



Committee of
State Road Authorities

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**GUIDELINES FOR THE HYDRAULIC
DESIGN AND MAINTENANCE OF
RIVER CROSSINGS**

**VOLUME I:
HYDRAULICS, HYDROLOGY
AND ECOLOGY**

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PREFACE

At the rate at which new information is generated and made available it is becoming increasingly difficult for the practising civil engineer to decide on the appropriate norms and analytical methods to be used in designs. Although there will always be cases necessitating a comprehensive independent literature study to ascertain the best suited norms and methods to achieve a sound solution, it is recognised that they tend to be the exception rather than the rule. The designer cannot be expected to undertake such detailed studies for each case as this would become impractical. Consequently the need for practical guidelines.

The main aims of these guidelines are to make recommendations on methods of calculation, design norms as well as legal and other issues which need to be taken into consideration in the pursuit of providing safe, economical and viable river crossings. The intention is not to stifle original thinking and new development, and thus designers are expected to deviate from the general recommendations where optimum solutions clearly fall outside the general applicable norms. The guidelines are furthermore intended to serve as a basis for governing bodies to formulate their policies on design standards with due consideration of legal and other risks.

These guidelines comprise seven volumes each dealing with a particular subject or related subjects.

SYNOPSIS

During the 1970s and 1980s a number of major floods caused serious damage to the road and rail infrastructure in southern Africa. These events prompted the re-evaluation of design norms to secure cost-effective and safe designs to withstand the expected imposed forces.

This volume provides guidelines on solving flood estimation problems, assessing the performance of a river crossing and the consequential flow patterns, precautionary measures against potential scour, evaluation of hydraulic forces resulting from flooding and wave actions, and the consequences of adverse ecological impact on the environment.

This volume should be read in conjunction with the SARB Road Drainage Manual.

Whilst this document should not be considered as comprehensive and all-inclusive, it presents the basic principles that need to be considered for sound design.

Keywords : Waterway hydraulics, flood estimation, sediment transport, hydraulic models, bridge design, bridge management, ecological impact, scour (hydraulic engineering)

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NOTATION

Symbols not included in, or varying from, those given in this list are of minor importance or are used in special cases only. Where they occur, they are adequately explained in the text.

		SI UNITS
a	Flow area for sub section	m^2
A	Sectional area/drainage area	m^2
A_c	Effective opening	m^2
A_1	Sectional area just upstream of the bridge	m^2
A_4	Sectional area some distance downstream of the bridge	m^2
A_n	Sectional area for normal depth of flow, y_n	m^2
A_{n2}	Sectional area at section 2 below the normal water level (with respect to flow through bridge openings)	m^2
$A_p \}$ $A_1 \}$	Area of piers in direction of flow ($= W_p \times y_n$)	m^2
A_s	Additional flow area created by scour	m^2
b	Stream width	m
b	Width of bridge pier	m
b	Effective width of abutment	m
b	Width of bridge opening	m
b	Overall width of bridge constriction normal to the flow	m
b	Width of leeward shore	m
b_o	Width of windward shore	m
b_s	Skew width of bridge constriction	m
b_N	Net width of bridge constriction excluding piers	m
b^*	True width of abutment	m
B	Top width of stream	m
B_n	Top width of stream for normal depth of flow	m
C	Chézy roughness coefficient	$m^{1/2}/s$

C	Correction factor for scour	-
C _A	Coefficient for Du Aubuisson's discharge formula	-
C _b	Backwater coefficient for Type II flow (constriction only)	-
C _b	Contraction loss coefficient (with respect to serious bridge damming)	-
C _c	Contraction coefficient	-
C _d	Coefficient of discharge for partially submerged bridge	-
C _d }	Drag coefficients	-
D _{dp} }		
C _{d1} }		
C _f	Discharge coefficient for flow over an embankment	-
C _f	Lift force coefficient	-
C _L	Transition loss coefficient	-
C _m	Rolling moment coefficient	-
C _s /C _f	Submergence factor for discharge over an embankment	-
C' ¹	Grain roughness coefficient	-
d	Depth of flow	m or mm
d	Rip-rap diameter	m
d ₅₀	Average particle size of sediment	m
d _o	Depth of flow upstream of obstruction (with respect to scour)	m
d _s	Depth of scour around abutments and bridge piers	m
D	Depth of flow	m
D _b	Differential water level ratio	-
e	Eccentricity of bridge opening (= $1 - \frac{Q_c}{Q_a}$ if $Q_a < Q_c$; = $1 - \frac{Q_a}{Q_c}$ if $Q_a > Q_c$)	

E	$(= y + \frac{v^2}{2g})$	-
E_{sn}	Specific energy for normal unconfined flow	-
E_{sc}	Specific energy for constricted section critical flow	-
f	Darcy-Weisbach roughness coefficient	-
F	Force	N
F	Blockage factor	-
F_d	Drag force per unit length of bridge	N/m
$\left. \begin{matrix} F_{d1} \\ F_{dp} \end{matrix} \right\}$	Horizontal force in line with pier axis	N
F_l	Lift force per unit length of bridge	N/m
Fr	Froude number $(= (\frac{Q^2 B}{gA^3})^{1/2} \text{ and } \frac{V}{\sqrt{gy}} \text{ for rectangular channel})$	-
Fr_n	Froude number for normal unconfined flow conditions	-
Fr_c	Critical Froude number (with respect to initiation of sediment movement)	-
F_s	Horizontal hydrostatic force on pier	N
g	Acceleration of gravity (design value 9,8 m/s ²)	m/s ²
h	Height of obstruction in stream (with respect to scour at abutments)	m
h	Height of upstream water level above road embankment for overflow of embankment	m
h_f	Friction head loss	m
h_1	Transition head loss	m
h_1^*	Head upstream of a bridge	m
h_1^{*1}	Approximate backwater produced by the constriction based on the first term of Equation 3.1	m
h_{1A}^*	Backwater produced by the constriction under abnormal stage conditions	m
h_{1s}	Backwater produced by the constriction under scoured bed conditions	m

h_{1D}^*	Backwater due to the constriction excluding piers but including eccentricity and skew	m
h_3^*	Vertical distance from water surface on downstream side of embankment to normal water surface at section 3	m
H_{3B}^*	Change in water level produced by dual bridges at downstream embankment of downstream bridge	m
h_{3s}^*	As for h_3^* but under scoured bed conditions	m
h_b	Energy loss caused by constriction assuming no piers	m
h_d	Downstream water depth	m
h_d^*	Backwater produced by dual bridges	m
h_u	Wind set-up above static water level	m
h_u	Upstream water depth	m
H	Head	m
H	Downstream depth, measured above overflow crest	m
H	Total height of bridge superstructure normal to flow	m
Δh	Difference in water levels upstream and downstream of constriction	m
I	Rate of inflow	m ³ /s
I	Rainfall intensity/infiltration rate	mm/h
i	Hydraulic gradient	-
J	Area factor ($=A_p/A_n$)	-
k	Measure of absolute roughness	m
k	Permeability	m/day
$k_1 \}$ $k_2 \}$	Coefficients	-
k	Conveyance of sub section	m ³ /s
K	Total conveyance of section	m ³ /s
K	Secondary energy loss coefficient	-

K_b	Base backwater coefficient for a normal crossing without piers or eccentricity	-
K_D	Stability coefficient	-
K_s	Correction factor for different types of abutment	-
K^*	Secondary energy loss coefficient (with respect to Type I flow through bridge openings)	-
ΔK_e	Incremental backwater coefficient for eccentricity	-
ΔK_p	Incremental backwater coefficient for piers	-
ΔK_s	Incremental backwater coefficient for skew	-
l	Length of pier	m
L	Distance along direction of flow	m
L^*	Distance from position of water line on upstream embankment to position of maximum backwater	m
L	Length of overflow portion of embankment	m
L	Fetch length (for wind)	m
L	Wave length	m
L_{1-3}	Distance from section 1 to section 3 (Figure 3.2)	m
L_{1-3B}	Distance from section 1 to downstream embankment of downstream bridge for dual bridges	m
L_d	Distance between upstream face of first bridge and downstream face of second bridge (dual bridges)	m
M	Mass	kg
M	Flow factor (Q_b/Q)	-
M	Overturning moment	Nm
n	Manning roughness coefficient	$s/m^{1/3}$
n	Slope	m/m or %
n	Number of layers of armour units	-
n_b	Porosity	-

		SI UNITS
N	Total number of armour units	-
N	Plan form factor	-
p	Pressure intensity	N/m ² or Pa
p	Wetted perimeter	m
P	Tidal prism	m ³
q	Flow rate per unit width	m ³ /s.m
Q	Flow rate	m ³ /s
Q _a } Q _c }	Flow over that portion of the natural flood plain obstructed by the embankment (Figure 3.2)	m ³ /s
Q _b	Portion of the normal flow spanned by the bridge opening	m ³ /s
Q _{max}	Maximum flow rate	m ³ /s
Q _r	Additional river flow rate	m ³ /s
r	Radius	m
r	Model to prototype ratio	-
R	Hydraulic radius (= A/P)	m
R _e	Reynolds number	-
S } S _f } S _h }	Energy gradient	-
S	Spacing of laterals	m
S	Water surface slope	-
S _o	Bed slope of river	-
t	Time	s
T	Tidal period	s
T	Wave period	s

SI UNITS

U	Wind speed	m/s
U_a	Wind stress factor	-
v	Flow velocity	m/s
$\bar{v}$	Average flow velocity	m/s
V	Volume	m ³
v	Constriction flow velocity for no scour	m/s
$V_1 \}$ $V_2 \}$	Flow velocities at sections 1 and 2	m/s
V_{2A}	Average flow velocity in constriction at section 2 under abnormal stage conditions	m/s
V_{2c}	Critical velocity in constriction at section 2 (with respect to hydraulics)	m/s
V_c	Critical flow velocity (with respect to initiation of sediment movement (See Fr_c))	m/s
V_i	Velocity for initiation of sediment motion	m/s
V_n	Average velocity for normal flow in the unconstricted river reach (= Q/A_n)	m/s
V_{n2}	Flow velocity at section 2 (= Q/A_{n2}) with respect to flow through bridge openings	m/s
V_s	Flow velocity near river bed (with respect to scour at abutments)	m/s
V_{ss}	Settling velocity of sediment particles	m/s
V_{*c}	Critical shear velocity (= $\sqrt{gd_o S_r}$) with respect to initiation of sediment movement	m/s
V_*	Shear velocity	m/s
W	Width	m
W	Top width of normal flow upstream of the constriction	m
W_p	Width of pier, perpendicular to flow direction	m

x	Distance along direction of flow	m
y	Depth of flow	m
y_c	Critical depth at which $Fr = 1$	m
y_1	Depth of flow at section 1	m
y_{2c}	Critical flow depth in constriction at section 2	m
y_3	Depth of flow at section 3	m
y_{3B}	Depth of flow at downstream embankment of downstream bridge (dual bridges)	m
$\bar{y}$	Normal depth of flow in contraction	m
y_J	Depth of flow downstream of hydraulic jump (excluding backwater) for Type IV flow	m
y_n	Normal depth of flow as calculated according to the Manning or Chézy equations	m
y_u	Vertical distance from upstream water surface to mean river bed at bridge	m
α	Velocity coefficient (value = 1,05 for main channels)	-
$\alpha_1 \}$ $\alpha_2 \}$	Velocity coefficients with respect to flow through bridge openings	-
α	Slope angle	-
α	Angle of approach with respect to damming upstream of bridges	-
β	Slope angle	-
β	Angle between direction of flow and centre line of abutment	-
Δ	Increment	-
γ	Specific weight (design value for water $9,8 \times 10^3 \text{ N/m}^3$)	N/m^3
γ_{fl}	Load factor	-
θ	Slope angle	-
λ	Darcy-Welsbach roughness coefficient	-
μ	Dynamic viscosity (design value for water 10^{-3} kg/m.s)	kg/m.s

SI UNITS

ν	Kinematic viscosity ($= \mu/\rho$) (design value for water 10^{-6} m ² /s)	m ² /s
ρ	Mass density (design value for water 10^3 kg/m ³)	kg/m ³
τ	Shear stress	N/m ² or Pa
ϕ	Pier shape factor	-
ϕ	Natural slope angle or angle of repose	-
ϕ	Skewness of bridge with respect to direction of flow	-
ω	Angular velocity	s ⁻¹
ω	Correction factor for eccentricity	-

1. GENERAL CONSIDERATIONS

1.1 Introduction

Our committee had as its task the development of new guidelines regarding the hydraulic, hydrological and ecological aspects of bridge design. The committee consisted of:

Prof A Rooseboom	(Chairman, Stellenbosch University)
Mr J Varkevisser	(Dept of Transport)
Dr G W Annandale	(Steffen Robertson & Kirsten)
Mr R Arnold	(Van Niekerk, Kleyn & Edwards)
Mr G I Orczy	(Transvaal Provincial Administration)
Prof G G S Pegram	(University of Natal)
Mr R Sutton	(Natal Provincial Administration)
Prof T Wooldridge	(University of Port Elizabeth)
Mr J A Zwamborn	(CSIR)

Messrs Arnold, Pegram, Rooseboom, Varkevisser, Wooldridge and Zwamborn wrote draft chapters for the document which were then distributed for comments by other committee members.

1.2 Purpose of guidelines

As in the case of the National Transport Commission (NTC) Road Drainage Manual* (Rooseboom *et al.* 1981) the primary aim of this document is to provide the designer with useful information, as well as guidelines on the application of this information. (Culvert design has been treated comprehensively in the NTC Road Drainage Manual and this information is not repeated here.)

A literature search, covering the topics which are addressed here, soon provides numerous publications which might be relevant. It cannot reasonably be expected of every practitioner in this field to keep abreast of all the literature and an efficient means of communicating this information must therefore be sought.

Guidelines, such as the NTC Road Drainage Manual, have proven that there is a large demand for the provision of design information in a pre-selected and user-friendly format. It is not the intention to restrict the competent designer in the use of his engineering judgement but to assist him in the decision-making process by providing useful information.

With so much and sometimes conflicting information available, the authors of guidelines face a daunting task. The simplest way out is to quote numerous formulae and other bits of information and to leave it to the reader to make his own choice. We have tried to select and recommend certain calculation procedures which we believe provide the most reliable answers under South African conditions. The competent designer may and should deviate from these in cases where he finds sufficient evidence to

* Renamed since second print of 1986 version as "South African Roads Board Road Drainage Manual (1993)".

justify a different answer, especially where the potential risk is low and the ideal solution prohibitively expensive relative to the safeguards being provided.

1.3 Economic and other considerations in bridge management

Various parameters need to be considered in both the strategic and tactical planning of bridge rehabilitation and maintenance. These include the direct costs incurred by the public using the bridge (or not being able to use it), the costs of remedial work, safety (which includes risk of collapse or damage during adverse conditions), and political or strategic considerations.

1.3.1 Ranking

Expressions have been developed for ranking bridges based on their current condition, and which can be used for deciding which bridges should receive immediate attention. Such ranking expressions should reflect the above parameters and should therefore be based upon the need of the society for an increased level of safety for river bridges. In principle, the same criteria should apply to all bridges. However, after the recent spate of bridge collapses, river bridges seem to be less safe than other bridges and there seems to be an increased public awareness about these bridges. A slightly modified perspective should therefore be used when focussing on river bridges. The parameters are briefly discussed here.

(i) User costs

It was pointed out above that the costs incurred by the user play a significant role, and forms a key element of the level of service provided by the bridge. These costs should be limited to acceptable levels. In the case of river bridges (which are otherwise in good condition), the costs incurred by the user will generally involve the cost of not being able to use the bridge during and after a flood (either due to collapse or due to damage to the bridge), and the possible delays caused during the flood when the public has to wait for the water to subside.

(ii) Safety

The safety of bridges plays a critical, but very intangible role in the level of service provided to the public. Several factors enter into creating an acceptable level of safety. These are briefly touched upon here, but are discussed in greater detail elsewhere.

It goes without saying that bridges should be kept in such a condition that floods do not cause damage to the bridges or their approaches. However, in extreme cases some damage or collapse could still be expected. A reasonable approach (well-known in engineering risk analysis) is to consider the risk-cost of each event, i.e. consider the consequence and its costs of failure, multiplied by the probability of occurrence of that event. That cost can then be compared with the cost needed to improve the bridge to prevent such a failure. The result is that the larger floods (with the larger potential resulting damage) have a very low probability of occurrence, and therefore also a low risk-cost. It may therefore not be worthwhile improving the bridge to prevent that mode of failure. Nevertheless, a further complicating factor needs to be taken into account: large catastrophies generate much more public awareness than a number of small events with the same total damage or number of fatalities. People are generally averse

to risk, and more emphasis may have to be placed on preventing the very severe collapses, even though the risk-cost may not justify it.

The public's perception of safety (or risk) is greatly influenced by the degree to which it can control their exposure to danger and to the degree of information provided to them regarding the perceived danger. Therefore, bridges which can fail suddenly without any prior warning, or which, after having failed hold a significant danger to the public, are bridges which need remedial work if flood damages could conceivably occur. Especially in the latter case it is necessary for the user to be able to see that a span has failed, to prevent him from using the bridge.

(iii) Political/strategic considerations

During and immediately after a flood, emergency services are usually operating at their peak. Bridges very often form the key link between different communities and it is of the utmost importance to ensure that these bridge are operational at such times. Alternative routes and the type of traffic expected at the bridge during and after a flood, will dictate to what extent that bridge is of strategic importance (as will the average daily traffic and the role of the bridge in the road network as a whole).

1.3.2 Degree of improvement

The degree to which a bridge should be repaired or maintained can conceivably differ from one situation to another. The one option is to do sufficient work on an existing bridge to ensure that it will perform satisfactorily for the next 25 years after which it will again need a considerable degree of improvement. Alternatively, it may be less expensive in the long run to repair a bridge every 10 years. Research to compare these options have been performed by Johnston (1987) and results indicate that the specific plan can differ from one case to the next. However, since river bridges may be subject to quite severe floods, the extent of deterioration may be difficult to predict. It may therefore be necessary to perform remedial work at shorter intervals to prevent catastrophic failures.

1.4 Organizational aspects

Inspection and maintenance of bridges call for a co-ordinated approach to the management of bridges. Clear goals need to be set and both activities need to be incorporated in the organization of the bridge authority. Currently, bridge management is handled in a number of ways: In some organizations the different activities related to bridges are all managed by a single person or office. In others, the aspects of design and construction are controlled by one office, while the maintenance and remedial work is managed by a completely different group (usually a department responsible for the maintenance of roads). In the latter case, only larger reconstructions and modifications to bridges are referred back to the central bridge design office.

Each of these approaches has its advantages and disadvantages. The centralized approach ensures that all planning can be done centrally, while in the other case the co-ordinated effort of working in a specific region could reduce costs. However, from an overall bridge management point of view, it is important to be able to plan, budget, and monitor on the total system level. For that reason it is advantageous to collect information on a system wide basis, and to rank, prioritize and budget on the full bridge system. Although it may, therefore, be advantageous to allocate work to a specific regional office, there is a strong

incentive to co-ordinate this from a central office where all data records can be kept. In setting up a bridge management function in an organization, it is important to ensure standardised procedures which will facilitate the exchange of data and enhance planning of all activities. The following aspects need to be addressed:

1.4.1 Inspection

Clear standards should be set for the inspection of bridges by using a standard bridge inspection manual to ensure a constant standard of inspection. Well documented record keeping of inspections is needed, as is the proper planning of regular inspections. Here it may be necessary to distinguish between normal inspections for all bridges, and the more frequent inspection of those bridges that may be prone to flood damage.

1.4.2 Record-keeping

A well planned record-keeping system is needed for recording all inspection records as well as the maintenance and repair activities on bridges. This data will in time prove invaluable in analysing trends and in predicting the rates of deterioration of bridges.

1.5 Bridge siting - Hydraulic aspects

From the hydraulics point of view a bridge should interfere as little as possible with the flow velocities within the stream which it crosses.

The magnitude of forces which flowing water exerts on bridge components as well as the scour which is caused by the bridge depend directly on the extent to which the bridge causes flow velocities to change in magnitude and direction. Ideally any bridge which is to interfere with the flow of a major river should be placed in position after the flow patterns have been studied, possibly in a model.

Moving a bridge a few hundred metres up or down a major river can make a significant difference to the risk of the bridge being damaged hydraulically. Similarly, shifting an abutment some meters along the road alignment can make a vast difference to the hydraulic attack on the abutment and its foundations.

River reaches where flows are confined to relatively narrow sections and where the flow directions do not vary significantly with varying discharge, generally provide the best crossing possibilities from the hydraulic point of view.

Flow around bends generate intensive scour along the outside of the bends as well as deposition of sediments along the inside. Crossings at bends in erodible material should therefore generally be avoided as special protective measures along the outside of the bend are required whilst bridge openings at the inside of the bend tend to be blocked by sediment deposits.

The most significant features in site selection are summarized below.

SITE SELECTION	
Positive features	Negative features
Solid beds and banks	Erodible material especially along outside of bend
Stable channels; constant position and direction, especially at higher discharges	Unstable channels; varying position and direction at higher discharges
Narrow main channels with small proportion of floodplain flow	Wide main channels with large scale floodplain flow

TABLE 1.1 : SIGNIFICANT FEATURES IN SITE SELECTION

2. HYDROLOGIC ESTIMATES

2.1 Introduction

"The Future is not what it used to be". - Quote

The purpose of this chapter is to inform the bridge designer of those methods of flood estimation currently available and appropriate in southern Africa and to assist with the choice of methods to solve the bridge designer's flood estimation problem.

The methods described are appropriate to catchments in predominantly rural areas. Where there is a high level of urbanization, the arrangement of contributing areas and drainage channels dominates the flood problem; the designer should then use one of the many computer packages designed for this purpose. These are not addressed in this chapter.

