a result of the test (which lasted about 4 minutes), the aerators were accepted. The actual oxygen transfer rate differed from the guarantee figure by a factor of 1,03.

Reconstitute the table showing the results likely to have been obtained during the test i.e. of dissolved oxygen concentration (mg ℓ^{-1}) with time (minutes).

(b) On completion of the reactors the designers are now faced with a problem: should they install a conventional secondary settler or a dissolved-air pressure flotation system.

(i) Given the following information, produce designs for the secondary settler(s) and as an alternative a flotation system. Sketch the two designs to scale on graph paper.

Design for solid-liquid separation based on Peak Wet Weather Flow = 3 * Mean Dry Weather Flow.

Mean Dry Weather Flow = 28 000 m^3 d^{-1}

Peak Dry Weather Flow = 2 * Mean Dry Weather Flow.

MLTSS concentration = 3 000 mg ℓ^{-1}.

(ii) For the flotation system, what features in design would you suggest, bearing in mind that it is to take the place of secondary settlers.

(iii) What concentrations (approximately) would you expect to achieve for the separated solids for both settling and flotation.

Q.1(iv)/page 2.

CE 513 - WASTEWATER TREATMENT

Q.1

(iv) If the sludge wasted per day from the plant is to be concentrated to say 60 000 mg ℓ^{-1} by flotation, discuss the advantages or disadvantages in withdrawing sludge from (1) the underflow from the secondary settler, or the float from the flotation unit and (2) the mixed liquor from the reactors.

(v) Discuss the factors you feel may influence the designer in his choice as to whether secondary settlers or a flotation system should be installed.

(c) The waste sludge from the activated sludge system is to be thickened to a concentration of 6%. Given that the sludge age of the plant is 15 days, design a dissolved-air (pressure) flotation system to achieve this concentration. Sketch your design, to scale, on graph paper.

Are there any design features in this application (i.e. thickening) which differ from those for the flotation system you designed to take the place of the secondary settler (i.e. clarification).

Q.2 An activated sludge plant is to be built for a town with a present population of 10 000 inhabitants in the Karoo. The population is projected to increase to 15 000 in 10 years. There is no significant wastewater contribution from industry. The water temperature ranges from 14°C in winter to 21°C in summer. Present water consumption figures are 140 ℓ/cap/day, but the figure is expected to increase to 160 ℓ/cap/day in 10 years. COD contribution is approximately 0,1 kg/cap/day.

It is required that the phospherous and nitrogen content of the waste flow be removed as much as possible by biological treatment methods requiring no addition of chemicals.

You wish to investigate the use of a completely mixed activated sludge system, designed for nitrogen and phospherous removal, to treat the raw sewage inflow. Furthermore, the climate is suitable for drying the sludge in drying beds, and you consider the sludge will be suitable for this purpose if the sludge age is maintained at 40 days.

Design and compare the biological processes of two proposed systems:-

(i) A system with a sludge age of 40 days.

(ii) A system with a sludge age between 10 and 20 days plus aerobic digestion of the wasted sludge. (You must justify the choice of your specific sludge age).

Q.2/page 3.

CE 513 - WASTEWATER TREATMENT

Q.2

The comparison must include the removal of phospherous. It is proposed that the waste sludge from the reactor will be thickened to 3% by flotation before any further treatment. You are not required to design the flotation system.

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

COURSE CE 522 : AQUATIC CHEMISTRY : JULY EXAMINATION

M.Sc. IN ENGINEERING

ANSWER ALL QUESTIONS

NOTE: You are requested to answer Questions 1 to 5 and Questions 6 to 10 in separate examination books.

Q.1 An underground water source has the following characteristics when measured

Temp 20°C, Ionic Strength = 0,01 and Ca = 280, Alk = 120, Mg = 40 mg/l as $CaCO_3$, pH = 7,5.

(a) Is this water under or oversaturated with respect to $CaCO_3$?

(b) If the water is pumped to an open reservoir and mixed to attain equilibrium with the atmosphere, determine the mass of $CaCO_3$ which should eventually precipitate and the final condition of the water.

(c) If the water is treated directly from the borehole, and it is required to remove the Mg to 6 ppm as $CaCO_3$ by adding $Ca(OH)_2$ determine the dosage to remove the Mg and give the condition of the water after the dosage.

(d) If the calcium is to be removed to 60 ppm as $CaCO_3$, determine the mass of soda ash (as $CaCO_3$) to be added.

Q.2 0,1 moles of $NaHCO_3$ is added to one litre of CO_2 free water at 20°C, ionic strength approximately 0,01

(a) Find or determine the pK_w, pK_1 and pK_2 values and using the species-pH diagram graphically determine the pH of the water.

(b) 0,05 moles of HCl is now added to the solution. What is the pH, alkalinity, acidity and total carbonic species concentration of the water after dosing?

(c) 0,075 moles NaOH is now added to the solution in (b) above. What is the final state of the water?

Q.3 In an experiment to grow algae in a laboratory the alkalinity of the feed is 300 mg/l as $CaCO_3$. The following observations are made on the culture:

The algal growth rate is higher at pH 7 than at pH 8,5; the alkalinity does not appear to change during algal growth; the pH rises with time during algal growth.

(a) What hypothesis can you form regarding the utilization of the carbonic species by the algae?

University Examination : July 1974

Q.3 Continued....

(b) Devise an experiment which in theory should confirm or refute your hypothesis.

Q.4 A chemically treated effluent from a sewage works has an alkalinity = 65, Ca = 10 (both as $CaCO_3$) pH = 9,1, and ionic strength of 0,01, the temperature in the river is about 20°C.

(a) The effluent is discharged to a river which flows over sandstone. What is the expected final state of the water in the river?

(b) The effluent is discharged to a river which flows over pure $CaCO_3$ rock formations. What is the final state of the water in the river?

(c) The effluent is discharged to a river in which the rock formation is half Ca and half $MgCO_3$ as molar quantities. What is the final state of the water?

Q.5 A water sample has a pH of 5,8. After adding 80 ppm (as $CaCO_3$) of NaOH to the sample, the pH is 6,8. What is the alkalinity and acidity of the water?

If you had the opportunity of redoing this test, in what fashion would you change it to obtain more accurate results?

Q.6 (a) What is meant by the "thickness of the diffuse part of the electrical double layer"? Does the double layer, in fact, have a finite thickness?

How is the thickness of the double layer affected by the concentration of coagulant applied and the charge on the species yielded by the coagulant in solution?

(b) Into an aqueous dispersion of colloidal clay particles is introduced a quantity of metal coagulant, $Al_2(SO_4)$. $14H_2O$. Describe in detail the sequence of events taking place. Your answer should present the behaviour of the metal coagulant and the probably behaviour of the electrical double layer at any stage from the moment of coagulant addition to a condition of colloid destabilization.

(NOTE: Stern's model for the complete double layer should be used for your discussion.)

Q.7 (a) What general name may be given to the ligand formed by the weak acid ethylenediaminetetra-acetic acid (EDTA) in aqueous solution. Discuss why ligands of this type tend to form more stable complexes with metal ions than other types of ligand.

(b) (i) In the calcium hardness test, a metal-ion indicator is added and the sample titrated with a standard EDTA solution. On addition of the indicator (itself a chelating agent) relatively stable (Ca^{++}-indicator) complexes are formed which exhibit a colour (red) different to that of the <u>free</u> indicator (blue). With the foregoing information, discuss the effect of adding EDTA to the sample and the significance of the end point.

Page 3/(ii) Only....

University Examination : July 1974

Q.7 Continued....

(b) (ii) Only about 2 drops of indicator solution are added to the sample. Do you think the end point would be sharper or less easily determinable if more indicator is added? Give your reasons.

(iii) In some instances, when the magnesium content of the sample is low, the end point in the titration for calcium hardness is indistinct. Sharpness of the end point may be improved by the addition of magnesium to the sample. A common procedure is to add a small amount of magnesium chloride to the EDTA solution before it is standardized. Given the information below, explain the enhanced spontaneity of the colour change at the end point:

The magnesium-indicator complex is more stable than the calcium-indicator complex, but less stable than the magnesium-EDTA complex.

Q.8 (a) In the jar test, it is found that the optimum pH for destabilization of a water sample containing a clay dispersion is about 7,0. On addition of 450 mg/l of orthophosphate, it is subsequently found that the optimum pH is shifted to about 5,5. Can you explain this phenomenon?

(Note: phosphoric acid is a weak acid yielding predominant species

H_3PO_4, pH < 2; $H_2PO_4^-$, 2 < pH < 7; HPO_4^{2-}, 7 < pH < 11,5; PO_4^{3-} pH > 11,5.