The methods described are all relatively easy to understand and quick to use, but they deal with an inherently complicated process, the conversion of rain-storms into floods. Therefore experience and engineering judgement play an important role in hydrologic estimates. Further, although guide-lines are given for the selection and interpretation of the methods, it must be emphasized that this chapter cannot be exhaustive (space does not permit a collection of the complete set of methods). The designer has the responsibility to become familiar with the methods as developed in the references. A good summary is given in the original NTC Road Drainage Manual (Rooseboom et al. 1981).

The chapter is divided into three sections:

- Summary of useful flood literature available locally,
- Collection and selection of the methods,
- An example of the recommended calculation procedures.

Because the methods are all relatively quick and easy to use, the designer may wish to ignore the problem of choosing between them and instead use as many as possible, and compare the results. Some methods are more appropriate than others in the various applications. Nevertheless, comparisons in flood calculations are **not** odious, indeed they are desirable. Designers are reminded that although each method can be used to calculate a flood to any desired numerical accuracy, this does not validate the estimate.

2.2 Some useful available local flood literature

"Of making many books there is no end; and much study is a weariness of the flesh". (Ecclesiastes 12:12)

It seems that only a limited amount of original work has been done to increase our understanding of flooding in the southern African region since the very able pioneering work done by the Hydrological Research Unit at Witwatersrand University in the early 1970s.

Some work has been done since then on small agricultural catchments by Schulze's research team of the Department of Agricultural Engineering at the University of Natal. Amongst other work, they applied the

US Soil Conservation Services (SCS) Method to local conditions by classifying South African soils in the various SCS categories.

An important document for the estimation of regional maximum floods according to locality and catchment area is the Department of Water Affairs Technical Report by Kovács (1988), based on the earlier 1980 publication. The method is based on empirical data collected by the Department of Water Affairs, and organized according to the Francou-Rodier system.

Mapping of rainfall with given duration and return periods has been accomplished in two independent studies by Schulze (1980) who gives maps, and Adamson (1981) who gives tables for more than 2 500 rain gauges in addition to maps for southern Africa. These form the basis of any rainfall-runoff calculation for modest return periods using unit hydrograph techniques or similar.

Recently, SANCOLD published Alexander's (1990) Design Flood Manual, which is a compendium of techniques and wisdom about floods acquired during a career in the Department of Water Affairs. This extensive volume serves as a useful summary of many of the methods available up to 1990.

Another guide/compendium which is not only exhaustive, but easy to follow, is the precursor to this volume, namely the NTC Road Drainage Manual first published in 1981 and subsequently translated into English (Rooseboom *et al.* 1981). There, mention is made of a method which has not received the amount of attention or recognition that it deserves, namely the runhydrograph method of Hiemstra and Francis (1979). The method is unique in that it ascribes probability not only to the peak of a flood hydrograph, but to the flood as a whole - peak and volume. Only with this method is it possible to estimate the flood hydrograph shapes directly from flow records where available, and to estimate families of flood hydrographs at partially gauged, and more importantly, ungauged sites.

The above reports and compendia are probably available in most consulting offices in the country. If not, they are relatively easy to acquire. Although they address the flooding problem in a classical way, they approach flood analysis in a piece-meal manner, not from a coherent fundamental philosophical viewpoint.

Such a philosophy undergirds the remarkable Flood Studies Report (NERC 1976) produced by the Natural Environment Research Council for flood evaluation in the United Kingdom. It is a large work and requires careful and patient reading. There is a Guide to the Flood Studies Report (FSR) which makes it more accessible (Sutcliffe 1978) and a report based on a conference aimed at airing the problems of implementing the FSR (Institute of Civil Engineers 1981). This comprehensive report (the FSR) has no parallel in southern Africa and, presumably because it appears to be so specific to Great Britain, has been under-quoted in this country. It contains a philosophy of procedure for employing well thought-out methodologies and is the fruit of the labour of a large team of engineers, hydrologists and statisticians at the Institute of Hydrology in Wallingford. Because of the important contribution that it makes to understanding the flooding problem, it will undoubtedly have a considerable impact on the future of flood research in this country.

In the sequel, frequent reference will be made to some key reports. They are listed in Table 2.1 for convenience; for the full reference consult the list at the end of this volume. Beside the column with name(s) of the authors, are four columns containing asterisks as flags. The first column is a (subjective) assessment of importance of the reference in terms of its indispensability, in the sense that information

is not readily available elsewhere. The remaining three columns indicate the methodological area with which the reference deals: deterministic (physical), empirical or statistical.

Reference	Importance	Det.	Emp.	Stat.
Adamson (1981)	***	*		*
Adamson (1989)	*			*
Alexander (1990)	***	*	*	*
Bauer and Midgley (1974)	**	*		
Hiemstra and Francis (1979)	***	*		*
Kovács (1980) and (1988)	***		*	
Midgley (HRU 1-72) (1972)	***			
Pitman, Middleton <i>et al.</i> (1981)	**			
Pitman and Midgley (1971)	**	*		
Pullen (1969)	*	*		
Rooseboom <i>et al.</i> (1981)	****	*	*	*
Schulze (1980)	*	*		
Zucchini and Adamson (1984)	*			*

TABLE 2.1 : REPORTS USEFUL FOR FLOOD ESTIMATION IN SOUTH AFRICA

2.3 Selection of calculation procedures

"For every complex problem, there is a solution that is simple, neat and wrong." (Mencken)

This section contains a list of the available methods, the problem types they address and the references where they may be found. Included is a flow chart to assist in the selection and systematization of the methods, and some comments on their usefulness and limitations.

2.3.1 Methods of flood estimation

As implied in Section 2.1, there are three broad divisions of flood estimation methods:

- Deterministic
- Empirical
- Statistical

(i) Deterministic methods

Deterministic methods are based upon our physical understanding of the rainfall-runoff process producing floods from storms.

These methods take into account (more or less) the influence of the following factors on the rainfall-runoff process:

- Catchment size
- Catchment slope
- Catchment shape
- Catchment soil type
- Catchment land use
- Climate as characterized by storm type, intensity duration, spatial distribution.

The Deterministic methods are:

(a) The Rational method

This method is useful especially for small, uncomplicated catchments and as a check method, but should not be used in isolation.

The peak flow is calculated using the following formula :

$$Q = 0,278 CIA$$

where

- Q = peak flow (m^3/s)
- I = average rainfall intensity over catchment (mm/hr)
- A = effective catchment area (km^2)
- C = run-off coefficient (ranging from 0 to 1)

The problem is to estimate the correct value for C. This is possible with experience, in which case it is a good formula for getting flood peaks, hence its wide use. The method reported here is used by the Department of Water Affairs and is well summarized in Section 2.4 of Rooseboom *et al.* (1981).

(b) The Unitgraph method

This method, also known as the unit hydrograph method, depends on the availability and applicability of a linear transfer function, also called the instantaneous unit hydrograph - (IUH). The IUH can be combined with a storm of a given depth and duration to give a unit hydrograph (based on a storm of a given duration and intensity), otherwise called a pulse response function.

This unit hydrograph (usually defined for a storm of 1 mm or 10 mm depth in one hour) can be combined with a design storm by numerical convolution to produce a flood hydrograph.

Certain assumptions are implicit in the method. The most important is that the catchment response is a linear process. This may be approximately true over a narrow range containing the floods used to calibrate the IUH. Extension to very intense storms is done, but is problematic. There are other difficulties associated with the separation of various phases of flow into surface and underground flow, and the definition of the design storm, but the method has been well developed and described in the literature.

The unitgraph is a popular and much used tool appropriate to large catchments. For example, the Australian Committee on Large Dams uses the method to compute the Probable Maximum Flood (PMF) from the Probable Maximum Precipitation (PMP) provided by their Meteorological Office, because they have not found a suitable alternative (Cantwell and Murley 1988).

In this country, the unitgraphs come from an old set of synthetic unitgraphs compiled by Pullen (1969) which were modelled using a linear storage model by Bauer and Midgley (1974). They were regionalized and are applicable (with caution) to ungauged locations.

(c) The SCS method

The third in the set of deterministic methods is the US Soil Conservation Service method developed for computing runoff from small rural (agricultural) catchments. It has been given a local flavour by Schmidt, Schulze and Dent (1987) and the method is summarized in Section 2.5 of Rooseboom *et al.* (1981).

The SCS method was based on a large number of actual storm events on a wide range of catchments, so has a strong experimental and experience basis. The advantage of the method is that it requires the designer to concentrate most of the estimation effort into a single parameter, the curve number (CN) which is, in effect, an infiltration parameter depending on soil type and land use - a very dependable method for small catchments.

The method is employed in a different guise by Hiemstra and Francis (1979) who use it to estimate families of hydrographs in partially gauged and ungauged catchments. In that application, CN is not estimated, but used as a calibrated parameter.

(ii) Empirical methods

The Francou-Rodier method, used, updated and extending by Kovács (1980, 1988) is an empirical method appropriate to southern Africa. The method is based on the plotting of envelope curves of observed flood peaks versus area for various regions.

(a) Regional maximum floods

Francou and Rodier examined some 1200 maximum recorded floods in the world, representative of most climatic regions. When envelope curves were plotted of log of flow versus log of catchment area, these curves tended to be straight lines which converged to a point representing the total mean annual runoff on all the land area of the earth.

The method is very easy to use and depends on only two pieces of information (locality and catchment area) to give an estimate of the regional maximum flood (RMF). Because these maxima are from observed sequences of limited length (40 - 100 years), they are expected to be adjusted upwards with time. This is a consequence of the following argument.

Let X be the maximum peak observed in a year. Then the probability that X is greater than a value y is $G(y)$. For example, if y is the 100-year flood, $G(y) = 0.01$. The probability that y will be exceeded in a sample of n years long is $1 - [1 - G(y)]^n$ (Benjamin and Cornell 1970, p.271), which increases, for fixed y , as

n increases. This means that the chance that a particular flood threshold y will be exceeded increases with n , the length of the observed sequences. This occurs whether the floods have an upper bound (PMF) or not. Note in addition and more specifically, that if the flood peaks follow a Gumbel distribution, the expected recurrence interval of the largest observation in a sequence of n years is approximately $1,6 n$ (Cunnane 1978).

The consequence is that the lines on the Francou-Rodier diagrams will inevitably shift upwards, if slowly, with time, although this is unlikely to materially affect design decisions. But note that these curves do not depend on catchment characteristics other than area. They are therefore useful for obtaining rough estimates of the magnitudes of "rare" floods that can be expected, but should not be used in isolation.

(iii) Statistical methods

Statistical methods are appropriate only where flood peak (and perhaps volume) data are available. The methods are therefore very site-specific and depend upon the existence of a long reliable record of flood flows, or alternatively, a sequence of spot estimates of flood flows measured after flood events.

The alternative for an ungauged site is to rely on regional studies, or other means of importing information to the problem site. The deterministic methods mentioned above purport to accomplish this, but the difficulty is that the recurrence interval (RI) of the flood does not match that of the storm in most circumstances, except, possibly, in rare floods.

Given that suitable data exist, the data must first be subjected to a screening and cleaning program as suggested by Adamson (1989). Only then can modelling be done with a chosen probability distribution.

The popular probability distributions are:

- Lognormal (2 or 3 parameter)
- Exponential
- Gamma
- Pearson Type III (or 3 Parameter Gamma)
- Log Pearson Type III
- Extreme Value (EV) distributions.

The last category has several members.

- | | |
|------------------------------------|------------------------------------|
| EV1 (Gumbel) | - constant skew, unbounded. |
| EV2 (Frechet) | - positive skew, lower bound. |
| EV3 (Weibull) | - negative skew, upper bound. |
| GEV (General extreme value) | - combination of EV1, 2 and 3. |
| TCEV (Two component extreme value) | - which is a mixture distribution. |

Other functions (such as the Wakeby) have been proposed - see Alexander (1990) for details.

Much has been written on distribution fitting; one of the more useful is the report by Zucchini and Adamson (1984), embodied in the book by Linhart and Zucchini (1986).

These methods address the problem of attempting to estimate the size of the flood peak which will be exceeded with given probability in any one year. The inverse of this probability is the recurrence interval (RI) in years, and typically the RI of interest to the designer is of the order of one or two hundred years, far exceeding the length of observed sequences commonly available.

Much discussion has raged over which is the "best" distribution. Zucchini and Adamson (1984) propose a method which fits the best model out of a repertoire of models which includes some of those listed above. The reason why the method is good is because it is adaptable and allows one to concentrate attention to fitting the upper tail which is where the designer's interest lies. Aware that the log normal performs as well as the others, that there are some theoretical misgivings about the log Pearson III and that the Gumbel has a fixed skew, the uninitiated would be wise to consult a statistician or an experienced stochastic hydrologist before taking decisions which could be expensive.

(iv) Statistical methods applied to peaks and volumes

To date, the determination of the joint probability of both peak and volume can be accomplished by only one method, the runhydrograph method described in Hiemstra and Francis (1979).

A comment on joint probabilities: Some people have difficulty understanding the concept of joint probabilities applied to flood peaks and volumes. Hopefully, the following explanation will help one to grasp the ideas.

Consider the following models : People in South Africa sized by height and mass; rainfall at a gauge - say Durban's Botanic Gardens - sized by intensity and duration.

Start with the people. For argument, assume that the median height of all people is 1,4 m. Of the people taller than the median, the median mass of those tall people is 70 kg, say. The proportion of the population both taller than 1,4 m and heavier than 70 kg is thus $0,5 \times 0,5 = 0,25$. Choose a different level of height as cutoff, say 1,6 m; for argument sake, it is found that the marginal probability of a person being taller than this is 0,417. We want to find the mass cutoff level so that these tall people have a probability equal to 0,6 of their mass exceeding this level. By searching, we find that this mass is 58 kg, say. The probability that any person is taller than 1,6 m and heavier than 58 kg is then $0,417 \times 0,6 = 0,25$, i.e. the same proportion as before.

The process of calculation can be repeated for many more pairs of points, but the marginal distribution of the height has a distinct value equal to the upper quartile. In other words, the height of the tallest quarter of the population, is 1,78 m, say.

In the second example, consider rainfall at Durban's Botanic Gardens as reported by Adamson (1981) as gauge 240891. Take out the 10 year recurrence interval rainfall totals for 1, 2, 3 and 7 days. They are 164, 226, 237 and 267 mm, with average intensities of 6,83, 4,71, 3,29 and 1,59 millimeters per hour; the longer the duration, the lower the average intensity for a given RI, which is intuitively obvious.

Adamson computed these statistics by examining the daily rain gauge record and extracting the largest 1, 2, 3 and 7 day rainfall totals in each of the 57 years of record. He then fitted a log normal distribution to them, calculating the level exceeded once every ten years on the average for each duration.

Clearly then, each of the 10-year storms (i.e. the 1, 2, 3 and 7 day storms and their associated total falls) will be observed to be exceeded each with the same probability (i.e. 0,1) in any one year. Here we are computing some of the family of 10-year storms.

In exactly the same way and with the same reasoning, Hiemstra & Francis (1979) propose the family of flood hydrographs of jointly distributed peaks and volumes of various probabilities of exceedance to describe the flood regime at a site.

(a) The runhydrograph method

The method was developed from records based on continuous measurements of streamflow, that is fully gauged catchments. The method was adapted to situations where there are only flood peaks available (partially gauged) and extended to ungauged catchment applications.

The disadvantage of the treatment of Hiemstra and Francis (1979) is that it was confined to data available prior to 1976, and therefore most currently active gauges are only in the "partially gauged" category. In common with all statistical methods, designers must satisfy themselves of the homogeneity and stationarity of data sequences before using them in the runhydrograph method.

2.3.2 Criteria for selection of methods

Most problem sites are completely ungauged, but if they are important enough, there is usually enough time (a year or two) to establish an economically justifiable gauging programme. This will enable some comparisons to be made between the flood/flow regime at the problem site and the regimes at gauges on the same or adjacent rivers so that information can be transferred. The programme can usually be justified by the added confidence and precision given to the flood estimates which in turn help prevent over- or under-design.

Part of the flow record of the Klip River at Standerton in the years 1910 - 1916 appears in Figure 2.1. Note that there is only one "typical" hydrograph; the others are thin, fat, multiple peaked or simply untidy. This is an example of a fully gauged site. A partially-gauged site is one where floods are measured after the event (not necessarily annually) by surveying the river and establishing the peak flow by the slope-area or similar method.

Which method or battery of methods to choose will depend on whether the site is gauged (fully or partially) or completely ungauged. The choice will also depend on whether only a flood peak is required (such as for a culvert). Alternatively, a complete hydrograph may be needed because storage is a factor, or because it is felt that more detailed information about time spent above various thresholds is desirable, especially in erodible river reaches. Again, a choice may be made between an estimated maximum flood or those with a given probability of being exceeded within a given time, usually a year or a design life.

Hall (1981) ably summarizes the FSR and, besides listing the variables mentioned above (availability of flow records, need for either peak or complete hydrograph and size of floods with given return periods), lists three more factors affecting the choice(s) of method(s) of flood estimation. They are:

- Climate, which affects the catchment response and the precipitation regime;

- The influence of human activity which causes land-use changes (and therefore response) and ultimately affects the homogeneity of records making them inappropriate for some methods;
- Size of catchment area, because an increase in size goes together with an increase in importance. Size also dictates the method used.

The following checklist summarizes the above discussion, after Hall (1981). The categories into which the problem site falls will guide the choice of methods to be used to assess the flood problem.

- CLIMATE CHOICE - HUMID / ARID, COASTAL / INTERIOR
SUMMER / WINTER RAINFALL.
- FLOW RECORDS - FULL / PARTIAL / NONE,
LONG / SHORT.
- HUMAN ACTIVITY - URBANIZATION / AFFORESTATION / FARMING
- SIZE OF CATCHMENT - SMALL / MEDIUM / LARGE
IMPORTANCE OF SITE

Before moving on to criteria for choice, the methods of flood estimation and their sources in locally published literature are given in summary form in Table 2.2.