(b) In the same water sample, the addition of 125 mg/l $SO_4^=$ spreads the optimum pH for destabilization from about 5 to 7. Explain this, given that $SO_4^=$ anions have a relatively high tendency to become hydrated.

Q.9 (a) If you wished to form an $Al(OH)_3$ precipitate, what would be the best pH to use? Having formed the precipitate, describe the effect of raising the pH further.

(b) Suggest a reason why, in destabilization with aluminium sulphate the optimum pH falls over a narrow range while ferric coagulants operate over a relatively wide pH range.

(c) In water softening where one wishes to precipitate $CaCO_3$ would you recommend the use of aluminium coagulants or iron coagulants to speed up the flocculation process? Give reasons for your choice. (Assume a "sweep-floc" type of mechanism".)

(d) (i) In the treatment of coloured waters with aluminium sulphate one may find an undesirable concentration of soluble aluminium compounds remaining in the water. Explain the above phenomenon in terms of the destabilization mechanism for coloured waters and the conditions lending to this mechanism.

(ii) What would be the most effective way of subsequently removing these compounds?

Page 4/Q.10(a) What....

University Examination : July 1974

Q.10 (a) What is the principal limitation in von Smoluchowski's treatment of orthokinetic flocculation?

(b) If one specified 99% flocculation efficiency, what would be the ratio of retention times required for a single tank, four tanks in series and a plug flow reactor.

(c) Besides the saving in total volume by having more than one flocculation tank in series, what other advantages can you suggest.

(d) What limitations can you suggest in the number of tanks comprising a flocculation system?

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UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

UNIVERSITY EXAMINATION, NOVEMBER 1974

CE 504: PROBABILITY AND STATISTICS FOR ENGINEERS

Total marks: 100 Time allowed: 3 hours

External Examiner : Professor D.M. Schultz

Internal Examiners: Professor G.v.R. Marais
Mr. M.S. Green

Attempt ALL questions in Section A and FOUR questions from Section B and C. Use separate answer book for Section C.

Name: ....

SECTION A

Note: Answer these questions in the spaces provided on this question paper. Do not show calculations, enter only the final answer.

1. Give formulae for:

(a) the coefficient of variation;

(b) the mean deviation about the mode.

2. Find the standard deviation of the data: 2; 6; 10.

3. In a particular experiment the result of 10 weighings showed 4 values between 20 and 25 g, 4 values between 25 and 30 g and 2 values between 30 and 35 g. What was the median weight?

4. An engineering firm has 100 electrical components in stock, 25 manufactured by process A and 75 manufactured by process B. Unknown to the firm, 13 of those manufactured by A are defective and 18 of those manufactured by B are defective.

A component is chosen at random from the 100 components. What is the probability that this component is:

(i) manufactured by B and defective?

(ii) either manufactured by A or is a defective component?

5. An item of radar equipment has three critical components A, B, C. The frequency of defect for component A was found to be 5 per 100, for B to be 6 per 100, and for C to be 8 per 100. Estimate the probability that a given item of equipment is defective.

6. A biased coin which has twice the probability of falling heads as falling tails is tossed with two unbiased coins. What is the probability:

(i) of at least two heads occurring?

(ii) of no heads occurring?

7. At a telephone exchange the average number of calls passed per hour in the morning is 96 and the rate can be regarded as constant. Calculate the probability of:

(i) exactly 3 calls in a period of 5 minutes;

(ii) more than 3 calls in a period of 5 minutes.

8. Packets are filled automatically and on the average, 5 per cent are underweight. An inspector takes a batch of twelve collected randomly. What is the probability that he will find 25 per cent or more underweight?

9. The mean diameter of steel rods produced by a process is 2 cm and the standard deviation is 0,05 cm. Assuming the diameters are normally distributed, find the value such that only 5 per cent of the rods will have a diameter exceeding this value.

10. A sample of 11 lengths of plastic were tested and found to have a standard deviation of 35. A second sample of 9 lengths of plastic, treated by a different process, was tested and found to have a standard deviation of 20. Test whether the standard deviations differ significantly.

11. State the assumptions required for the use of the t-test for the difference between the means of two independent samples.

12. If one denotes by y' the values of y which are calculated by means of the equation of the regression line, what is the least squares criterion?

13. Give the formula for the variance of the mean value of y, that is $\bar{y}$, where y is estimated from a regression line.

14. Give the formula for the correlation coefficient for two variables x and y.

15.

	Defective	Good	Total
Process A	25	15	40
Process B	35	25	60
Total	60	40	100

Test whether there is a difference between process A and process B in the above table.

SECTION B

Answer these questions in the answer books provided

1. A laboratory balance is used to weigh the same object 100 times. The values are given in the table below.

Weight in g.	Number of observations
4,55 - 4,65	10
4,65 - 4,75	20
4,75 - 4,85	45
4,85 - 4,95	15
4,95 - 5,05	10
	100

(a) Using this data calculate:

(i) the mean,
(ii) the mode,
(iii) the median,
(iv) the variance and standard deviation,
(v) the coefficient of variation.

(b) By fitting a normal distribution to the data, find the expected frequencies in the first two class intervals.

2. (a) Derive the binomial distribution from first principles and hence derive the Poisson distribution from the binomial distribution.

(b) The probability of a light bulb failing during the first twelve hours of service is 0,0049. If 1000 light bulbs are installed, use the Poisson distribution to find the probability of exactly ten bulbs failing within the first twelve hours.

(c) A machine is known to produce piston rings of which 10 per cent are defective. Find the probability that in a random sample of 400 rings:

(i) at most 35 rings will be defective;
(ii) between 35 and 50 will be defective.

3. (a) Define with diagram a type I error, type II error and the power of a test.

(b) The outputs from two production plants A and B were measured on each of 5 days. The data was given as follows:

Output (tons)

Plant A	Plant B
2,0	2,2
1,7	2,0
2,6	2,7
1,7	1,7
2,0	1,9

/Test whether .....

3. (b) (Continued)

Test whether the output of Plant B is significantly higher than that of Plant A at the 5 per cent level of significance if:

(i) sample A was considered to be independent of sample B:

(ii) it was believed that the day on which the observation was made was a relevant factor, and the observations were considered to be paired.

4. An experiment was carried out to measure the resistance of wire from three sources by taking five samples from each source.

(a) Use analysis of variance to determine whether or not there is a significant difference between the resistance of the wire from the three sources.

Sample \ Source	A	B	C
1	7,2	8,5	8,3
2	7,3	8,6	8,6
3	7,4	9,0	8,6
4	7,9	8,7	8,7
5	7,7	8,7	8,8

(b) It is believed that the 5 samples for each source were taken on consecutive days and that the resistance increased each day due to an external factor. Explain how you would test this hypothesis <u>for Source A only</u>.

SECTION C

(Answer this question in a separate answer book).

1. (a) In a set of 10 compressive tests on concrete cubes, two of the tests exceeded the capacity of the testing machine (12 MPa). The 8 definite test results were (in MPa):

11,9 8,0 8,8 11,3 10,7 10,7 9,9 9,7.

For the set of 10 cubes, determine graphically the mean compressive stress and its standard deviation.

(b) On two succeeding days a set of data was obtained on the concentration of bacteria in the effluent from a sewage works. The two sets of ranked data are:

<u>Set 1</u>:	400	700	850	1200	1900					
<u>Set 2</u>:	750	1200	1700	1900	2400	3100	3600	5000	6000	11000

The data is expected to be log-normally distributed.

/(i) Determine

1. (b) (Continued)

(i) Determine (using graphical procedures) the log-mean, geometric-mean of each set of data.

(ii) Test if the log-means are significantly different at 96 per cent level of significance.

(iii) Briefly explain why you performed the test for significance (in (ii) above), on the differences of the log-means and not on the differences of the geometric and arithmetic-means.

(c) List the conditions which must prevail for (i) a normal, (ii) a log-normal, distribution to arise.

M.Sc. examination CE 525
Feb.1976

3.(a). A storm at sea generates waves with a period range of 6 to 12 seconds. The resulting swell travels towards a harbour 400 km away. Estimate the time required for the longest waves to cover the intervening distance, assuming deep water throughout. Also estimate how much later the shortest waves will begin to arrive.

(b). A refraction diagram is constructed for a bay and the spacing between a particular pair of adjacent orthogonals doubles in travelling from deep water to the 10 m depth, the wave period being 7 seconds. Estimate the percentage change in wave height ocurring between these zones on the assumption that no breaking waves are present between the zones.