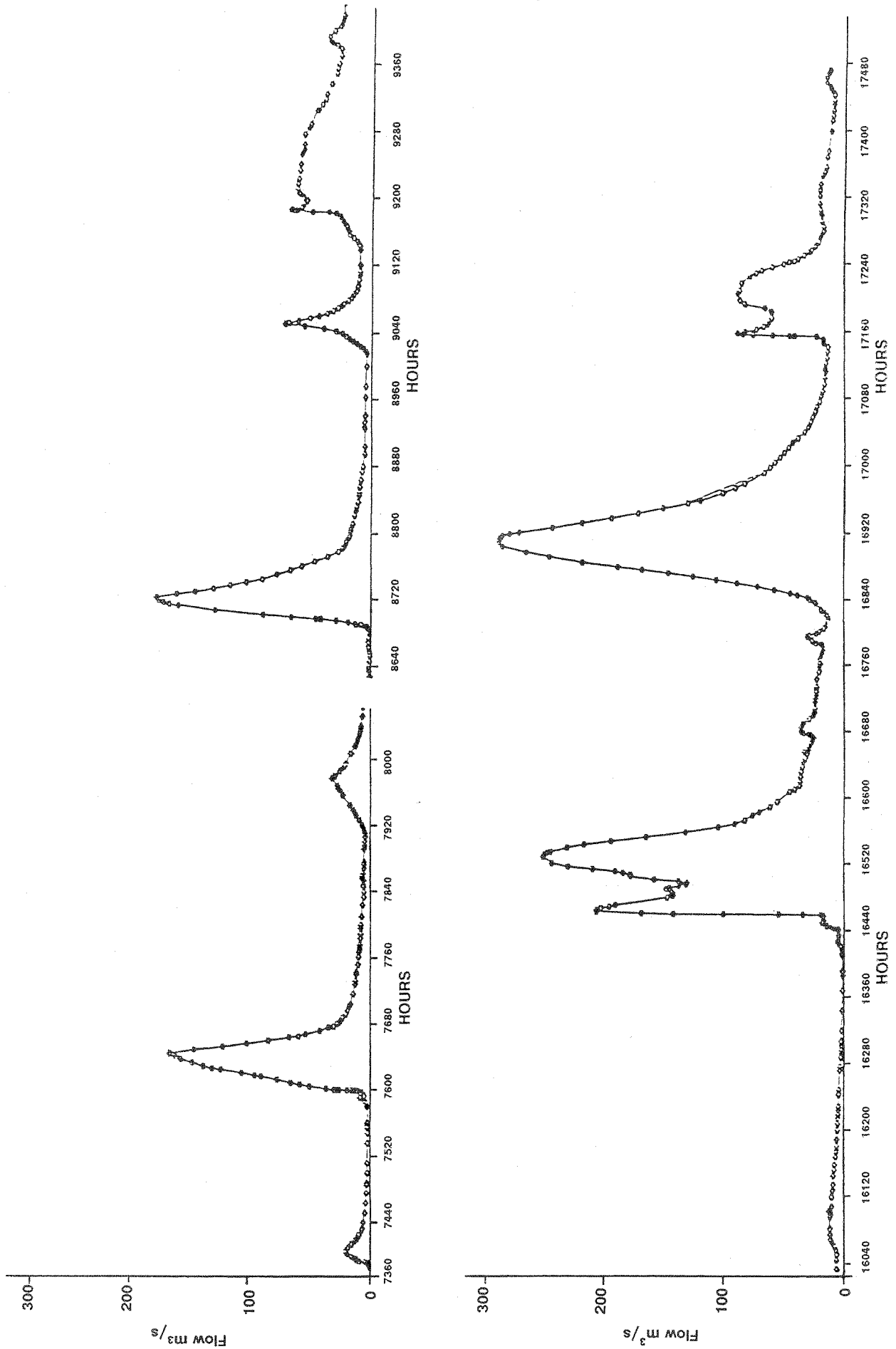


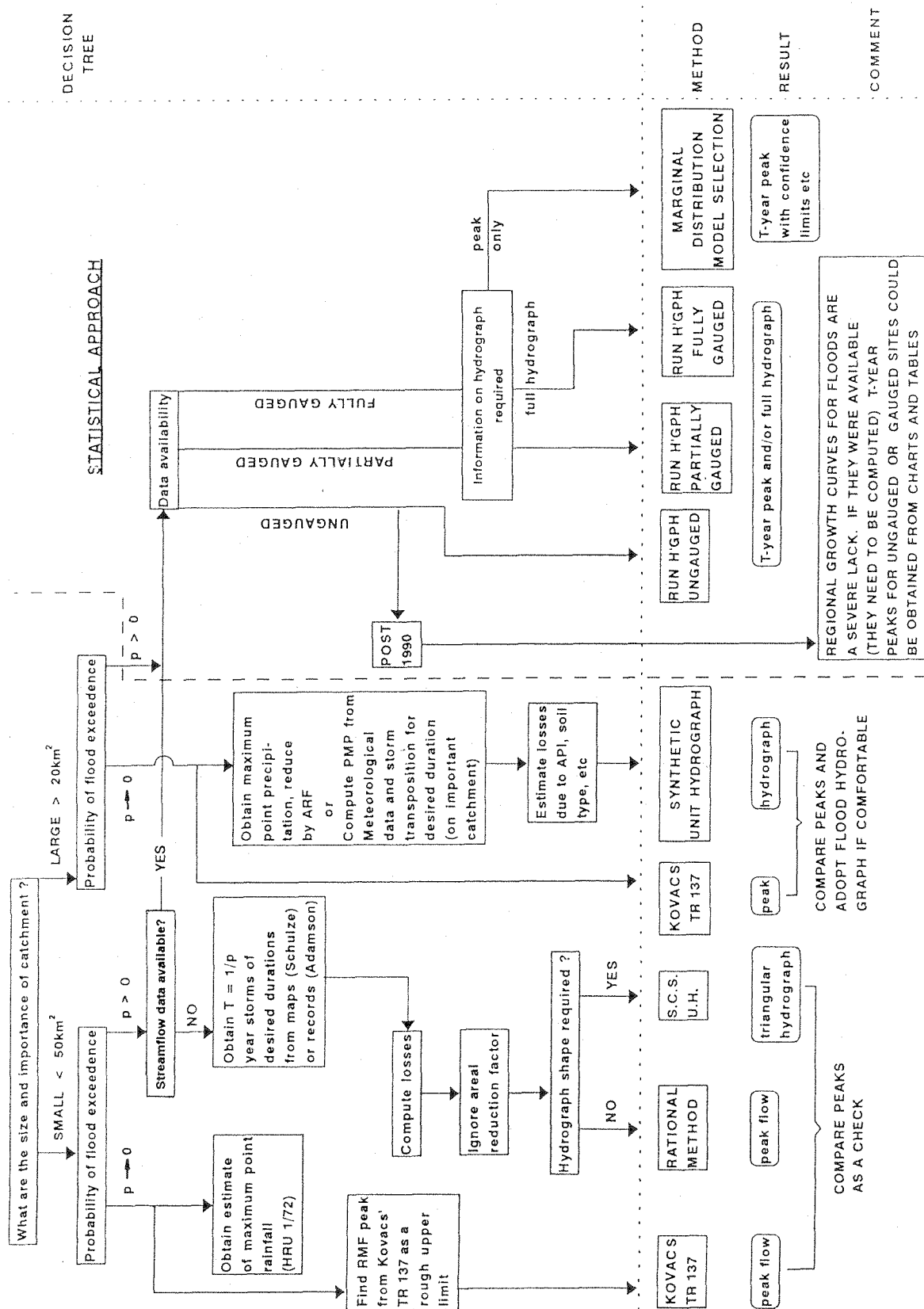
FIGURE 2.1 : SOME MEASURED HYDROGRAPHS ON THE KLIP RIVER AT THE LANGESDRIFT

Method	Catchment data required*	NTC (1981) Section No.	Prime Source (S. African)
A. Deterministic methods (Rainfall-runoff)			
1. Rational	Full	2.4	D.W.A.
2. SCS unit hydrograph	Full	2.5	Hiemstra & Francis '79, Schmidt et al. '87
3. Synthetic unitgraph	Area & location	2.6	Pullen '69, Bauer & Midgley '74
PMF from PMP based on (3).	Full	2.8	Pitman & Midgley '71
B. Empirical curve (Deterministic)			
4. Envelope curves for RMF/PMF	Area & location	2.8	Kovács '88
C. Statistical methods			
Data cleansing	None	-	Adamson '89
Runhydrograph		2.7	Hiemstra & Francis '79
5. Un-gauged	None		
6. Partial- "	to		
7. Fully- "	full		
8. Flood peak RI by fitting marginal probability distribution	None	2.3	Alexander '90
9. Ditto, but use model selection criteria	None	-	Zucchini and Adamson '84

* "Full" catchment data includes area, shape, slope, proportion of dolomite, proportion of urban / rural / lakes, soil types, vegetation cover.

TABLE 2.2 : METHODS OF FLOOD ANALYSIS - LOCAL SOURCES

To help with the choice of method(s) for a problem, the guide to the FSR (Sutcliffe 1978) incorporates a flow chart as A.1, which repeats Figure A 1 of the FSR. Using that as a basis, Table 2.3, has been drawn up. It lists the methods from Table 2.2, but indicates their strengths and weaknesses. Figure 2.2 gives a skeleton flow chart for the choice of method suggested as a solution to the flooding problem at a given problem site. The basic idea comes from Figure A 1 of the FSR, but that flow-chart cannot be used here because some methods developed in the FSR have no equivalent in southern Africa; a glaring example is the collection of so-called regional "growth curves" for defining the exceedence probability of rainfall.



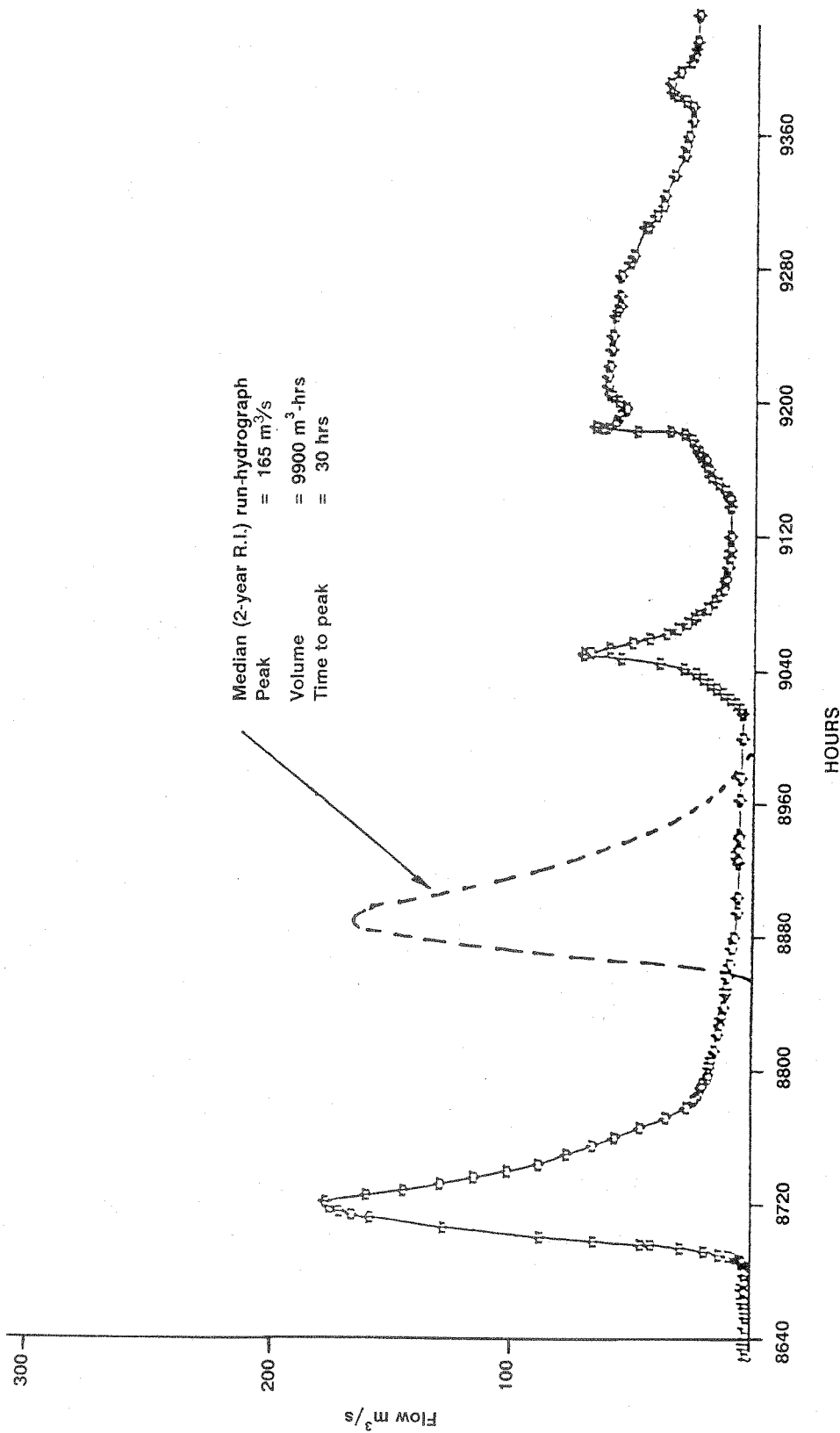


FIGURE 2.3 : HYDROGRAPHS ON KLIP RIVER AT DE LANGESDRIFT

CRITERIA	METHOD (SEE TABLE 2.2)								
	Deterministic			Emp'l	Statistical				
	1	2	3	4	5	6	7	8	9
Flow records									
None	■	■	■	■	■	★	★	★	★
Short	○	○	○	○	○	■	★	○	○
Long	○	○	○	○	○	■	■	■	■
Partial	○	○	○	○	○	■	★	■	■
Full	○	○	○	○	○	○	■	■	■
Catchment size									
Small	■	■	○	○	○	○	○	○	○
Large	★	★	■	■	■	■	■	■	■
Hydrograph									
Full	★	■	■	★	■	■	■	★	★
Peak	■	■	○	■	■	■	■	■	■
Flood probability									
EMF	■	■	■	■	■(1)	■	■	★	★
T-year	■(2)	■	■	★	■	■	■	■	■
Precision of method (depends on experience)*	D	B	B	B	C	B	A	B	A

Legend:

- method appropriate
- method does not require this condition, but can still be applied - there are probably better methods.
- ★ method is inappropriate with this condition
- A precise, reasonably independent of experience, but needs statistical facility
- B imprecise, needs some experience / elementary statistics
- C less precise, needs more skill, some local knowledge
- D imprecise, needs considerable experience, local knowledge

NOTES. (1) PMF flood - RI from 10^4 to 10^6
 (2) depends on the RI of the storm.

TABLE 2.3 : APPLICABILITY OF FLOOD ESTIMATION METHODS

2.3.3 Comments on choice of methods

Some comments on the choice of the methods suggested in Tables 2.2 and 2.3 and in Figure 2.2 are appropriate.

(i) Availability of records

If only peak flows are desired and peak flood data are available, then go directly to the statistical method and analyze the annual or partial duration (or what FSR call the peak-over-threshold or POT) data. This should be preferably done by employing the model selection techniques of Zucchini and Adamson (1984) which are outlined where appropriate in Section 2.3 following.

(ii) Hydrograph estimation

Three of the methods listed produce hydrographs as part of their output.

(a) The synthetic unitgraph

The synthetic unitgraph, based on observed or calculated hydrographs has a fixed shape. The synthetic unit hydrograph is a discrete analogue of a linear cascade formed from one storage element. Like all unit hydrographs, its use assumes that the output hydrograph's ordinates are increased in proportion to the increase of the rainfall for a given duration over the unit amount. There is no implicit change in the base length of the hydrograph. cursory examination of actual flow records confirms that this may well be a reasonable assumption over a narrow range.

(b) The runhydrograph

The second method to produce a hydrograph is the runhydrograph method. Shown in Figure 2.3 is a section of the flow record taken from Figure 2.1. Superimposed (dashed line) on Figure 2.3 is the median (2-year) runhydrograph for comparison with the measured flow. The recurrence interval of the measured hydrograph has not been determined so they cannot be compared in that sense, but the base-lengths are comparable, so presumably they were generated by storms of similar durations, i.e. about 2 days long. Other hydrographs exhibit the same behaviour even though we know that the rainfall-runoff process is non-linear in many of its sub-processes.

(c) The SCS hydrograph

The unit hydrograph is based on a linearity assumption. By contrast, in the SCS hydrograph, the runoff amount is non-linearly related to the rainfall amount for a given duration. However, the time base and time to peak are assumed to be functions only of catchment geometry, which supports the limited observations made above.

Expected Maximum Flood: There is some confusion about the various definitions of maximum flood that abound. Originally called the Maximum Probable Flood, the Probable Maximum Flood (PMF) has many detractors.

There seems to be scant evidence from our short historical records (mostly less than 60 years long) that there is an upper bound to the sampled flood distribution, even though probability distributions with upper bounds exist (Weibull, Johnson's Sb, etc.). On the other hand, a pragmatic appraisal of the meteorologically based flood-producing mechanisms suggests that an upper bound is not only plausible, but sensible. Given that this position is adopted, there are two approaches, which are not necessarily

complementary. The first is the RMF approach of Kovács (1988) who observes, "experience has shown that PMP values, while theoretically the upper limit, are still being revised and increased". Note also that Australian practice is to adopt PMP values computed by their meteorologists, not by engineers.

Clearly, an important site will merit some care and deliberation, perhaps some consultation with the Weather Bureau. A quick analysis, by contrast, can be achieved by working from HRU 4/69 or 1/71 (Pitman and Midgley, 1971).

Catchment data: In Table 2.2, an indication of the catchment data requirements of the various methods was given. There are loosely two categories of data requirements, minimal and full. The minimal requirement is by methods that employ regionalizing techniques to eliminate the need for detailed data gathering. However, they should be seen as broad-brush techniques, which ignore the remarkable variability that exists from catchment to catchment, even in the same "region".

Most catchment data relevant to the methods listed in Table 2.2 are available from Petras and du Plessis (1987) for 137 catchments. The information is obtained from gauges maintained by the Department of Water Affairs.

Of course, the most extensive collection of data on southern African catchments is the monumental "Surface Water Resources of South Africa" (Pitman *et al.* 1981). The data appropriate for use in flood studies include the land-use in all areas of the country in terms of irrigation, afforestation, reservoirs and their areas. In addition, the collection of maps of topography, geology, soil types and veld types prove particularly useful. This Surface Water Resources Series was compiled with yield in mind, not floods, but the information is usefully collected in a compact and accessible format. A sub-set of these maps appears in Alexander's (1990) compendium.

2.4 Determination of design flood values - An example

"An estimate is meaningless unless you know how good it is" (Jazwinski)

The designer may select the appropriate methods of flood estimation from the chart in Figure 2.2. They will consist of all or some of the following steps, remembering that comparisons are obligatory.

For a small rural catchment where only a flood peak is required of a given RI - say 50 years:

- Compute the RMF from TR 137
- Use a design storm and the rational formula
- Use a design storm and the SCS method.
- Compare the results. The RMF has an RI of the order of 200 to 2 000 years and should therefore lie above the other two estimates, unless the catchment is short, steep and bare of vegetation on a rocky surface.

- For a large catchment, repeat the above, but in addition compute the hydrograph using the unitgraph method for the RI of interest. Treat the result with caution.
- The hydrograph shape is of particular interest where storage is a problem (or an advantage) for instance where the opening in the embankment constricts and dams the flow. One would use the runhydrograph method here to get the hydrograph shape. It is the most reliable method to give a hydrograph with a given RI.
- For an important crossing, it would be wise to collect some flow data at the site for at least a year or two and compare that with gauged data on the same or a nearby river.
- In addition to the above analysis, one should then cautiously add a statistical analysis of the data. A complete analysis of the peak-volume data at a site is a specialist task and should be conducted by a team of engineers and statisticians. Nevertheless, this would give as full a picture as is possible for a given site. If this were not possible, reliance should be given to the runhydrograph analysis.
- In addition, a PMF calculation should be made.

In the remainder of this section, we use an example to illustrate some of the methods.

Assume that the flood regime is required at the C5M04 catchment near Bloemfontein, where there was a Water Affairs gauge. This site has been chosen because it is "fully gauged", and therefore can be used for comparison of methods. The catchment is fairly large and the site is likely to be an "important" crossing should a heavily trafficked road be envisaged.

2.4.1 Calculation of the estimated maximum floods

RMF: The first thing to do is to determine the size of the catchment: this turns out to be 4 939 km².

Immediately one obtains from TR 137 (Kovács 1988), Figure 7, that the appropriate curve for the region is 4,6. The RMF, read off from Figure 9c is 4 730 m³/s.

PMF: Using a quick approach, find the maximum point rainfall for a representative gauge in the catchment. (A more thorough approach would be to combine the records of several gauges in the catchment). The relevant quarter-degree squares by the Weather Bureau numbering are 231 & 2 and 261 & 2. According to TR 102 (Adamson 1981) the maximum observed rainfall in this area over three days is 283 mm in Bloemfontein. The recurrence interval of the rainfall is in excess of 200 years, but it is not high enough for a PMF.

Figure 5 in TR 137 (Kovács 1988), based on longer records than TR 102, records a maximum in the area of about 340 mm. Assuming, (after Adamson), that rainfall maxima follow a log normal distribution, a plot of data for Bloemfontein (Kings Park) on log probability paper yields the following 1 in 10 000 year point rainfall:

Duration (days)	Total (mm)
1	280
2	360
3	400

It seems that a rainfall of 340 mm in 3 days is exceptionally rare.

The 50 year recurrence interval rainfall figures are:

Duration (days)	Total (mm)
1	119
2	160
3	179

From Alexander (1990, Figure 13.8), the ARFs for 4 939 km² are:

Duration (days)	ARF
1	77 (85)
2	82 (87)
3	83 (88)

The values in parentheses are the ones recommended in FSR and can be found in Hlemstra and Francis (1979). From HRU 1/74 (Bauer 1974), the S-curve for C5M04 indicates that catchment equilibrium has been reached after about 50 hours. The duration of the storm producing the unitgraph is 10 hours, while the K value is 11,83 hours. We shall use information from HRU 4/69 (Pitman and Midgley 1971) and 1/74 to get the maximum peak flow from a unitgraph starting with a duration of 24 hours and working down.

A warning about the use of the synthetic unitgraphs from HRU 1/74. They are for total precipitation of 1 mm, not 10 mm which is sometimes used. Also, if the S-curves are to be used as checks on calculations, they are dimensioned and calculated incorrectly. The total flow out of a catchment due to 1 mm of rain falling on A km² should be equal to A/3,6 cumec-hours. The ordinates of the S-curve are given incorrectly in cumecs and the figures given for the total have been incorrectly divided by the duration of the storm. This does not invalidate the computation using the fitted or regionalized values of K. Therefore the examples in the text do not suffer from this problem.

Using the method suggested in HRU 1/74 for 280 mm, shared evenly over 24 hours (which is the standard way of applying the unitgraph) results in a peak flow of 14 600 cumecs at the 24th hour. Calculations show that this value is nearing a maximum. The maximum is confirmed as being 16 000 cumecs by using the rational formula for 11,67 mm/hour and a conservatively high coefficient C = 1,0.

For comparison, a calculation for the 12-hour storm yields the following. From TR 102 (Adamson 1981), the proportion of 24 hour rain that falls in 12 hours for the inland region is 0,92 that is 263 mm in 12 hours as a storm with a RI of 10 000 years; the resulting peak using HRU 1/74 with K = 11,83 hours is 19 000 cumecs. Reducing the duration further to 10, 8 then 6 hours, following the standard unitgraph approach, shows that the maximized peak is 19 750 from a storm of 8 hours duration. This is fortuitously getting close to the PMF of 20 000 cumecs for a catchment of 5 000 km², using a curve of exponent 6 (very

conservative) in TR 137. Although 20 000 is highly improbable, it is physically possible - the true definition of a PMF.

In summary then, the following expected maximum floods are appropriate to C5M04:

	EMF Peak (cumecs)	Source
RMF	4 730	TR 137
PMF	20 000	TR 102, HRU 1/74 & TR 137

The discrepancy between these two values should be cause for consideration. Recall, however, that the RMF is based on up to 60 year's experience and has an RI (recurrence interval) lying between 200 and 2 000 years, whereas the PMF has an average recurrence interval which goes back before Noah!

2.4.2 Floods of a given recurrence interval

Unitgraph: A technique not recommended for anything other than a cursory or approximate calculation is to use a rainstorm of given probability with a unitgraph (synthetic in the southern African region unless the designer computes one for the problem site). This technique is risky, because for moderate probabilities, the storm producing a n-year flood with a given peak can have a variety of shapes and recurrence intervals. Collective wisdom of ICOLD and the FSR is that when the floods are large, with vanishingly small probabilities, the storms causing them have similarly small probabilities: In these circumstances, the unitgraph may be used for PMF calculations. This correspondence does not happen for more frequent floods. Nevertheless (with caution!) we see that the 1:50 year, 1 day rainfall in Bloemfontein is about 120 mm. (We should feel confident about this value, because the records are of the same order of length as the desired RI.)

From Figure G2 in HRU 4/69 the mean infiltration and interception losses are of the order of 70 mm, so the residual storm runoff is about 50 mm.

The unitgraph is linear, therefore a direct scaling gives the 50 year flood peak as $19\,750 \times 50/280 = 3\,527$ cumecs.

Ungauged: Continuing to move from left to right across the flow chart of Figure 2.2, the first method suggested for ungauged locations where a probability is required, is the runhydrograph. As recommended by Hiemstra and Francis (H&F) (1979) in their Section 5.3, "select a nearly similar catchment with a known 2-year return period flood peak". C5M15 is downstream on the same river and is therefore a natural choice. (C5M15 has a non-concurrent shorter record, so that as chance would have it, a smaller mean exists for a larger area; but one would not know that if CM504 was ungauged.)

Listed here for convenience, are some relationships between parameters of the log normal distribution.

If a random variable X has a lognormal distribution with parameters m and s^2 , then $Y = \log_e X$ has a normal distribution with mean m and standard deviation s.

The median of X = exp (m)
 The mean of X = exp (m + s²/2)

If a particular value of X, X(a) has a probability equal to 1-a of being exceeded, then it is related to the corresponding value of the standard normal Z(a) by the following formula:

$$X(a) = \exp (m + s \cdot Z(a))$$

Thus if (1-a) = 0.02 (so that X(a) corresponds to the 50 year RI flood), Z(a) = 2,0537, which shall be used in the sequel.

Continuing with the example, from Table 2 in H&F it is found that m, the mean of the log of the peaks of C5M15 is 6,252. the median flood peak is thus exp(6.252) = 519 cumecs.

Catchment information for C5M15:

Area = 3 046 km²
 L = 159 km
 S = 0,00216

From TR 102,

P_{2,24} = 50 mm
 ARF = 0.85 (0.77 from Alexander).

Using the SCS method as H&F suggest,

T_c = 35 hours.

After "optimization" following the usual unitgraph approach as recommended in H&F Section 5.3, a choice of curve number equal to 83,6 yields a median flood peak of 515 cumecs, which is close enough to the median of 519 cumecs found above.

Applying this same CN = 83,6 to C5M04 whose catchment characteristics are

A = 4 939 km², L = 143 km, S = 0.00139

yields a median flood of 393 cumecs (from median storm rainfall). Where no other information is available, the variance of the log of the peaks is taken as 1,0 which follows from the fact that the mean of the coefficient of variation of the untransformed flood peaks is 1,3 - see their Figure 8. Because H&F found that the lognormal distribution was invariably applicable to flood peaks and volumes, the floods of various probabilities are simply a linear function of the median flood, i.e. q(p) = exp[Z(p)] . q where Z(p) is the standard normal variate corresponding to probability 1-p, and q is the median flood.

These values are as follows:

RI = 1/p	Factor
10	3,57
20	5,18
50	7,80
100	10,24
200	13,14
1 000	22,00
10 000	41,24

Note that under this assumption, the mean flood is $\exp(0,5) = 1,65$ time the median.

The 50-year flood is thus $7,8 \times 393 = 3\,065$ cumecs, while, as a check, the "PMF" is $41,24 \times 393 = 16\,200$ cumecs, i.e. of the same order of magnitude as the previous calculation.

Note that the SCS curve number CN is used here as an adjustment factor to match the "optimized" SCS peak to the median measured peak in the control catchment. Transferring this CN value to the ungauged catchment is a sensible way to calibrate imported information. It is therefore a definite improvement on the synthetic unitgraph approach for probability calculations in an ungauged catchment.

Should a family of hydrographs with a given recurrence interval be desired, then one needs to follow the runhydrograph route, and accept the median flood calculated using the SCS equations as suggested by H&F. The data that are missing are the standard deviation of the log of the flood volumes (whose median value from H&F turns out to be close to 1,0) and the cross-correlation between the log (peak) and log (volume) which turns out to have a median of about 0,85. One then follows their method to calculate the shape of the desired hydrographs. The computed family should be chosen from hydrograph peak/volume pairs within the range 3, 2, 1, 2', 3' as is suggested in H&F, because very few hydrographs have been observed outside these bounds (See their Figure 7).

Partially gauged: Peak flood data were gathered from the record of annual flow peaks kept by the Department of Water Affairs. H&F recorded the data extracted from the partial duration series in their Table 2. This can only be obtained from a complete record. By contrast, an annual series is usually available, extracted from the flow-records by the DWA Directorate of Hydrology. The statistics will be different, though related.

Assuming that only the annual flood data are available at C5M04, they comprise 28 years of data starting in 1919. This may seem like archaeology, and one must take care that current catchment conditions are the same as (or at least different in a known way from) the catchment conditions at the time for which records are available. Ignoring these problems for the purposes of this example, let us use the data to estimate the flood regime. Ranked, the 28 floods are:

84	84	91	98	142	173	207	231	231	243
270	306	313	357	439	534	539	454	816	816
1168	1180	1192	1580	1816	1849	2248	3200		

The median is 398 cumecs, while the standard deviation of the logs is 1,05, yielding a 50-year flood peak of 3 439 cumecs, calculated from $398 \cdot \exp(2,0537 \times 1,05)$, as suggested above.

Fortuitously (and quite surprisingly, because these calculations were done in sequence without backtracking to clean up, therefore there is no fudging of results), the median flood of 398 cumecs calculated from the list of 28 annual data comes close to the flood estimated from the SCS calculation above based on C5M15, namely 393 cumecs. The calculated standard deviation of the logs of the flows is 1,05, which is close to that assumed for the ungauged case, so there is no point in repeating the calculation here. Suffice it to say that if they were different enough, then the way to proceed to obtain a complete family of (say 50-year) runhydrographs would be to use the SCS method as adapted by H&F to compute the "optimizing" median storm and therefore flood volume. From there proceed as they suggest.

Fully gauged: The information now available is the complete set in Table 2 of H&F. Note, however, that these are data taken from a partial duration series, or what FSR calls a POT (peak over threshold) series.

The statistics for C5M04 from Table 2 in H&F are:

Mean rate (24 peaks in 28 years)	=	0,857
Truncation proportion	=	12,44%
Mean of log of peaks	=	6,863
Standard deviation of log of peaks	=	0,652
Mean of log of volumes	=	9,640
Standard deviation of log of volumes	=	0,838
Cross correlation, log (peak)/log(volume)	=	0,88 say 0,9

Thus the calculated modified rate of arrival = 0,979 -> 1,0.

Following their method, the peaks of the 50-year family have a median value of 956 cumecs and a maximum (marginal) value of

$$956 \cdot \exp[2,0537 \times 0,652] = 3\,647 \text{ cumecs}$$

This ties in fairly well with the estimates from the ungauged and the partially gauged treatments.

A comment on the size of the 50-year flood compared to the maximum observed in 28 years of data - namely 3 200 cumecs. The values of the 50-year flood calculated here are close to the observed maximum. Is this reasonable? It is important to realize that the recurrence interval of the biggest flood observed in an n-year sequence will be, on the average, 1,6 times as large as n (NERC 1976). In our case $1,6 \times 28 = 44,8$, so we can expect that the largest flood of 3 200 cumecs will occur roughly every 45 years on the average. In this example the two values, 3 200 and 3 600, are fairly close.

In other samples, the largest one or two values may easily be outliers which are huge compared to the rest of the observations, but these would have to be isolated in specific cases.

Marginal distribution model selection: This is the last of the methods applied to flood peak analysis, and as can be seen from Figure 2.2 and Table 2.3, is only applicable when annual or partial duration flood peak data are available.

When one has some data, like the small table of 28 annual flows of C5M04, it is tempting to "do statistics", quoting confidence intervals, distribution models and other arcane phrases. One should never lose sight of the fact that the sample is only a murky window into what happened over a relatively short space of time, when the conditions that existed were not what they are now, nor will be in our rapidly changing catchments. Given all of that, how do we use the information that we have?

In the analysis above, all the distributions assumed for both rainfall (TR 102) and streamflow, peaks and volumes (H&F) were lognormal. This follows the pragmatic statistical observation that most of these data yield a non-significant chi-square test when fitted by the lognormal.

The United States Geological Survey adopted the log Pearson III as the standard method in order to stop the bickering about which model to use. FSR recommends the generalized extreme value (GEV) distribution, and there are others. Alexander (1990) has collected these methods and programmed them for ease of use on a diskette available with the SANCOLD flood manual.

Given these models, we should not ignore two important issues associated with sampled data. The first is pre-screening and the second is model selection. A useful guide to pre-screening data, with programs available on diskette is available from the Department of Water Affairs (Adamson 1988) as TR 138.

The technique of model selection is based on the work of Linhart and Zucchini (1986) and is available in palatable form in Zucchini and Adamson (1984).

Suffice it to say that using these techniques on the C5M04 data, they seem to be clean (no outliers, trends, jumps, etc.) and a model selection routine based on Adamson's work employing the AIC and bootstrapping, yields the following. Under the assumption that the upper end of the sampled distribution is of interest in calculating rare floods, the criterion used for selection was:

$$\left| F(x_i) - G(x_i) \right|^{10}$$

where $F(x)$ is the model (sample) cumulative distribution function and $G(x)$ is the Cunnane probability $(i - 0,4) / (n + 0,2)$ where i is the rank of x_i in n data.

The models were ranked by this criterion in the following order:

Model	Rank	Criterion
Exponential	1	0.235
Weibull	2	0.237
Lognormal	3	0.240
Gamma	4	0.256
Gumbel (EV1)	5	0.388

The criteria for the exponential, Weibull and log normal are all reasonably close compared to the Gumbel, which is clearly not a competitor. Although the exponential distribution comes out best on these data, one should note that this is the exception rather than the rule and bear this in mind when examining the estimates of floods with various recurrence intervals (RI) given in the next table.

RI	Exponential	Lognormal
10	1 730 (360)*	1 690 (440)
50	2 880 (610)	3 740 (1230)
100	3 460 (730)	5 168 (1700)
1 000	5 200 (1100)	11 600 (4600)
10 000	7 100 (1400)	21 000 (9900)

* The standard deviations of the estimates, computed by the bootstrap algorithm, are given in parentheses.

A comment on the results is in order, without belabouring the statistical analysis too much. The 50-year flood estimated by the exponential model seems to be too low. It lies between the two largest observations of 2 248 cumecs and 3 200 cumecs. On this count, even though the model is "best" by a short head, one would treat the result with suspicion. On the other hand, the "PMF" of 7 100 cumecs is not much larger than the 4 730 cumecs estimated from TR 137.

The weight of evidence points to the lognormal distribution as being more suitable as a predictor. Note the large standard deviation associated with estimates of rare floods using the lognormal distribution. This wide spread is to be expected from such a short record rather than the modest (and seemingly over-precise) spread from the exponential distribution.

Summary : To complete the analysis, listed here are the various estimates of the flood peaks at the C5M04 site:

50 year flood (cumecs)	Source
3527	Unitgraph (approximately 45 year RI)
3065	Runhydrograph (ungauged)
3395	Runhydrograph (partially gauged)
3740 (1230)	Statistical analysis

For comparison, the EMFs are:

RI	Flood (cumecs)	Source
Max observed	3200	in 28 year record (approximately 45 year RI)
RMF	4730	TR 137
1:10000	21000 (9900)	lognormal distribution
PMF	20000	TR 137 + TR 102 + HRU 1/74

The numbers in parentheses are estimates of the standard errors of estimating the sizes of the floods.

Last Word: The long-duration, low-intensity storms in KwaZulu and Natal in September 1987 went on for days. In particular, in Pinetown, the maximum depth after 3 days was of the order of 600 mm. The maximum intensity measured was of the order of 30 mm per hour in this storm, which is not very remarkable in itself. However, the probability of this 3-day total being exceeded in any year is less than 1 in 200 and is nearer to 1 in 500 under the assumption that the log normal model is correct. It is not surprising that so much damage was done to foundations, houses, land, crops and infrastructure.

Nevertheless, because of the low intensity and the reasonably short rivers in most of the region (except the Tugela), the flood peaks did not exceed values with an estimated RI of between 1 in 50 and 1 in 100 years. The cause of the problem was the duration. It is rare for flood levels of this relatively moderate intensity to have been maintained for so long, and it was this continued attack which did the damage.

As Midgley says: "Beware the short, fat man as well as the tall, thin one!"

3. WATERWAY HYDRAULICS

3.1 Introduction

When it is necessary to cross a river by means of a road bridge it is often cheaper to reduce the length of the bridge by extending the approach road embankments as far as possible into the flood plain than to span the full flood plain with the bridge. (The ecological consequences need to be considered.) The constricted width, as well as the presence of bridge piers, result in increased flow velocities, and, unless the constriction is very slight or the flow in the unconstricted channel remains supercritical, cause a backing up of the water level immediately upstream of the bridge. The extent to which the bridge embankments can be permitted to constrict the flow is limited by the potential for scour due to the increase in flow velocities, and the effects that the backwater will have upstream.

The hydraulics of bridge waterways is therefore concerned primarily with the determination of these two factors. The analysis concentrates on calculating the backwater effect of the waterway and pier geometry, and the velocities are determined in the process.

The method most commonly used and generally followed here is that developed by the United States Bureau of Public Roads (1970). It is referred to here as the BPR method. The method is based on the results of hydraulic model studies as well as a number of field measurements taken during actual floods. The model studies were generally carried out for normal flow conditions prior to the introduction of the constriction, but abnormal flow stage conditions are also dealt with, although only in an approximate manner. The method concentrates mainly on the case where subcritical flow exists throughout the constriction, but a tentative method for the case where the flow becomes supercritical through the constriction is also given. Special cases such as dual bridges, bridges with scour and partially inundated bridges are also covered.

3.2 Basic equations

3.2.1 Flow types

Flow through a constriction in a channel can be divided into four types as shown in Figure 3.1.

(i) Type I flow

Here the normal flow depth, as well as the constricted flow depth, is above critical depth throughout, i.e. the flow is everywhere subcritical.

(ii) Type IIA flow

Normal flow depth is subcritical. The constricted flow passes through the critical depth of the constricted section but emerges from the constriction above the channel critical depth and is therefore supercritical through the constriction but subcritical downstream of the bridge.

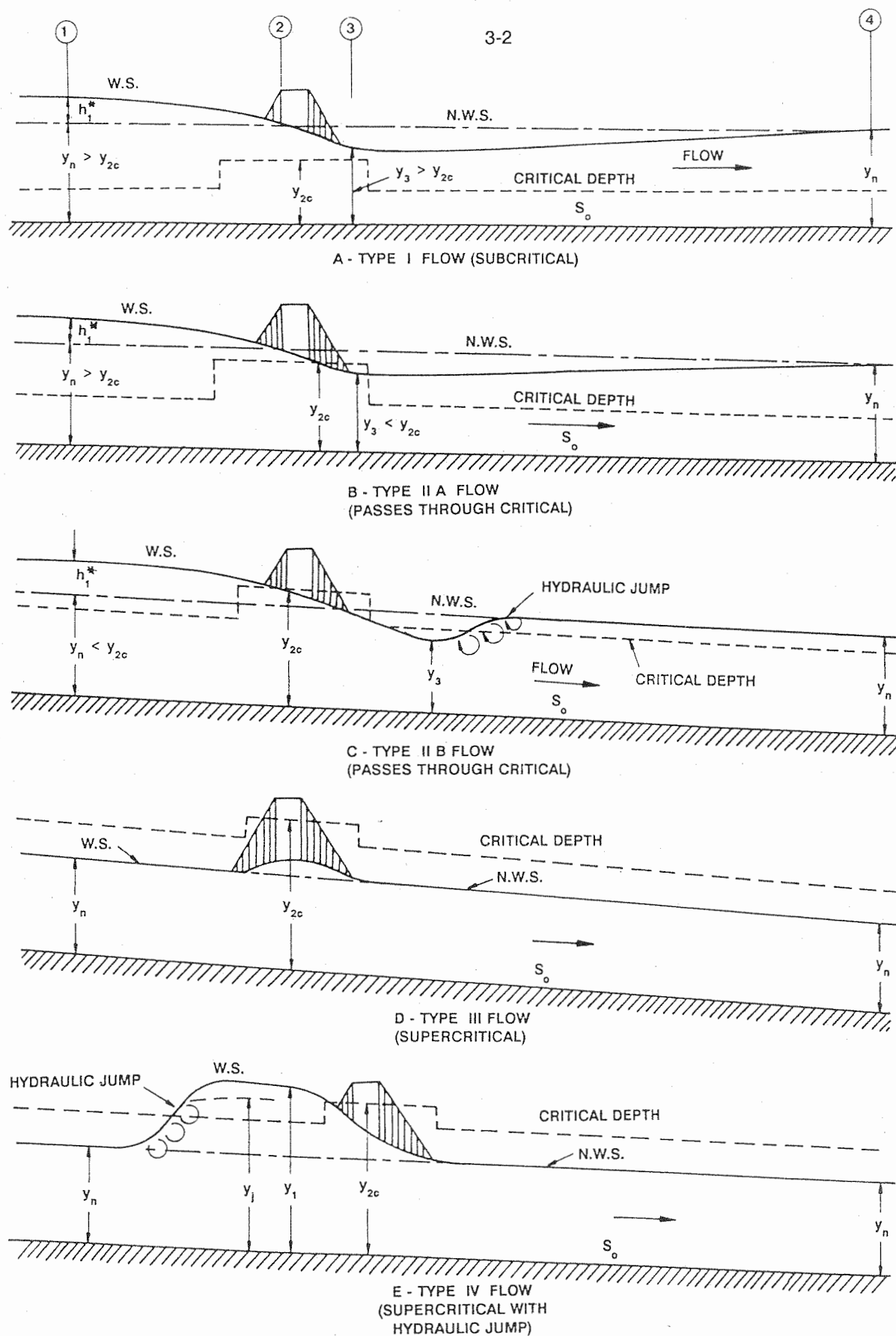


FIGURE 3.1 : FLOW TYPES THROUGH A CONSTRICTION

(iii) Type IIB flow

Similar to Type II except that the flow emerges from the constriction at a depth that is less than the channel critical depth, i.e. the flow immediately downstream of the bridge is supercritical. The flow returns to normal depth with an abrupt hydraulic jump.

(iv) Type III flow

Here the normal water surface and the surface through the constriction is below critical depth and the flow is therefore supercritical throughout.

(v) Type IV flow

The normal flow here is supercritical but the constriction is sufficient to force a hydraulic jump upstream of the bridge. The flow immediately upstream of the bridge is therefore subcritical becoming supercritical through the constriction and continuing supercritically downstream.

Flow Types I and II are the most common to be found in bridge waterway problems, whilst flow Types III and IV are encountered in streams having steep slopes such as to be found in mountainous regions.

3.2.2 Backwater equations

The BPR equations for backwater produced by a constriction are derived from energy principles (United States Bureau of Public Roads, 1970). In the derivation cross-sections through the channel are required both upstream and downstream of the constriction as shown in Figure 3.2, but for practical application of the method, a single representative cross-section is normally assumed throughout. With reference to Figure 3.2 the equations are:

(i) Type I flow

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} + \alpha_1 \left[\left(\frac{A_{n2}}{A_1} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \quad (3.1)$$

(ii) Type II flow

$$h_1^* = (C_b + 1) \frac{\alpha_2 V_{2c}^2}{2g} + y_{2c} - \bar{y} - \frac{\alpha_1 V_1^2}{2g} \quad (3.2)$$

where

- h_1^* = the backwater produced by the constriction (m)
- K^* = coefficient for total energy loss between sections 1 and 2 resulting from the constriction
- α_1, α_2 = velocity head coefficients at section 1 and 2
- V_{n2} = average velocity through the constriction based on the flow area below the unconstricted normal depth (m/s)

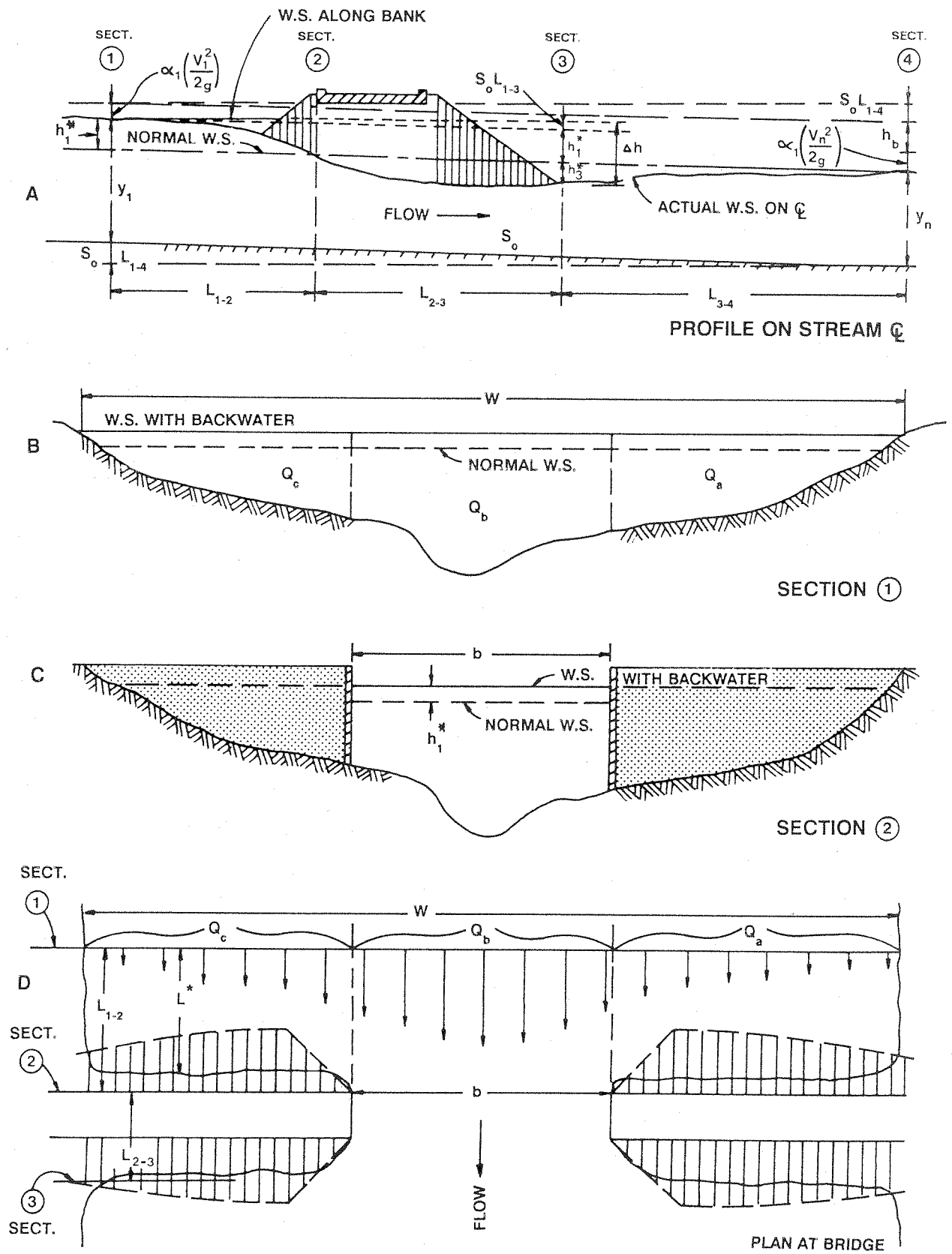


FIGURE 3.2 : NORMAL CROSSING

V_1	=	average velocity at section 1 ($= Q/A_1$) (m/s)
V_{2c}	=	average velocity through the constriction based on critical depth in the constriction (m/s)
A_{n2}	=	flow area in the constriction below the unconstricted normal depth (m ²)
A_1	=	flow area at section 1, including backwater (m ²)
A_4	=	flow area at section 4, usually assumed equal to A_n (m ²) where A_n is the flow area for the unconstricted case at section 1
C_b	=	backwater coefficient for Type II flow (constriction only)
y_{2c}	=	critical depth in constriction (m) [$= (Q^2/b^2g)^{1/3}$]
Q	=	flow (m ³ /s)
b	=	constriction width (m)
$\bar{y}$	=	mean flow depth below normal water level within the bridge constriction ($= A_{ns}/b$) (m)

In the BPR method the total backwater coefficient for Type I flow is considered to be made up as follows:

$$K^* = K_b + \Delta K_p + \Delta K_e + \Delta K_s \quad (3.3)$$

where

K_b	=	the base backwater coefficient for a normal crossing without piers or eccentricity
ΔK_p	=	the incremental backwater coefficient for piers
ΔK_e	=	the incremental backwater coefficient for eccentricity
ΔK_s	=	the incremental backwater coefficient for skew.