(c). Suggest some of the requirements you would incorporate into a specification for armour blocks.

4. The overleaf page shows the plan views of three separate coastal structures on which oblique waves impinge. In each case indicate areas where you consider deposition or erosion will occur, and also estimate the shape of the breaker line once stable conditions are established

5. There is a continuous dissipation of energy due to tidal movements of water over the earths surface, and in some instances useful power is abstracted from the sea in tidal power schemes.
Suggest what effect this may have on the dynamics of the earth-moon system over very long periods of time.

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

UNIVERSITY EXAMINATION NOVEMBER 1977

CE 526 COASTAL ENGINEERING PRACTICE

Time allowed: 2 hours

Answer ALL questions

OPEN BOOK

There is a potential of 142 marks
120 marks will be regarded as 100%

Section 1 is to be handed in at the end of the first hour

1. Answer all questions on the attached sheets, in the space provided. If additional space is required the answer is to be completed in an Examination Answer Book where the answer must be clearly numbered.

[64]

2. The attached plan shows the bathymetry of False Bay to M.S.L. Using this plan and annotating it if necessary, answer the following questions, stating all assumptions and sources of information.

If the wind were to blow from the North at an average speed of 100 km/hr estimate for a point in the vicinity of Whittle Rock:

i) The time taken to develop a fully arisen sea (2)

ii) The significant wave height: H_s (2)

iii) The significant wave period: T_s (2)

iv) The depth at which this wave would break (2)

[8]

3. If a wave recorder of the 'Wave Rider' type were to generate a record with the following characteristics:
Record length = 340 seconds
Number of 'zero-upcrossings' = 43
Number of crests = 104

i) Calculate the zero crossing period (1)
ii) Calculate the mean crest period (1)
iii) Calculate the spectral width parameter (1)
iv) What type of waves are these (i.e. swell, sea, mixed etc) Give your reason (1)
v) If the height of the highest crest in the record is 2,1 m * and the depth of the lowest trough is 1,9 m* calculate the value of H_s (significant wave height) (2) and H_{max} (the 6 hour maximum) (2)
Use the method proposed by L. Draper 1967 in his paper 'The Analysis and Presentation of Wave Data - A Plea for Uniformity'.

* *dimensioned from the zero crossing line*

[8]

4. A lake of area 4×10^6 m^2 is to be joined to the sea by a navigation channel with sides formed by vertical sheet piles driven into the sand bed. The tidal range is 1,8 m.

a) What dimension would you recommend for the width of the channel assuming a bed depth of 2 m below M.S.L. ? (4)

b) Estimate the average outflow velocity (2)

c) If the size and grading of the sand is typical of the Cape Flats at what velocity would you anticipate scour would commence ? (1)

[7]

5. Two vertical aerial photographs of the coast taken at 12 seconds apart are mounted in a viewer. Two adjacent wave crests (A and B) approaching a shallow shoreline are examined. In the first photograph the distacce between A and B is 127 m apart. In the second photograph the distance between A and B is 117 m apart. The second position of A is 84 m ahead of its position in the first photograph.

a) Estimate the average wave celerity of crest A and of crest B during the twelve second interval (2)

b) Assuming the water is effectively shallow, estimate the average water depth under each crest, and check that the assumption is valid. (3)

c) Calculate the wave period for each crest (2)

d) To what do you attribute the difference in period (2)

e) Have these waves been generated locally, or at a considerable distance (1)

[10]

6. a) Explain the term 'spectral window' as applied to electro-magnetic radiation in the region 0,2 to 20 micrometres (3)

b) Explain why colour-false infrared film is particularly suitable for demarcating the tide line in an estuary (3)

c) Explain what is meant by the term 'spectral signature' of a ground material such as sand or grass; and hence explain how a 'classification' of a set of multi-spectral images of a ground scene can be achieved (7)

[13]

7. The attached plan shows contours of the sea bed at the Strand, near Gordons Bay. It will be seen that rocks outcrop in many places and provide a relatively calm area which is considered to have some potential for a small craft harbour and in particular a boat ramp.

a) Outline briefly the investigations and work you would recommend to establish the feasibility of constructing a small craft harbour in this location (7)

b) Identify the personnel and equipment required to undertake each of the investigations outlined above in (a). Estimate the time, rates and hence the cost of undertaking this work. (7)

c) Draw on the plan provided the main features of a proposal to provide a small craft harbour at this location (10)

d) Identify the number of boats at moorings and in dryboat storage that can be accommodated (3)

e) Give a rough estimate of quantities for any harbour protection works (e.g. breakwaters) proposed. State any assumptions. (5)

[32]

QUESTION 1

Name ....