The above coefficients have been determined by means of hydraulic model studies. (Figure 3.3)

The backwater coefficient for Type II flow (C_b) is tentative and is derived from model tests. Piers, eccentricity and skew are not considered.

(iii) Type III flow

Theoretically no backwater is produced with Type II flow, but the bridge freeboard should be adequate to allow for the rise in water level through the constriction. It is not advisable to create a constriction in supercritical flow; however, should this be unavoidable, expert advice should be obtained. Type III flow is not dealt with further herein.

(iv) Type IV flow

Where a constriction is created across supercritical flow and is sufficiently severe, Type IV flow results.

The equation for backwater for Type IV flow is the same as for Type II with section 1 taken in the subcritical flow region downstream of the hydraulic jump. The depth of flow downstream of the hydraulic jump is given by the equation:

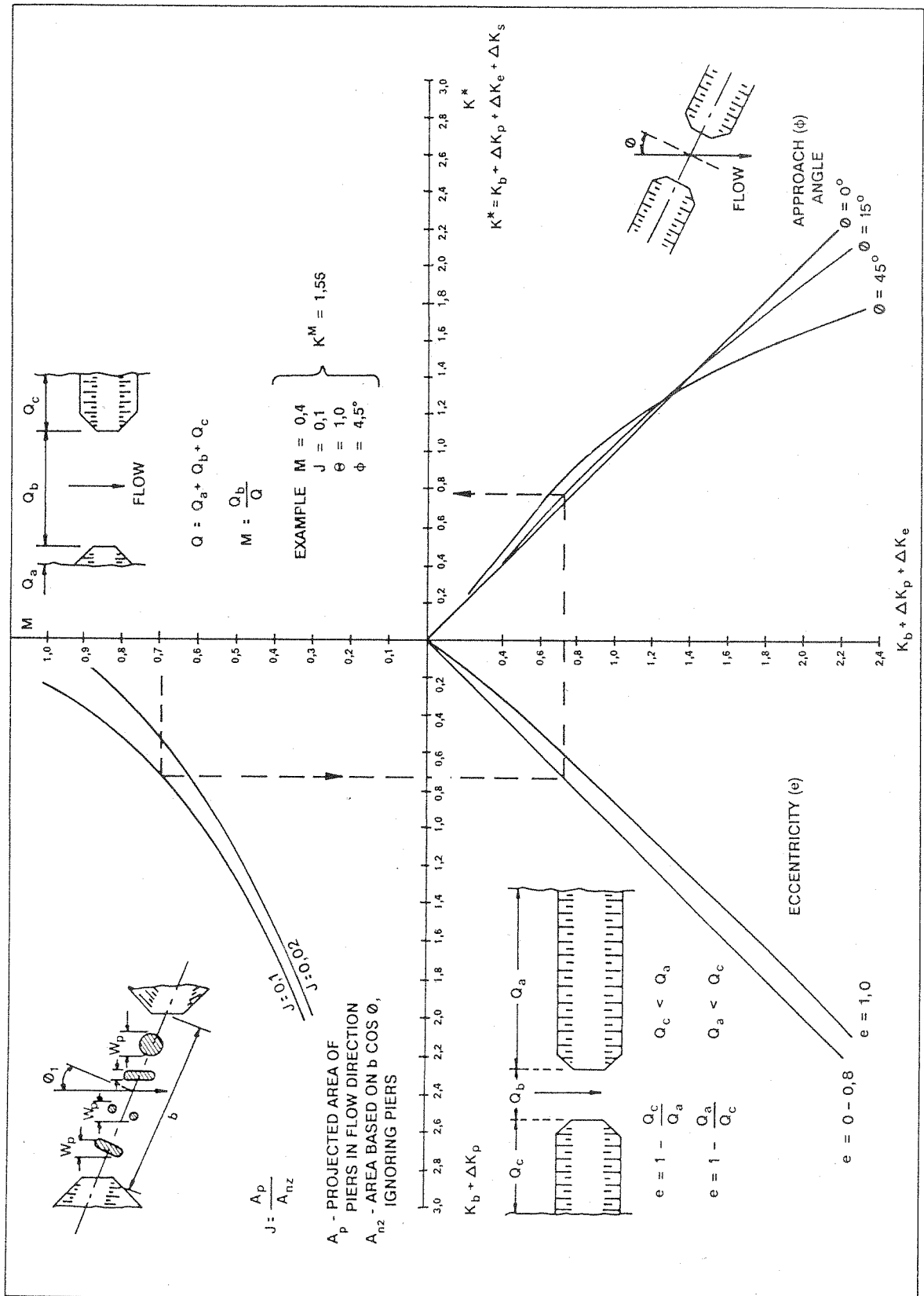


FIGURE 3.3 : CHART TO DETERMINE BACKWATER COEFFICIENT K^* (BASED ON BUREAU OF PUBLIC ROADS, 1970 AND PUBLISHED IN NTC MANUAL ROOSEBOOM ET AL. 1981)

$$\frac{y_J}{y_n} = \frac{1}{2}((1 + 8 Fr_n^2)^{1/2} - 1) \quad (3.4)$$

where

- y_J = flow depth downstream of the hydraulic jump (m)
 y_n = flow depth upstream of the hydraulic jump (m)
 Fr_n = Froude number of normal unconfined flow upstream of the hydraulic jump
 $= \left[\frac{Q^2 B}{g A_n^3} \right]^{1/2}$
 A_n = normal unconfined area of flow (m²)
 B = normal unconfined top width of flow (m)

3.3 Data requirements

For analysis of backwater and velocity by the above method, the following data are required:

3.3.1 Hydrological data

The estimation of the design flood is dealt with in Chapter 2 as well as the NTC Road Drainage Manual (Rooseboom *et al.* 1981).

3.3.2 River channel data

(i) Cross-section through the river channel

At least one cross-section is required at the bridge position but additional sections upstream and downstream should also be obtained for better definition of the channel. Sometimes this information is obtainable from contour plans, but for major structures a field survey is preferred.

(ii) River channel slope

This should be obtained with the cross-section data above.

(iii) Channel roughness data

Estimates of Manning's n or absolute roughness k for the channel and floodplain portions.

(iv) Downstream controls

The possibility of abnormal stage conditions existing at the site as a result of downstream influences such as weirs, constrictions or incoming tributaries should be investigated.

Additional field information required relating to foundations, scour potential, debris and previous flood magnitudes and levels, are dealt with elsewhere in this manual.

3.3.3 Bridge data

This data comprise :

- Horizontal alignment of the bridge including eccentricity and angle of skew.
- Vertical alignment of bridge.
- Type of abutments, i.e. wingwall or spillthrough.
- Number, size and shape of piers, and alignment to river flow.

The bridge data will generally be preliminary, as the object of the backwater and flow velocity calculation will be to determine the optimum waterway dimensions.

3.4 Calculation procedure

The procedure for the calculation of backwater and flow velocities is outlined below as it would be carried out by hand. However some or all of the steps are usually carried out using computer programmes.

It should be noted that the incremental coefficients were determined for normal flood stage conditions. However these can also be used for abnormal stage conditions as dealt with in Section 3.4.8.

The procedure for determination of the total backwater coefficient has been simplified in the co-axial diagram given in Figure 3.3 (NTC Road Drainage Manual, Rooseboom *et al.* 1981). This is adequate for most bridge problems encountered in practice but the designer may refer to the United States Bureau of Public Roads (1970) for more detailed charts.

3.4.1 Calculation of unconfined flood stage

- Divide the river cross-section into subsections according to geometry and Manning n values. It is convenient here to allow some subsection boundaries to correspond with the trial bridge widths to be used.
- Calculate the hydraulic properties of the section using the table below for the given discharge Q and a trial normal water surface elevation.

$$E_{sn} = y_n + \frac{V_n^2}{2g}$$

and the specific energy of the critical depth flow in the constriction.

$$E_{sc} = y_{2c} + \frac{V_{2c}^2}{2g}$$

where

y_n = the upstream normal depth (m)

$$V_n = \frac{Q}{A_n} \text{ (m/s)}$$

y_{2c} = the equivalent critical depth in the constriction (m)

$$= \left[\frac{Q^2}{gb^2} \right]^{1/3}$$

If $E_{sn} > E_{sc}$ then Type I flow will prevail, whilst if $E_{sn} < E_{sc}$, Type I or Type II may prevail. In the latter case both conditions should be checked and the most conservative used.

(ii) $Fr_n > 1$

To differentiate between flow Types III and IV, Figure 3.4 can be used, bearing in mind that the figure is for flow in rectangular channels.

The above methods of differentiating between the different flow types can only be considered as a rough guide when applied to real conditions, and it is preferable to look at both flow types where the cases are borderline.

3.4.3 Bridge opening ratio

$$M = \frac{Q_b}{Q}$$

where

Q_b = that portion of the normal flow within the width spanned by the bridge opening (obtained from the table in Section 3.4.1 above) (m^3/s).

3.4.4 Velocity head coefficients

$$\alpha_1 = \frac{\Sigma(qv^2)}{QV_n^2}$$

where $\Sigma(qv^2)$ is obtained from Section 3.4.1 above.

α_2 is determined from Figure 3.5.

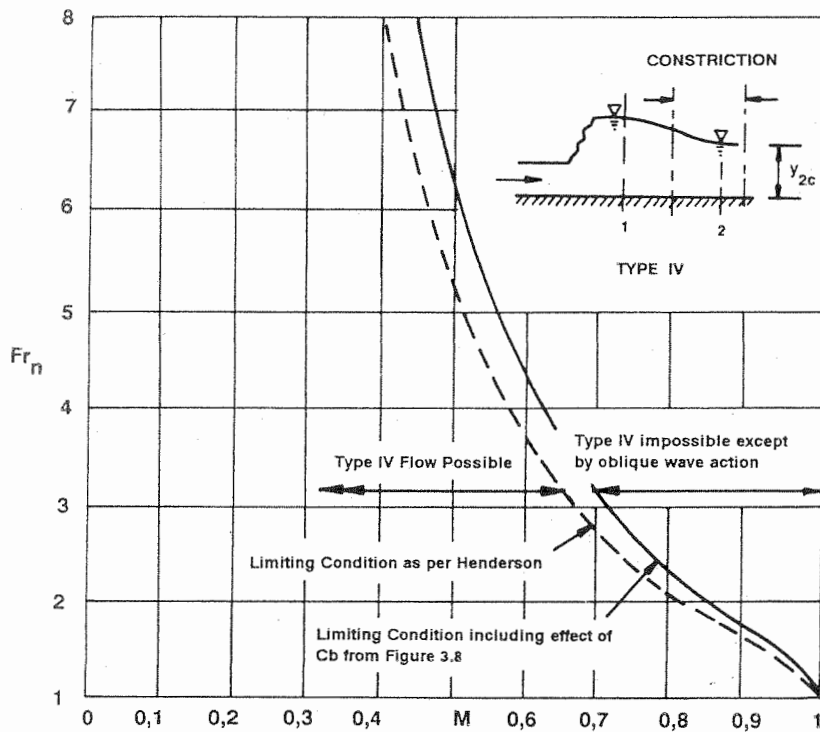


FIGURE 3.4 : CHART INDICATING THE LIKELIHOOD OF TYPE IV FLOW OCCURRING (BASED ON HENDERSON, 1966)

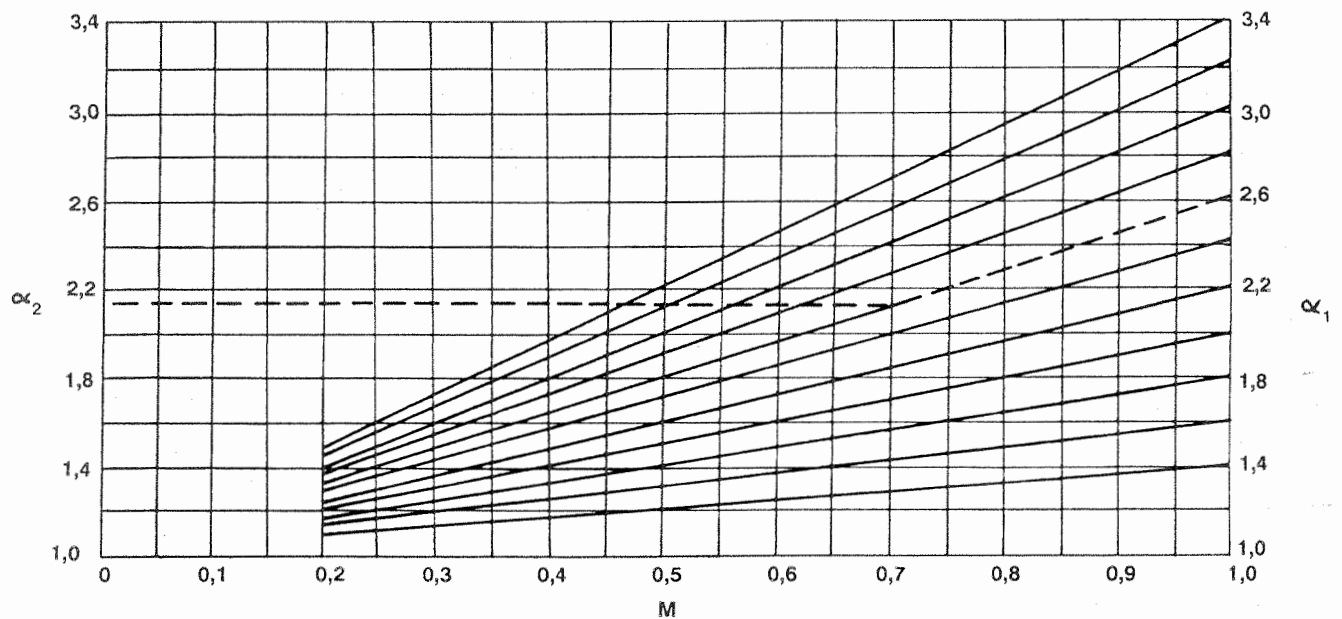


FIGURE 3.5 : VELOCITY HEAD COEFFICIENT IN CONSTRUCTION (BUREAU OF PUBLIC ROADS, 1970)

3.4.5 Type I flow

(i) Backwater determination

An approximate backwater is calculated from the first term in Equation 3.1 i.e.:

$$h_1^{*1} = K^* \alpha_2 \frac{V_{n2}^2}{2g}$$

where

K^* = total backwater coefficient obtained from Figure 3.3

$$V_{n2} = \frac{Q}{A_{n2}} \text{ (m/s)}$$

where

A_{n2} = area of flow below normal depth within the constriction boundaries and is obtained from the table in Section 3.4.1

The flow area of section 1 is then taken as

$$A_1 = A_n + h_1^{*1} B$$

and the total backwater is calculated from

$$h_1^* = h_1^{*1} + \alpha_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g}$$

where

A_4 = area of flow at section 4 assumed equal to A_n (m^2).

If h_1^* is substantially greater than h_1^{*1} , the last calculation can be repeated using a new

$$A_1 = A_n + h_1^* B$$

For skewed crossings, the constriction width, b , is replaced by $b_s \cos \phi$ where b_s is the skew width of the constriction and ϕ is the flow approach angle as shown in Figure 3.3.

(ii) Water surface level at the downstream embankment

This can be found using Figure 3.6. For normal crossings with or without piers, h_{1D}^* is the backwater from the constriction assuming no piers but including eccentricity and skew. However, with eccentric crossings the figure applies to the longer approach embankment only.

In all cases $h_3^* = h_{1D}^* \left(\frac{1}{D_b} - 1 \right)$

where

- h_3^* = change in water level at section 3 due to the constriction (m)
 D_b = the differential level ratio from Figure 3.6
 h_{1D}^* = backwater due to constriction excluding piers but including eccentricity and skew

The downstream water level is obtained by subtracting h_3^* from the normal water level at section 3.

The difference in water surface across the embankment is given by

$$h = h_3^* + h_1^* + S_o L_{1-3}$$

where

- L_{1-3} = distance from section 1 to section 3 (m)
 S_o = bedslope (m/m)

(iii) Position of maximum backwater

This is determined using Figure 3.7.

where

$$\bar{y} = \frac{A_{n2}}{b}$$

and L^* is measured from the position of the waterline on the upstream side of the embankment.

An iterative solution is necessary since y is dependent on the distance L_{1-3} as described above.

The maximum backwater level for normal crossings is assumed to extend over the areas A B C D and A E F G (Figure 3.7) with C D and G F being taken as about two bridge lengths.

(iv) Flow velocity

The highest flow velocity will occur at section 3 and can be estimated from:

$$V = \frac{Q}{(y - h_3^*)(b - \Sigma W_p)}$$

where

- b = width of constriction (m)
 ΣW_p = total width of piers normal to the flow (m)

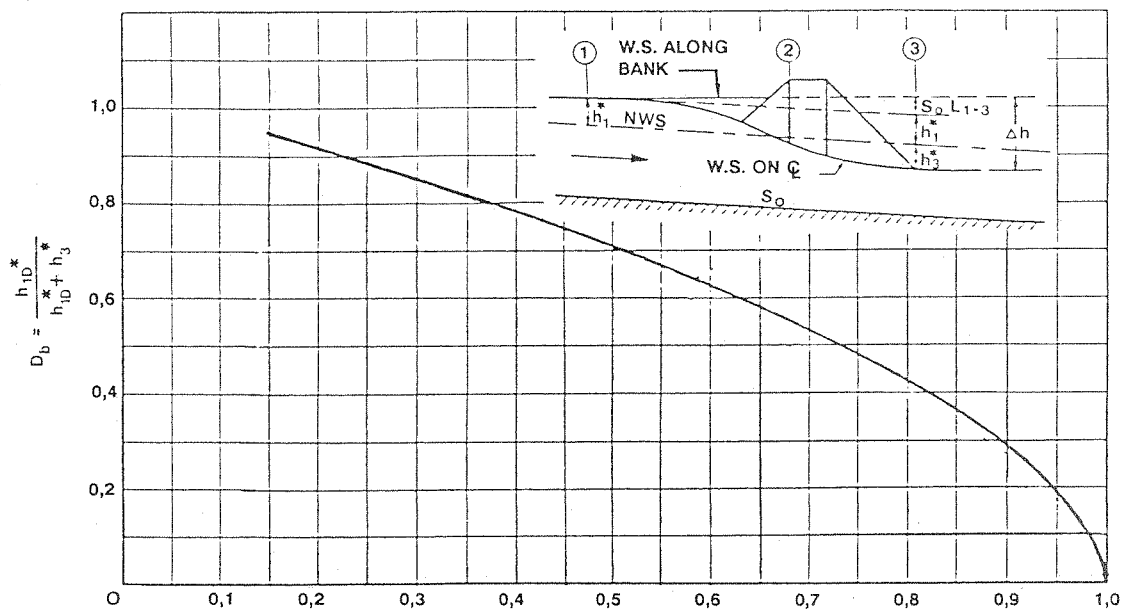


FIGURE 3.6 : DIFFERENCE IN WATER LEVEL ACROSS EMBANKMENT
(BUREAU OF PUBLIC WORKS, 1970)

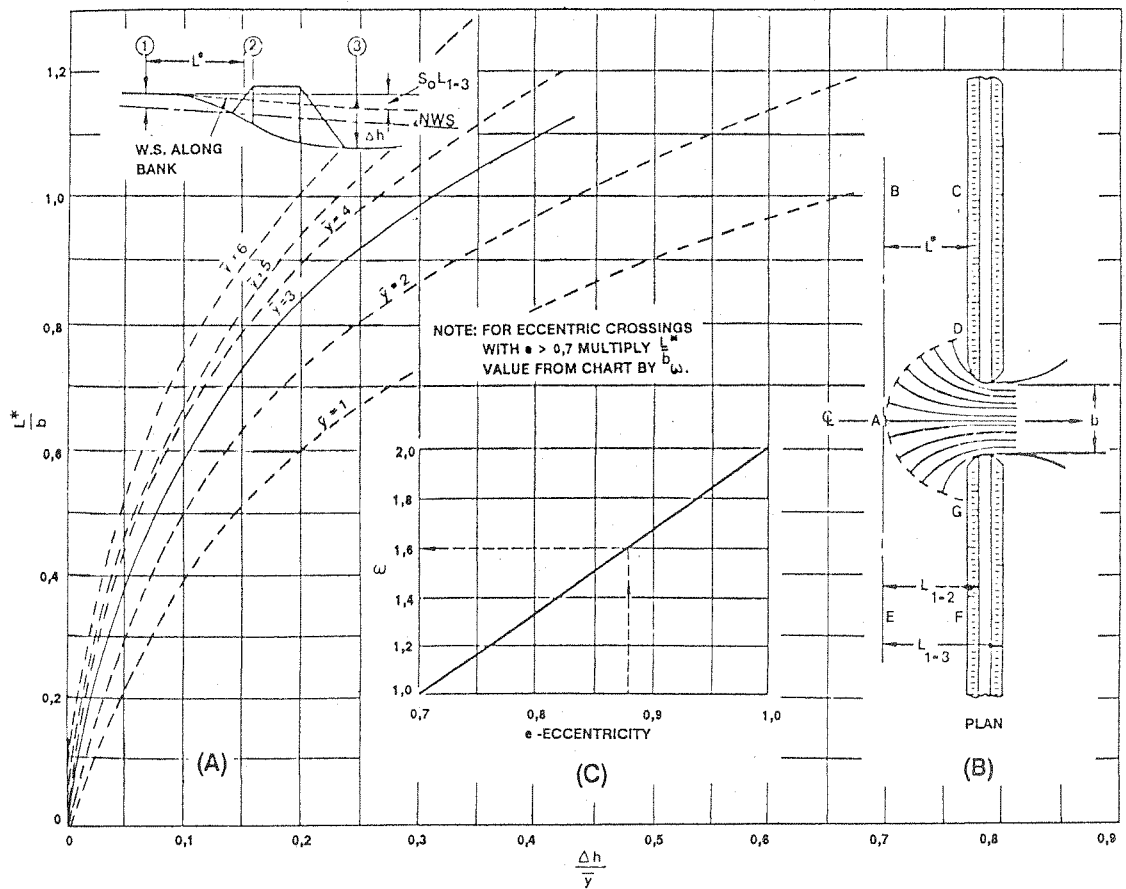


FIGURE 3.7 : DISTANCE TO MAXIMUM WATER LEVEL
(BUREAU OF PUBLIC ROADS, 1970)

3.4.6 Type II flow

(i) Determination of backwater

Because of the tentative nature of the Type II backwater coefficient, the effects of piers, eccentricity and skew are omitted.