1.1 The optimum orientation for the mooring of sailing craft is

-- (1)

1.2 The economic advantage in providing locks in a tidal harbour is in respect of --

-- (1)

1.3 The optimum location for waterside fuelling facilities is

----

because -- (2)

1.4 Minimum dredged depths in a small craft harbour are the sum of individual depths allowed for the following

--------------------	typically	----------------------	m
--------------------		----------------------	m
--------------------		----------------------	m
--------------------		----------------------	m
--------------------		----------------------	m
--------------------		----------------------	m
--------------------		----------------------	m
--------------------		----------------------	m
--------------------		----------------------	m (3)

USE ADDITIONAL LINES IF REQUIRED

1.5 Detail in plan and dimension typical floating berthing to provide double occupancy for boats of length 8 m. Show the system of mooring proposed.

(3)

Assuming an average overall cost of R100/m^2 of floating berth deck area, estimate the cost of providing this berthing <u>per boat</u>

--------------------- m^2 @ R100/m = ---------------------

(2)

1.6 Give a local example of a leeshore anchorage ---------------------
--- (1)

1.7 Why is a leeshore disadvantageous to a harbour --------------------
--- (1)

1.8 Explain the significance of providing a turning basin in the harbour on the Buffalo River at East London ----------------------------
----
--- (2)

1.9 Explain briefly how the position of a dredger may be ascertained by using a sextant

(3)

1.10 Explain what is meant by the term 'controlling depth' of a harbour
----

--- (2)

1.11 Sketch the main elements of a float type tide recorder such as is installed at Hout Bay

(3)

1.12 Explain how a 'clinometer' is used for wave recording. Identify the main elements of the system ________________________________

____

____

____

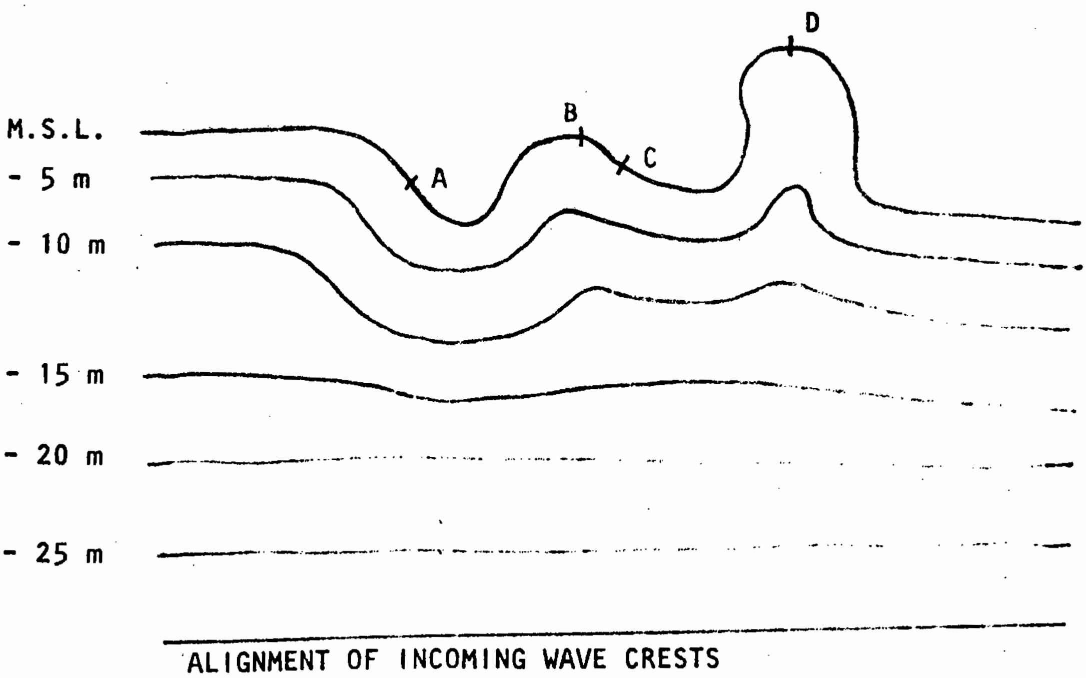
____ (3)

1.13 Tabulate the advantages and disadvantages of a 'clinometer' system as compared with a 'wave rider' system of wave measurement

Clinometer		Wave Rider	
Advantages	Disadvantages	Advantages	Disadvantages

(5)

1.14 Draw the approximate form of the wave orthogonals to reach A, B, C and D as they approach the coastline drawn in plan below. (Assume refraction without diffraction).