The backwater is calculated from Equation 3.2

$$h_1^* = \alpha_2 \frac{V_{2c}^2}{2g} (C_b + 1) - \alpha_1 \frac{V_1^2}{2g} + y_{2c} - \bar{y}$$

where

$$\begin{aligned} V_{2c} &= \text{critical velocity through the constriction (m/s)} \\ &= (gy_{2c})^{0.5} \end{aligned}$$

$$\begin{aligned} C_b &= \text{Type II backwater coefficient from Figure 3.8} \\ y_{2c} &= \left(\frac{Q^2}{gb_N^3} \right)^{1/3} \end{aligned}$$

$$\bar{y} = \frac{A_{n2}}{b}$$

$$b_N = \text{net constriction width i.e. b-total width of piers}$$

$$\begin{aligned} V_1 &= \text{velocity at section 1 (m/s)} \\ &= \frac{Q}{A_1} \end{aligned}$$

Initially A_n is used for A_1 in computing V_1 . Thereafter the calculation is repeated using:

$$A_1 = A_n + h_1^* B$$

where

$$B = \text{upstream top width at normal depth (m)}$$

The water level at section 1 is given by $(y + h_1^*)$.

Note that an equivalent trapezoidal section of depth y below the normal water surface in the constriction is assumed.

The incremental backwater coefficients for piers, eccentricity and skew are not applicable for Type II flow.

(ii) Water surface level at downstream embankment

The drop in water level below normal depth at the downstream embankment is estimated as follows using Figure 3.6:

$$h_3^* = h_1^* \left(\frac{1}{D_b} - 1 \right)$$

This figure is subtracted from the normal water level at section 3 to get the water level at the downstream embankment.

(iii) Velocity of flow

The highest flow velocity is at section 3 and can be estimated from

$$V = \frac{Q}{(y-h_3^*)(b-\Sigma W_p)}$$

where

b = width of constriction (m)

ΣW_p = total width of piers normal to the flow (m)

3.4.7 Dual bridges

Figure 3.9 can be used for determining the backwater for dual bridges once the backwater for a single bridge has been calculated.

The water level at the downstream embankment of the downstream bridge can be determined from Figure 3.9(B).

The water level between the two bridges cannot be determined but is usually above the normal depth.

3.4.8 Abnormal stage discharge condition

The procedure for abnormal stage conditions is applicable when flow in the unconstricted channel is influenced by downstream conditions. The unconstricted flood water level at the bridge must then be determined by one of the methods for computing flow profiles, such as the standard step method (Henderson 1966, Rooseboom *et al.* 1981). This level is used in place of the normal flood water level, and the backwater base coefficient and incremental coefficients determined as for Type I flow. The method is only approximate and the second term in Equation 3.1 is therefore ignored. The backwater is calculated from

$$h_{1A}^* = K^* \alpha_2 \frac{V_{2A}^2}{2g}$$

where the subscript A refers to abnormal stage conditions.

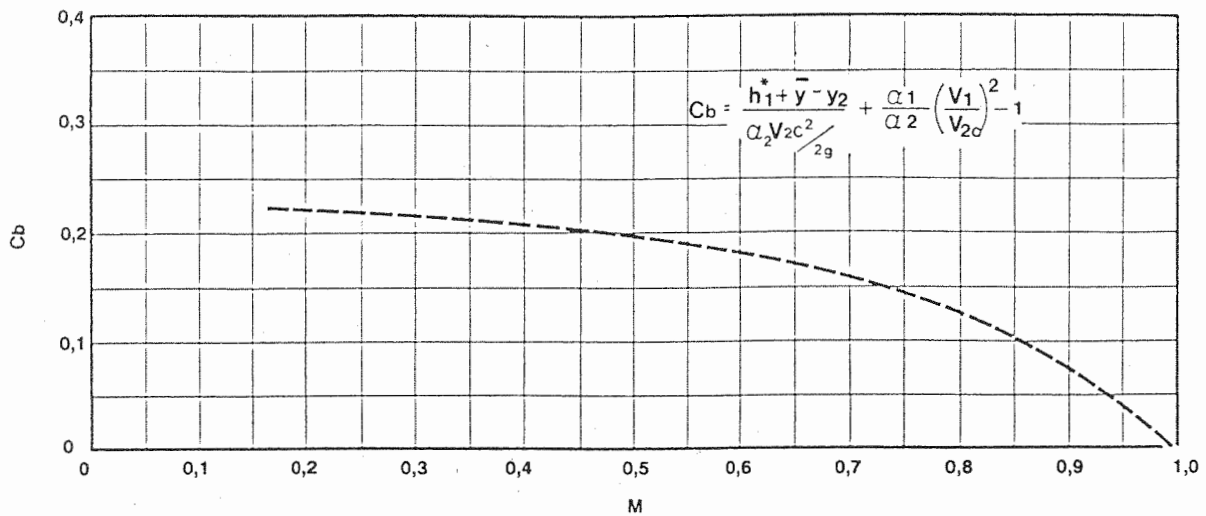


FIGURE 3.8 : DISCHARGE COEFFICIENT FOR TYPE II FLOW
(BUREAU OF PUBLIC ROADS, 1970)

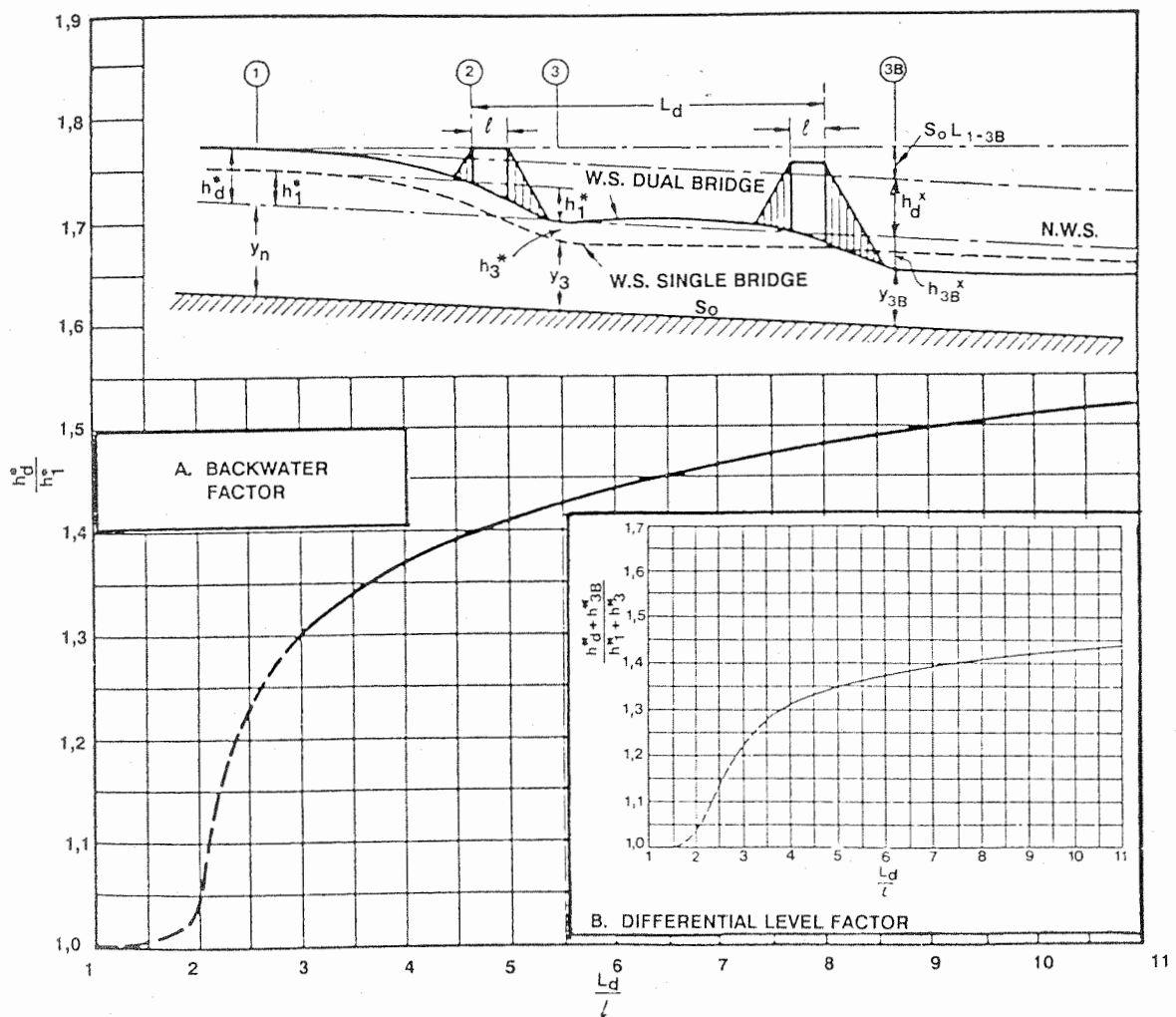


FIGURE 3.9 : FACTORS FOR DUAL BRIDGES
(BUREAU OF PUBLIC ROADS, 1970)

3.4.9 Effect of Scour on Backwater

The subject of scour is dealt with in more detail in Chapter 4. The BPR method introduces a correction factor to allow for scour:

$$C = \frac{h_{1s}^*}{h_1^*} = \frac{h_{3s}^*}{h_3^*}$$

where the subscript s refers to the condition where the bed has been enlarged through scour, and the other values are for the unscoured condition. C is obtained from Figure 3.10 where A_s is the additional flow area created by scour.

The designer should be cautious about relying on scour to enlarge the waterway (Section 5.5).

3.4.10 Superstructure partially inundated

The case where the superstructure of the bridge becomes partially inundated is normally only considered when considering the stability of the structure under more extreme floods than the waterway design flood. From a hydraulic point of view the flow under the bridge may be considered as either a free orifice (sluice gate) or a submerged orifice, depending on the tailwater level.

These conditions are illustrated in Figure 3.11 for the following discharge equations:

(i) Upstream girder in contact with flow : Case I

Where the flow is in contact with the upstream girder of a structure and subject to having free discharge downstream, the total discharge is given by the equation :

$$Q = C_d b_N Z \left[2g \left(y_u - \frac{Z}{2} + \frac{\alpha_1 V_1^2}{2g} \right) \right]^{0.5}$$

where

Q	=	total discharge (m ³ /s)
C_d	=	coefficient of discharge (from Figure 3.11)
b_N	=	net width of waterway excluding piers (m)
Z	=	vertical distance from underside of bridge deck to mean river bed (m)
y_u	=	vertical distance from upstream water surface to mean river bed at bridge (m)

(ii) All girders in contact with flow : Case II

Where the entire area under the structure is occupied by the flow and subject to having submerged discharge downstream, the total discharge is given by the equation :

$$Q = C_d b_N Z (2g h)^{0.5}$$

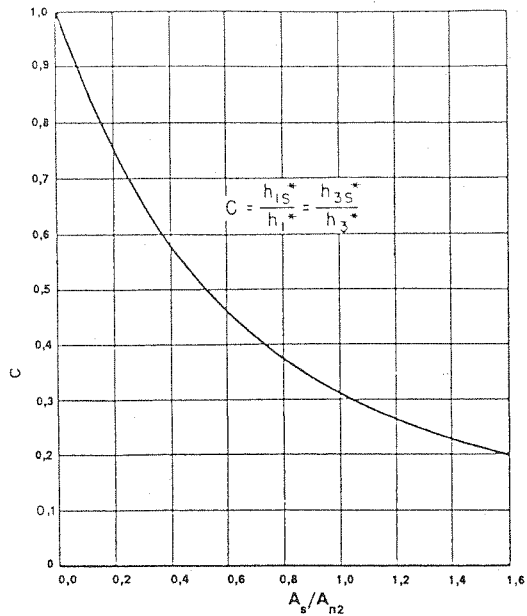


FIGURE 3.10 : SCOURING CORRECTION FACTOR
(BUREAU OF PUBLIC ROADS, 1970)

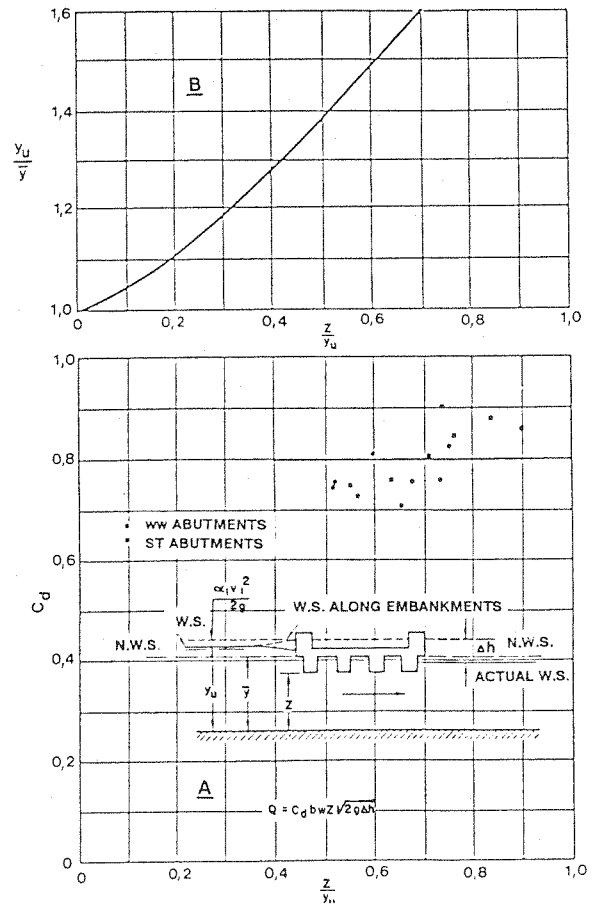
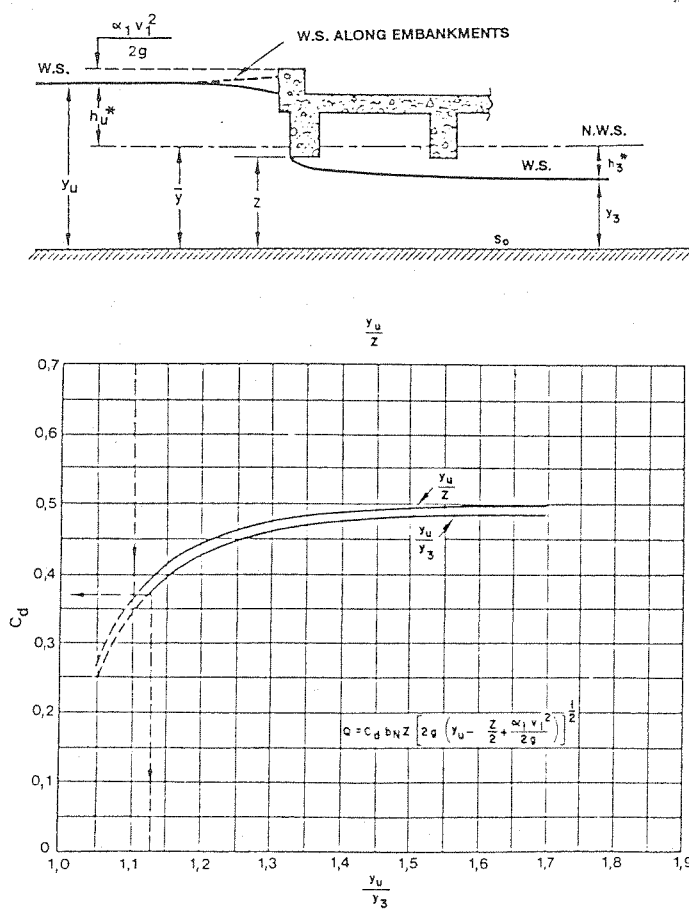


FIGURE 3.11 : DISCHARGE COEFFICIENTS FOR PARTIALLY SUBMERGED FLOW
(BUREAU OF PUBLIC ROADS, 1970)

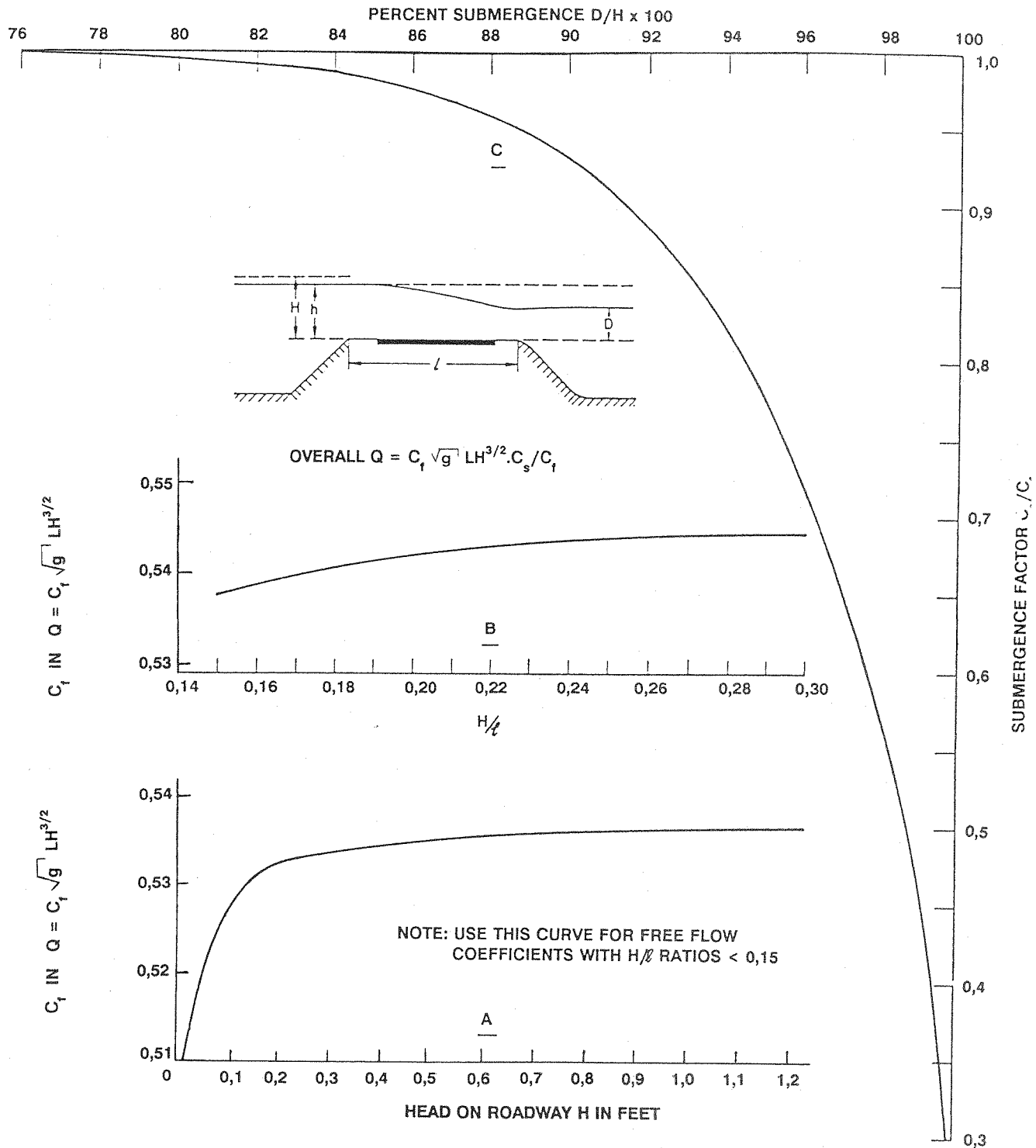


FIGURE 3.12 : FLOW OVER EMBANKMENTS
(BUREAU OF PUBLIC ROADS, 1970)

where

h = difference in water level at upstream and downstream embankments (m)

For Case II, Figure 3.11 (B) allows the determination of y_u from h and the normal water depth, y .

(iii) Flow over approach embankments

With inundation of the bridge comes the potential for flow over the bridge approach embankments. This can be estimated from the equation for a broad crested weir, with a correction for downstream submergence if necessary.

$$Q = C_f g^{0.5} L H^{1.5} C_s / C_f$$

where

C_f = discharge coefficient from Figure 3.12

L = length of the overflow portion of embankment (m)

H = upstream total head above embankment crest (m)

$$= h + \frac{V_1^2}{2g}$$

$\frac{C_s}{C_f}$ = submergence factor Figure 3.12

The combined discharge through the bridge opening and over the embankment requires an iterative solution for the upstream water level which is greatly facilitated by the use of a computer program or spreadsheet.

3.4.11 Type IV flow

Type IV flow is analysed for backwater in the same manner as Type II flow but first requires the determination of the subcritical flow depth upstream of the bridge. This is obtained from Equation 3.4.

$$\frac{y_J}{y_n} = 1/2 ((1 + 8 Fr_n^2)^{1/2} - 1)$$

where

y_n = normal flow depth (supercritical) obtained for the unconstricted channel in a similar manner as described for subcritical flow in Section 3.4.1

y_J = water depth after the jump

The upstream Froude number is calculated from:

$$Fr_n = \left(\frac{Q^2 B}{g A_n^3} \right)^{1/2}$$

where

A_n = upstream flow area (m^2)

B = upstream top width of flow (m)

y_u is then used to calculate A_n in the procedure for Type II flow.

The total backwater caused by the constriction is then: $y_1^* = y_1 - y_n$

3.5 Assumptions and limitations

The methodology based on the above equations and coefficients from Bureau of Reclamation (1970) is subject to the following assumptions and limitations:

- Normal flow is assumed in the channel prior to constriction. In other words the effects of a water surface prior to the constriction which is not parallel to the river bed are not accounted for in the coefficients for Type I flow.
- The method is intended for relatively straight reaches of streams having subcritical flow and approximately uniform shape, although for Type II flow variation in cross-section will not cause serious errors. Streams with extremely sinuous channels on wide flood plains introduce a special case for which the design procedure may be inadequate because of the uncertainty regarding flow distribution at any cross-section.
- Island or major obstructions in the main channel at or upstream of the bridge may necessitate modifications of the procedure.
- Where the river flow is split between a number of bridges the flow should be divided properly between each bridge. The Bureau of Reclamation (1970) mentions a method by Matthai for apportioning flow whilst the method of Davidian et al. was used by Colson and Schneider (1983).
- The BPR method is reported (Colson and Schneider 1983) to significantly undercompute the backwater on wide flood plains, apparently due to underestimation of friction losses.

3.6 Other methods

In view of the limitations and assumptions inherent in the method outlined above, a list of references to other methods is given here which may assist the designer where he feels the BPR method is inapplicable or he would like to check his result with a second method :

- Du Aubuisson's equation (described in Chow 1959, Webber 1965)

This simple equation is based on flow constricted by piers. It is derived from energy principles assuming no recovery of head after the constriction and its use is therefore limited to flows with high turbulence at the constriction.

As it is useful for a preliminary estimate of the required waterway width, the equation is given below:

$$Q = C_A b y_3 (2gh_1 + V_1^2)^{1/2}$$

where

- Q = discharge (m³/s)
- C_A = a coefficient which can be between 0,96 and 1,31 for piers but which can be assumed as 1 for preliminary estimates of constriction widths
- b = constriction width normal to the flow (m)
- y₃ = downstream depth i.e. normal depth (m)
- V₁ = upstream velocity for backed up depth of flow (m/s)

- Nagler's equation (described in Chow (1959) and Henderson (1966))

Slightly more complex than Du Aubuisson's, this equation is also based on pier constrictions but can be used where lower energy losses are involved since it allows for some recovery of head.

- Yarnell's equation (described in Chow (1959) and Henderson (1966))

Also based on pier constrictions, Yarnell's equation and charts give a quick solution for both sub- and supercritical flows through pier constrictions.

- Dubuat's method (described in Cape Provincial Administration, 1983)

This rough but quick method is limited to small, relatively confined streams.

- Kindsvater, Carter and Tracy's methods (described in Chow (1959))

This method gives more exact results than the above discharge equations. Unlike the BPR method, the assumption of normal flow prior to constriction is not made due to the use of a horizontal flume in the experimental setup. The Bureau of Reclamation (1970) suggests use of this method as an alternative for abnormal flow.

- Skogerboe et al. method (1970)

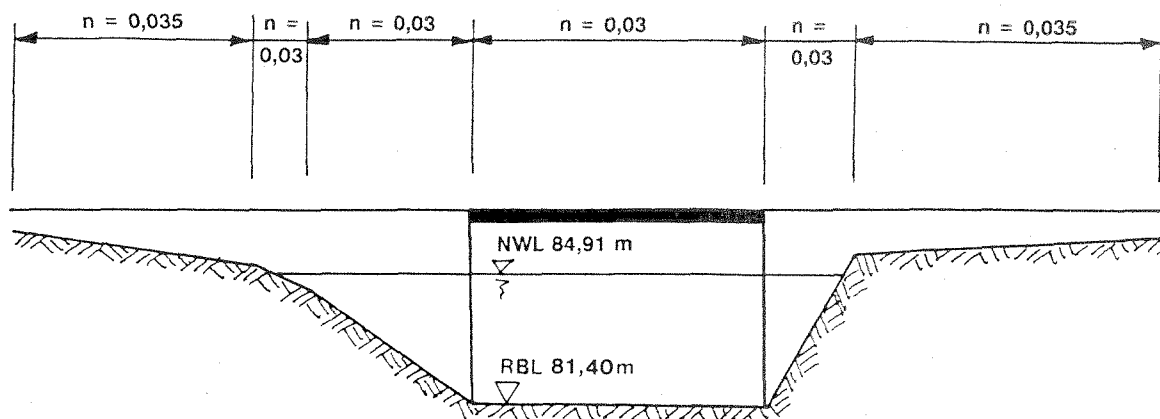
A method is described for constrictions where abnormal stage conditions exist prior to the introduction of the constriction.

■ Colson and Schneider's methods (1983)

The case where a number of separate bridge openings are involved is considered here. Two methods are compared using data collected from nine floods. Reference is also made therein to a study of width constrictions on heavily vegetated flood plains.

In a recent study (Basson 1991) it was found that the BPR equations provide realistic answers in cases where the bridge opening ratio $M < 0,85$. However where $M > 0,85$ i.e. where a bridge causes little contraction of the approach flows, this equation leads to underestimation of damming and the Du Aubuisson equation tends to provide more realistic answers.

EXAMPLE CALCULATION



Design discharge	$Q = 191 \text{ m}^3/\text{s}$
Average bedslope	$S_o = 0,00082 \text{ m/m}$
Angle of skew	$\phi = 15^\circ$
Bridge span on skew	$b_s = 17,6 \text{ m}$
Projected bridge span	$b = 17,0 \text{ m}$
No of rows of piers	$N_p = 1$
Projected width of pier	$W_p = 2,00 \text{ m}$

1. Calculation of unconfined flood state:

Sub-section	n	a (m ²)	p (m)	m = $\frac{a}{p}$ (m)	k = $\frac{am^{2/3}}{n}$	q = kS ₀ ^{1/2}	v = $\frac{q}{a}$	qv ²
1	0,035	0	-	-	-	-	-	-
2	0,030	0,50	2,49	0,20	5,72	0,16	0,33	0,02
3	0,030	25,07	12,43	2,02	1 334,00	38,20	1,52	88,69
4	0,030	59,67	17,00	3,51	4 593,81	131,55	2,20	639,33
5	0,030	14,68	7,91	1,86	738,57	21,15	1,44	43,93
6	0,035	0	-	-	-	-	-	-
		99,92			6 672,09	191,06		
		(A _n = Σ a)		(K = Σ k)		(Q = Σ q)	(Σ q v ²)	

Flood stage level = 84,91 m

Width at flood stage = 38,32 m

2. Determine flow type:

$$Fr_n = \left[\left(\frac{Q^2 B}{g A_n^3} \right)^{1/2} - \left(\frac{191^2 \times 38,32}{9,81 \times 99,92^3} \right)^{1/2} \right]$$

$$= 0,38 < 1$$

Flow is Type I or Type II.

Calculate specific energy of unconfined normal flow:

$$y_n = \text{Flood stage level} - \text{river bed level}$$

$$= 84,91 - 81,40$$

$$= 3,51 \text{ m}$$

$$V_n = \frac{Q}{A_n} = \frac{191}{99,92}$$

$$= 1,91 \text{ m/s}$$

$$E_{sn} = y_n + \frac{V_n^2}{2g} = 3,51 + \frac{1,91^2}{2 \times 9,81}$$

$$= 3,70 \text{ m}$$

Calculate specific energy of constricted flow critical depth:

$$y_{2c} = \left[\left(\frac{Q^2}{g b^2} \right)^{1/3} - \left(\frac{191^2}{9,81 \times 17^2} \right)^{1/3} \right]$$

$$= 2,34 \text{ m}$$

$$V_{2c} = \frac{Q}{y_{2c}b} = \frac{191}{2,34 \times 17}$$

$$= 4,80 \text{ m/s}$$

$$E_{sc} = y_{2c} + \frac{V_{2c}^2}{2g} = 2,34 + \frac{4,8^2}{2 \times 9,81}$$

$$= 3,51 \text{ m} < E_{sn} \text{ indicating Type I flow.}$$

Because the values of E_{sn} and E_{sc} are fairly close, and other losses are so far ignored, it would be prudent to check Type I and Type II flow.

3. Calculate bridge opening ratio:

$$M = \frac{Q_b}{Q} = \frac{131,55}{191}$$

$$= 0,69$$

4. Calculate velocity head coefficients:

$$\alpha_1 = \frac{\Sigma(qv^2)}{Q V_n^2}$$

$$= 1,11$$

$$\alpha_2 = 1,08 \text{ (from Figure 3.5)}$$

5. Calculate backwater:

5.1 For Type I flow

$$A_p = N_p \bar{W}_p = N_p \frac{A_{n2}}{b} W_p$$

$$= \frac{1 \times 59,67 \times 2,00}{17}$$

$$= 7,02 \text{ m}^2$$

$$J = \frac{A_p}{A_{n2}} = \frac{7,02}{59,67} = 0,12$$

Eccentricity

$$e = 1 - \frac{Q_a}{Q_c} = 1 - \frac{21,15}{38,20}$$

$$= 0,45$$

From Figure 3.3, $K^* = 0,78$

Approximate backwater

$$h_1^{*1} = K^* \alpha_2 \frac{V_{n2}^2}{2g}$$

$$V_{n2} = \frac{Q}{A_{n2}} = \frac{191}{59,67}$$

$$= 3,20 \text{ m/s}$$

$$h_1^{*1} = 0,78 \times \frac{1,08 \times 3,20^2}{2 \times 9,81}$$

$$= 0,440 \text{ m}$$

$$\begin{aligned} A_1 &= A_n + h_1^{*1} B = 99,92 + 0,440 \times 38,32 \\ &= 116,78 \text{ m}^2 \end{aligned}$$

$$h_1^* = h_1^{*1} + \alpha_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \quad (\text{Eqn. 3.1})$$

$$= 0,44 + 1,11 \left[\left(\frac{59,67}{99,92} \right)^2 - \left(\frac{59,67}{116,78} \right)^2 \right] \frac{3,20^2}{2 \times 9,81}$$

$$= 0,50 \text{ m}$$

5.2 For Type II flow

$$\begin{aligned} b_c &= (b - \Sigma W_p) = 17,0 - 2,00 \\ &= 15,00 \text{ m} \end{aligned}$$

$$C_b = 0,16 \text{ from Figure 3.8}$$

$$y_{2c} = \left(\frac{Q^2}{g b_c^3} \right)^{1/3} = \left(\frac{191^2}{9,81 \times 15^3} \right)^{1/3}$$

$$= 2,55 \text{ m}$$

$$y = \frac{A_{n2}}{b} = \frac{59,67}{17}$$

$$= 3,51 \text{ m}$$

Take

$$V_1 = \frac{Q}{A_n} = \frac{191}{99,92}$$

$$= 1,91 \text{ m/s}$$

for V_{2c} based on the net width:

$$\begin{aligned} V_{2c} &= (g \cdot y_{2c})^{0,5} = (9,81 \times 2,55)^{0,5} \\ &= 5,00 \text{ m/s} \end{aligned}$$

$$\begin{aligned} h_1^* &= \alpha_2 \frac{V_{2c}^2}{2g} (C_b + 1) - \alpha_1 \frac{V_1^2}{2g} + y_{2c} - \bar{y} \\ &= \frac{1,08 \times 5,0^2 (0,16+1)}{2 \times 9,81} - \frac{1,11 \times 1,91^2}{2 \times 9,81} + 2,55 - 3,51 \\ &= 1,60 - 0,21 + 2,55 - 3,51 \\ &= 0,43 \text{ m} \end{aligned}$$

Setting

$$\begin{aligned} A_1 &= A_n + h_1^* B = 99,92 + 0,43 \times 38,32 \\ &= 116,40 \text{ m}^2 \\ V_1 &= \frac{191}{116,40} \\ &= 1,64 \text{ m/s} \\ h_1^* &= 1,60 - \frac{1,11 \times 1,64^2}{2 \times 9,81} + 2,55 - 3,51 \end{aligned}$$

Although the difference in this case is negligible, to be conservative the higher value should be used, i.e. taken $h_1^* = 0,50 \text{ m}$.

4. SEDIMENT TRANSPORT AND SCOUR

4.1 Undisturbed sediment transport patterns

4.1.1 Incipient motion

Rapid progress has been made recently in the development of relationships which describe sediment transportation and deposition. Such relationships have to allow for the intricate interaction which occurs between fluids and transported sediment and it is therefore understandable that it has taken so long for this subject to progress beyond simple empirical relationships which are not universally valid.

Relationships do exist which indicate at what stage a stream will begin to erode its bed and banks and the maximum load of sediment that a stream is capable of carrying.

The classical and best known criteria depicting the boundary between flow conditions under which cohesionless bed material will be transported and those under which such material will not be transported are the Hjulstrom (1935) and Shields (1936) diagrams.

Hjulstrom's diagram simply relates critical velocity to particle diameter and does not provide an accurate criterion. The Shields diagram was developed on the basis of dimensional analysis of certain variables involved in sediment transport. Primary variables in the Shields analysis are: bed shear stress, particle diameter, particle density, fluid density and fluid viscosity.

Its main shortcoming is that particle grain size is neither a truly representative nor a unique measure of transportability. In certain practical situations e.g. where artificial armouring units are present, particle size becomes a meaningless concept. The settling velocity of particles is a more significant and unique measure in the case of non-cohesive material, while cohesive forces play a determining role in the entrainment of cohesive materials.

Incipient motion can be analysed comprehensively in terms of power considerations (Rooseboom and Mülke 1982).

Relatively simple relationships have been derived (Figure 4.1) which define critical flow conditions. As long as turbulent boundary layer conditions exist along on even bed movement of bed material will take place if

$$V_* = \sqrt{gDS} > 0,12 V_{ss}$$

where

D	=	depth of flow (m)
S	=	energy gradient
V_{ss}	=	settling velocity of bed material elements (m/s)
V_*	=	so-called shear velocity (m/s)

Turbulent boundary conditions along an even bed are ensured if the bed particles are larger than 5 mm.

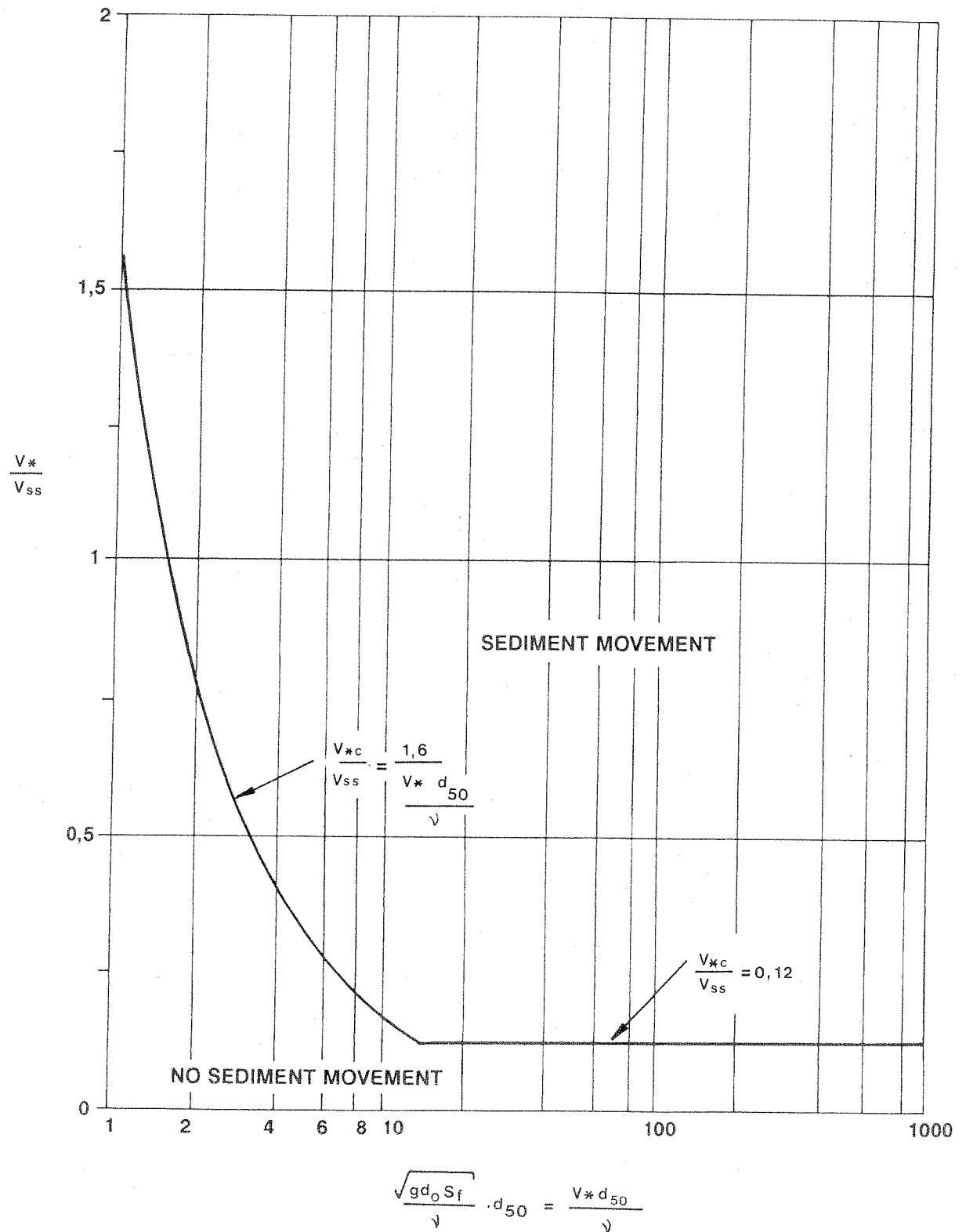


FIGURE 4.1 : INITIATION OF SEDIMENT MOVEMENT (ROOSEBOOM & MÜLKE)

The value of $\sqrt{gDS}$ can be derived from the Chézy equation which reads

$$V = C \sqrt{RS}$$

where

- V = average velocity (m/s)
- C = roughness coefficient
- R = hydraulic radius (m)
- ≈ D, depth of flow in a wide stream

The criterion $\sqrt{gDS} > 0,12 V_{ss}$ is valid for cohesionless bed material including artificial armour units of any shape. As it is possible to determine the settling velocity of any element, the equation can be readily applied. This relationship can thus be used for the design of bed protection units.

4.1.2 Sediment loads

Where rivers carry a major portion of their sediment loads in suspension, as in the majority of southern African rivers, the maximum observed value of the sediment load for a given discharge value can be more than a thousand times larger than the minimum observed value for the same discharge, especially at low discharge values.

Figure 4.2 illustrates the typical scatter of mean sediment concentration (daily observations) plotted against corresponding discharge.

The points shown represent values recorded at Jammersbergdrift on the Caledon River. Close examination of comprehensive sets of data such as those shown in Figure 4.2 leads to the conclusion that the sediment load of a river that carries a large amount of fine material in suspension depends strongly on antecedent conditions. Relatively small flood flows tend to carry large quantities of sediment after lengthy dry periods. On the other hand, high flood flows after lengthy wet periods tend to carry relatively smaller sediment loads.

Whilst there is no singular relationship between discharge and sediment load, it becomes even more difficult to predict what the carrying capacities will be during a given flood event and what the scouring capacity of the stream will be as well as to what extent general lowering of the bed profile will take place.

4.1.3 General scour

It is common practice to add different scour components in order to obtain the total scour depth at bridge piers and abutments.