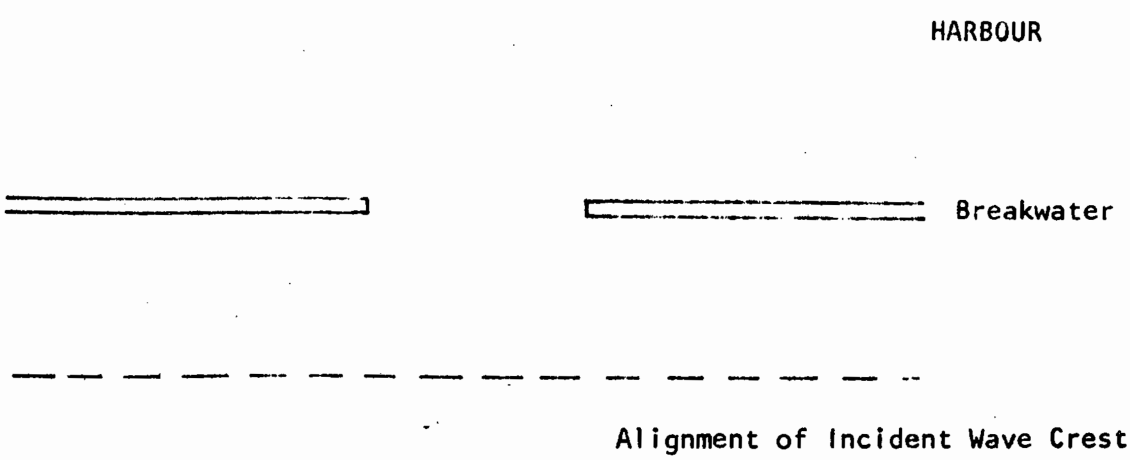
(4)

1.15 For the harbour entrance detailed below sketch

a) the approximate form of 4 wave orthogonals as they enter the harbour (2)

b) 3 wave crests (1)

c) If the gap width is equal to 1 wave length draw in a dotted line the location of diffracted wave heights of one half the incident wave height. (Refer to Fig. 2-44 in CERC Shore Protection Manual) (2)



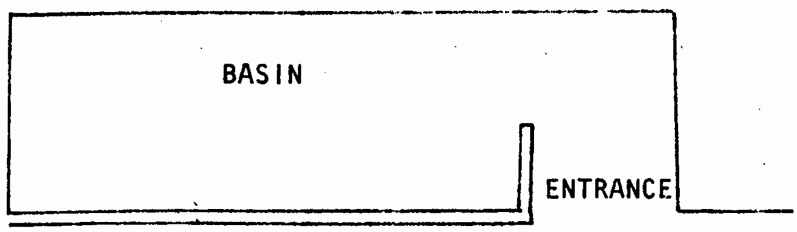
1.16 a) What is the significance of waves entering a harbour with a period equal to the fundamental period of oscillation of one of the basins ?

-- (1)

b) Name two design features that may be incorporated in a harbour design to reduce reflection ? __(1)______________________________

________________(2)____ (2)

c) Sketch where you would site the two features described above in 16b) in the basin shown below



(2)

d) Sketch sectional elevations of the two features described above

(2)

1.17 Give an example of a situation in which it would be appropriate to commission:

a) A 3-dimensional hydraulic model ________________________________

____

b) A 2-dimensional hydraulic model ________________________________

____

c) A mathematical model ____

____ (3)

1.18 Explain briefly the function of coastal sanddunes in maintaining the stability of a sandy coastline ________________________________

____

____

____

____

____

____ (3)

1.19 Identify by means of annotated sketches the procedures involved in implementing the following stages which might occur in the construction of a jetty on a rock bed covered in a thin layer of sand.

a) Temporary staging (2)

b) Airlift (2)

c) Placement of precast bases (2)

d) Placement of bearer piles (2)

e) Placement of concrete underwater (2)