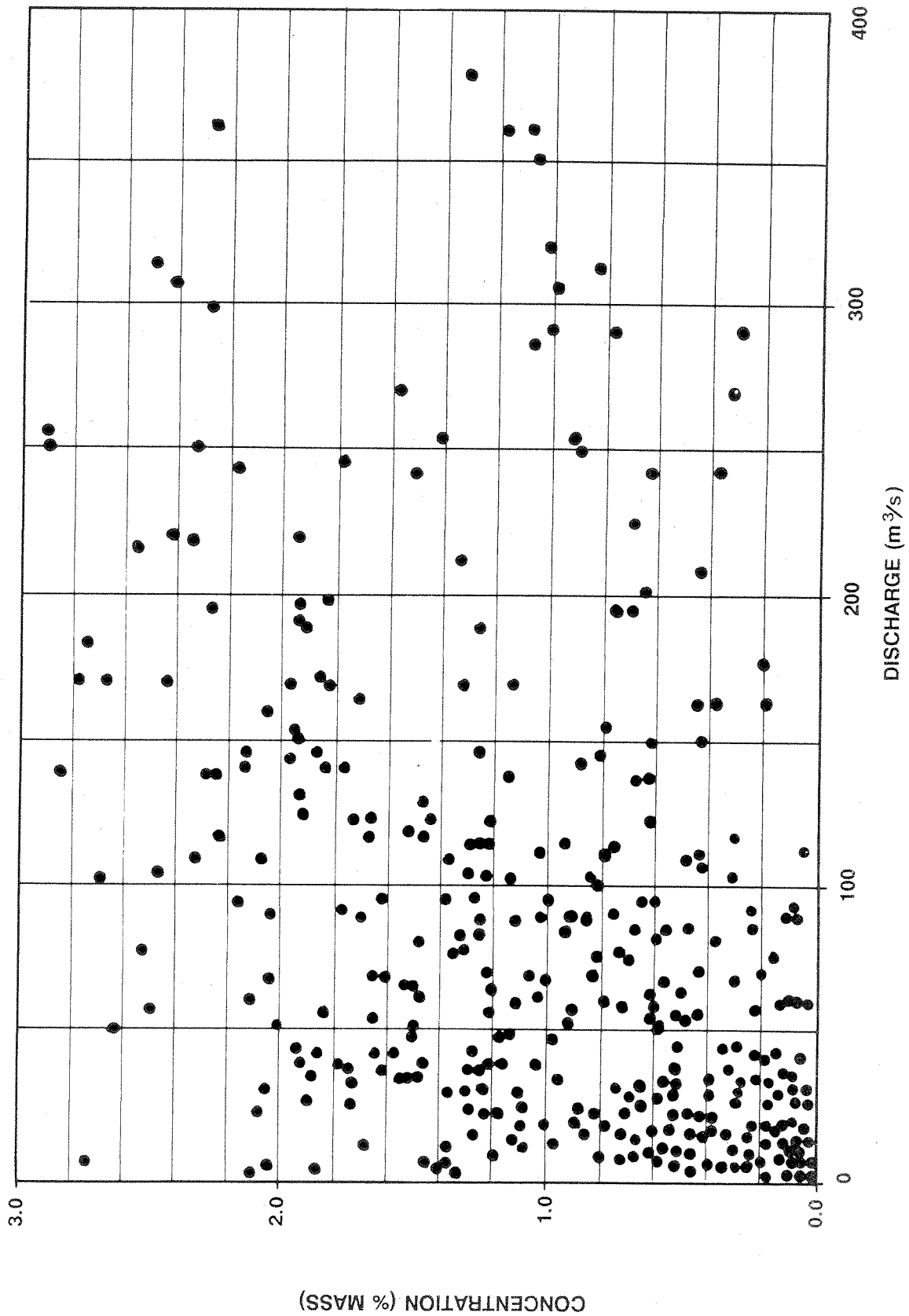


FIGURE 4.2 : CALEDON - JAMMERSBERGDRIFT SEDIMENT CONCENTRATION VS DISCHARGE
MAY 1969 - MAY 1976

Total scour is taken as the sum of:

- Long term general scour,
- Short term general scour,
- Constriction scour, and
- Local scour.

Long term general scour refers to scour which leads to a permanent general lowering of the bed. This occurs for instance downstream of a new dam and calculations can be performed by means of techniques as described in the International Commission on Large Dams' (ICOLD) Guidelines on Sedimentation Control of Reservoirs (1989). It must be remembered that whilst scour takes place in the region below a dam, general aggradation or bed build-up often occurs further downstream where the sediment load of the stream has increased to the point that general deposition once again becomes possible. The build-up occurs mainly during periods when minor to medium floods occur i.e. floods which are strongly attenuated by the reservoir upstream.

Hopkins et al. (1980) have come to the conclusion that

"The most significant contribution to total scour at a bridge comes from general or contraction scour. It is concluded that local scour by itself, typically would not be of sufficient magnitude to explain bridge failures. The results of this study indicate that local pier scour accounts for less than 20% of the total scour depth."

Short term general scour calculations tend to be based on equilibrium depth and width calculations.

The most widely used relationships for calculating equilibrium river widths and depths have been based on the work of Blench (1969).

It is necessary to calculate equilibrium widths before equilibrium depths can be calculated.

According to the Blench relationships, equilibrium widths are given by the following equation:

$$B = 14Q^{0,5} D_{50}^{0,25} F_s^{-0,5}$$

where

- B = equilibrium width (m)
- Q = discharge (m³/s)
- D₅₀ = median particle size (m)
- F_s = empirical coefficient
- = 0,1 for sandy loam
- = 0,2 for sandy clay loam
- = 0,3 for cohesive soils

Similarly, equilibrium flow depths are given by the equation:

$$y = 0,38 q^{0,67} D_{50}^{-0,17}$$

where

$$\begin{aligned} y &= \text{depth of flow (m)} \\ q &= \text{discharge per unit width (m}^3\text{/s.m)} \\ D_{50} &= \text{median particle size (m)} \end{aligned}$$

The equilibrium width and depth values refer to constant discharge conditions and may therefore be taken as upper limits under short term flood conditions. Equilibrium depths are very sensitive to changes in discharge per unit width and this factor alone creates serious problems when calculations have to be performed for South African rivers where the effective flow width past bridges tends to be ill-defined.

In the case of cohesive bed and bank materials Farraday and Charlton (1983) recommend that the following critical tractive stress values be used:

Type of soil	Critical tractive stress, N/m ²			
Sandy clay	1,9	7,5	15,7	30,2
Heavy clay	1,5	6,7	14,6	27,0
Clay	1,2	5,9	13,5	25,4
Lean Clay	1,0	4,6	10,2	16,8

TABLE 4.1 : PHYSICAL PROPERTIES OF CLAY

In order to obtain maximum channel depths, based on the equilibrium depths indicated above, the following multiplication factors, attributable to Lacey (1958), are still commonly used:

Location	Multiplying factor
Straight reach of channel	1,25
Moderate bend	1,50
Severe bend	1,75
Right-angled abrupt turn	2,0

TABLE 4.2 : MULTIPLYING FACTORS FOR MAXIMUM CHANNEL DEPTH

Having calculated equilibrium depths and having adapted these to allow for non one-dimensional effects, the scour depth may be calculated.

As an increasing discharge leads to both an increase in water surface level and a lowering of the bed at the same time, the scoured bed level is not uniquely defined in terms of the equilibrium flow depth.

Designers tend to calculate flood levels without allowance for scour and then to set the bed level at a distance equal to the equilibrium depth below the local water surface. The result should obviously be wrong where the bed level will drop over a long distance (downstream) leading to a general lowering of bed and water surface profiles.

According to a number of case studies in South Africa, the uncertainty regarding water surface levels is much less than that relating to maximum scour depths and it is therefore recommended that the water surface levels calculated by traditional means (Chapter 3) be accepted as the levels relative to which bed levels are to be calculated in terms of equilibrium depths for the purpose of calculating general short term scour.

Currently it must unfortunately be stated, that scour depths cannot be calculated with a great deal of accuracy and that all results should be considered in terms of maximum total scour depths which have been observed albeit indirectly (Section 4.3). The results of Standard Penetration Tests at bridge sites often provide indications of materials which have been disturbed in the past and can provide valuable indications of lateral and vertical channel deformations.

Bridge designers have in the past been mostly concerned with erosion in a vertical direction. The following list of main causes of serious hydraulic related problems at bridges in the USA (Brice 1983) however proves that lateral erosion and channel changes should receive much more attention in future. He lists the relative importance of different processes in hydraulic problems at bridges in the USA as follows:

Lateral erosion	40%
General scour	20%
Local scour	17%
Channel degradation	13%
Debris accumulation	10%

Ideally a bridge should have a minimum total span equal to the equilibrium width of the river as given above.

4.2 Bridge induced scour and deposition

4.2.1 Scour due to contractions

This refers to scour which is induced by a gradual increase in unit discharge due to a decrease in channel width, whilst the streamlines within the contracted reach are basically parallel.

The additional scour which is thus brought about is often calculated by means of (Rooseboom et al 1981):

$$\left(\frac{y_2}{y_1}\right) = \left(\frac{B_1}{B_2}\right)^{2/3}$$

Where y_2 and y_1 are the equilibrium depths within the contracted and uncontracted reaches and B_2 and B_1 are the corresponding channel widths. This equation is in effect a restatement of the equations used for calculating general scour in Section 4.1.3 in which equilibrium depth is also proportional to discharge per unit width to the two-thirds power.

$$\begin{aligned} \left(\frac{y_2}{y_1}\right) &= \left(\frac{B_1}{B_2}\right)^{2/3} \\ &= \left(\frac{Q.B_1}{Q.B_2}\right)^{2/3} \\ &= \left(\frac{q_2}{q_1}\right)^{2/3} \end{aligned}$$

It follows therefore that as long as the appropriate width is used in calculating general scour, no additional provision need to be made for scour due to general contraction.

4.2.2 Scour at piers and pile groups

Numerous formulae exist for the calculation of scour at bridge piers. Comparative studies such as those of Hopkins et al (1980) confirm the confusion as a result of the conflicting results which the different equations provide. They list the following results obtained by means of the different equations for a case where the measured scour depth below bed level was 3,0 m and the depth below water surface level 6,4 m.

Although certain formulae give results that tend to agree with the measured results in a specific case no single formula consistently produces reliable results.

It is believed that as in other sediment transport problem situations a formula is only valid for the conditions under which it was calibrated. The quantity of sediment which needs to be transported in order to create a hole of a given depth differs greatly from case to case so that even if equivalent floods caused scour in different situations (which is highly unlikely), the resulting scour depth is likely to differ significantly from case to case.

In the light of the findings by Mouton (1982) and Farraday and Charlton (1983) it remains our conclusion that the most reliable formulae for calculating scour at piers are those of Shen and Jain and Fischer. As the use of these formulae is set out comprehensively in the existing NTC Road Drainage Manual this information is not repeated here.

One consolation which relates to our inability to calculate local depths of scour around piers with confidence is that under severe flood conditions the local scour depths around piers in many cases plays a secondary role. Errors involved in calculating local scour around piers therefore tend not to be very significant in the total picture.

		Investigator	Scour depth
Predicted scour depth from	Mean-bed level	Bata	0,2
		Breusers	5,1
		Carstens	2,0
		Knezevic	0,4
		Larras	2,8
		Laursen (Design Curve)	4,9
		Laursen (Long Contraction)	4,0
		Liu	2,4
		Shen-1	2,0
		Shen-2	0,4
		Shen-3	2,7
	Water surface	Ahmad	3,3
		Blench-1	4,1
		Blench-2	6,5
		Blench-3	3,5
		Chitale	3,5
		Inglis-Poona-1	3,3
		Inglis-Poona-2	6,2
		Inlis-Lacey	7,3
		Varzeliotis	2,9

FIGURE 4.3 : SCOUR DEPTHS AS PREDICTED BY VARIOUS FORMULAS FOR THE RED RIVER

It is our conclusion that at this stage, that local scour around bridge piers cannot be calculated with a great deal of accuracy and that we will have to rely to a large extent on field observations to provide us with estimates of total scour depths (Section 4.3). By using field data for calibration purposes, the following advantages are obtained:

- Serious scale effects (i.e. false angles of repose, laminar boundary layer effects etc.) which play a role when model results are extrapolated, are eliminated.
- A total integrated picture is obtained without the need to differentiate somewhat artificially between the different scour components.

4.2.3 Scour at abutments and training works

It is even more difficult to quantify scour around abutments than in the case with piers especially as fewer model tests have been done in this case. Prototype information on scour depths around abutments is also much scarcer than in the case of scour around piers. There appears to be a measure of consensus that scour depths at abutments tend to be smaller than in the case of piers in their neighbourhood.

The equation in the NTC Road Drainage Manual after Laursen and Veiga da Cunha are recommended for use. Alternatively in agreement with Farraday and Charlton (1983) it is recommended that the additional scour depth attributable to abutments and training works be obtained by multiplying the value for general scour as calculated under Section 4.1 by one of the following factors from Blench (1969).

Nature of location	Multiplier
Nose of groynes or guidebanks	2.0 to 2.75
Flow impinging at right angles on bank	2.25
Flow parallel to bank	1.5 to 2.0

FIGURE 4.4 : MULTIPLIERS FOR ESTIMATING SCOUR DEPTH AT ABUTMENTS AND TRAINING WORKS

By recommending these values, we are again saying that most situations are so complex to analyse in detail and that we have to rely heavily on averaged empirical factors which have been derived from total depths of scour.

4.3 Combined scour

4.3.1 Introduction

The foregoing chapters have dealt with the conventional approach whereby a number of scour components are added to provide the total depth of scour.

Having worked through a number of overseas and local case studies, the following conclusions may be drawn regarding scour calculations:

- Due to the complications which have previously been mentioned the accuracy of scour calculations is rather low. Even amongst commonly used formulae calculated scour depths for a given situation may differ by a factor 2 or more.
- Maximum scour depths which have been observed under extreme flood conditions in models as well as in prototype situations often occur in between piers and not around piers as would be expected. This is in line with the observation that general scour becomes the predominant mechanism in determining maximum scour depths under extreme flood conditions i.e. those conditions that designers normally have to cater for.
- It is believed that a relationship which describes hydraulic conditions under which actual failures of piers have taken place as a result of scour provides a valuable yardstick in the calculation of maximum foreseeable scour depths.

4.3.2 Observed failure conditions

It is very difficult to record maximum scour depths at bridges under extreme flood conditions and for this reason, a limited amount of information is available. Valuable insights into likely scour depths may be obtained from standard penetration tests which are performed after the event but it is not always possible to relate a zone of low resistance to a single known flood event (Vol. II Section 3.1.6.(iv)(e)).

Recent extreme flood events (mainly in Natal) during which a number of bridges were damaged have been studied by H H Lotriet and A Rooseboom in order to develop a relationship which could be used for the design of minimum pier lengths.

In order to calibrate the relationship only those bridges where the following requirements were met were selected for calibration of the relationship:

- Only piers which were not supported on rock and which failed were considered.

- Only cases where piers that failed whilst the neighbouring pier or pier group remained intact were selected. It is trusted that this approach provides us with "critical" conditions i.e. conditions where failure just begins as opposed to cases where scour might have occurred well below the pier.

Piers or pier groups at the following bridges were accordingly selected for calibration purposes, having met the criteria:

- Mpambanyoni
- Mhloti
- Lovu
- Mfolozi
- Zwartkops

In the analysis of the data it was not possible to determine at what discharge a specific pier failed and the peak discharge, derived for the flood was accepted as the discharge when the pier failed.

The depth of scour was set equal to the length of pier below bed level. It may be argued that failure will likely take place, before the scour hole reaches the lower end of the pile. It is believed that whatever the mechanism of failure is, the total effect is accounted as the pier length is the most important variable in designs.

4.3.3 Critical flow conditions

Critical flow conditions at which non-cohesive sediment particles will begin to be eroded from a bed can be described by the relationship (Rooseboom and Mülke 1982)

$$\frac{\sqrt{gDS}}{V_{ss}} = \text{Constant}$$

for the case of uniform turbulent flows with turbulent boundary conditions,

where

- g = acceleration due to gravity (m/s²)
- D = flow depth (m)
- S = energy gradient
- V_{ss} = settling velocity of bed particles (m/s)

It is assumed that flow past a bridge pier under equilibrium scour conditions can be approximately described by this one-dimensional relationship. This relatively simple approach can be justified in terms of the predominant role that general scour plays when extreme floods occur and the fact that the flow patterns between piers do not deviate that much from one-dimensional flows.

In the case of one-dimensional flow past the piers, the above relationship can be re-written as follows:

$$\frac{C(y_o + y_s) V_{ss}}{q\sqrt{g}} = \text{Constant}$$

where

- C = Chézy roughness coefficient
- y_o = flow depth without scour (m)
- y_s = scour depth relative to undisturbed bed level (m)
- q = discharge per unit width between piers (m³/s)

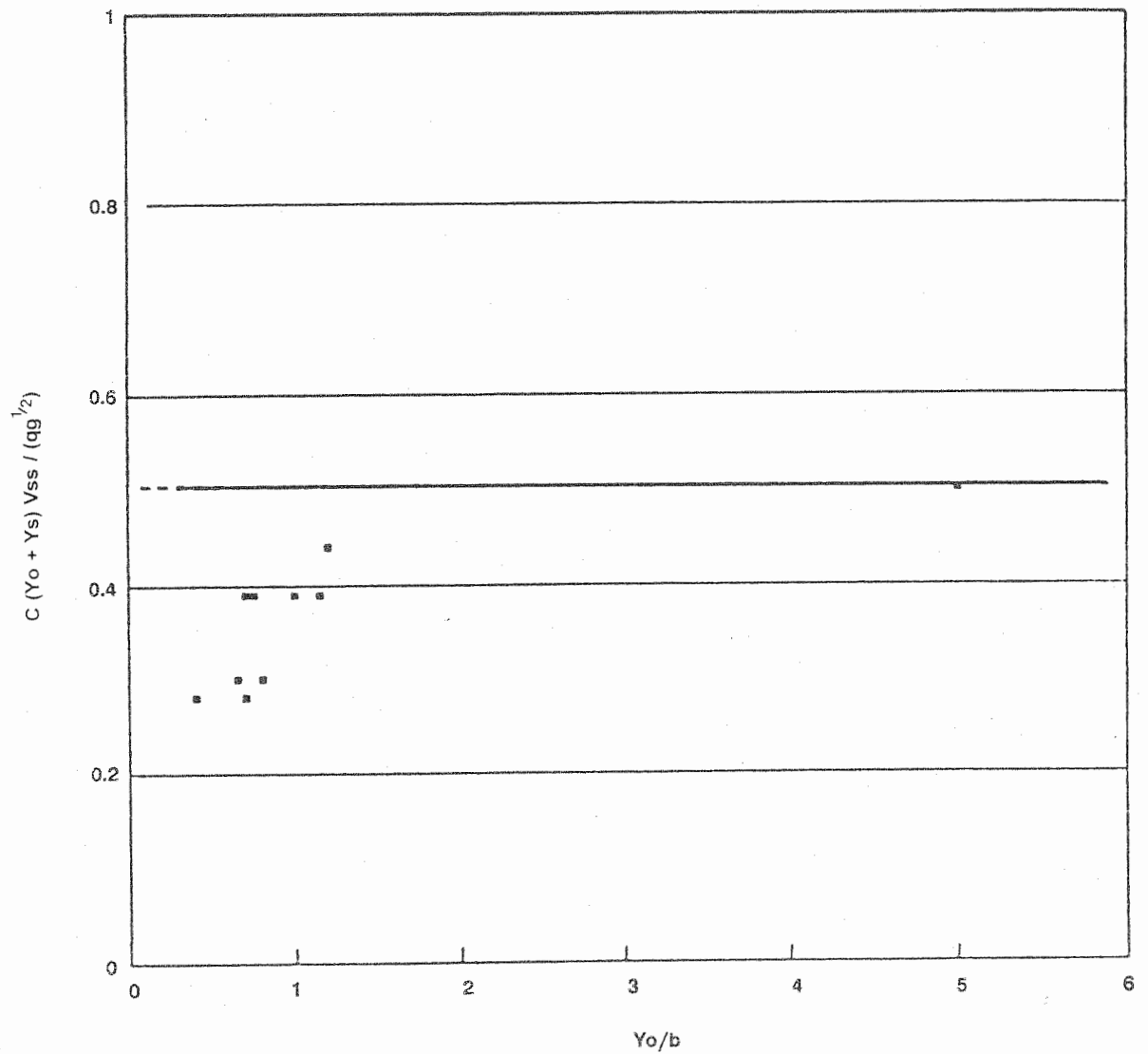


FIGURE 4.3 : ENVELOPE CURVE ENCLOSING RECORDED SCOUR DEPTHS AS DERIVED FROM PIER FAILURES

The values of this function have been plotted as a function of y_o/b where b , the pier width is regarded as the most significant parameter not included in the above relationship (Figure 4.3).

All the observed values for piers which failed and those adjacent piers which remained standing are included by an envelope curve with the equation:

$$\frac{C(y_o + y_s) V_{ss}}{q\sqrt{g}} = 0,5$$

4.3.4 Model data

Results of scour tests performed in hydraulic models are not always reliable. Two main scaling effects come into play

- Angles of repose which differ for prototype and model materials. This leads to scour hole depths which are not representative.
- Laminar boundary layer conditions in models, especially at the bottom of scour holes. Whilst laminar boundary conditions are unlikely to occur under prototype conditions they do occur in some models and lead to false results.

Using the Ettema model results, (Figure 4.4), maximum scour depths observed in models can also be included by the equation

$$\frac{C(y_o + y_s) V_{ss}}{q\sqrt{g}} = \text{Constant}$$

provided that $y_o/b > 4$ i.e. when pier widths are not significant relative to flow depths i.e. whilst flows are predominantly one dimensional.

The constant in this case comes out at 12 whereas the corresponding value for prototype data equalled 0,5. This difference can be attributed partially to the fact that single particle sizes are generally used in model studies. In the case of prototype data, the settling velocities were calculated for material obtained from the bed surface.

Excavations in river beds clearly indicate that the average particle sizes of the armouring layers which develop where scour reaches equilibrium are much greater than the particle sizes of the bed surface materials.

Furthermore, model results tend to represent equilibrium conditions i.e. maximum scour depths. It is unlikely that equilibrium conditions would be reached in practice.

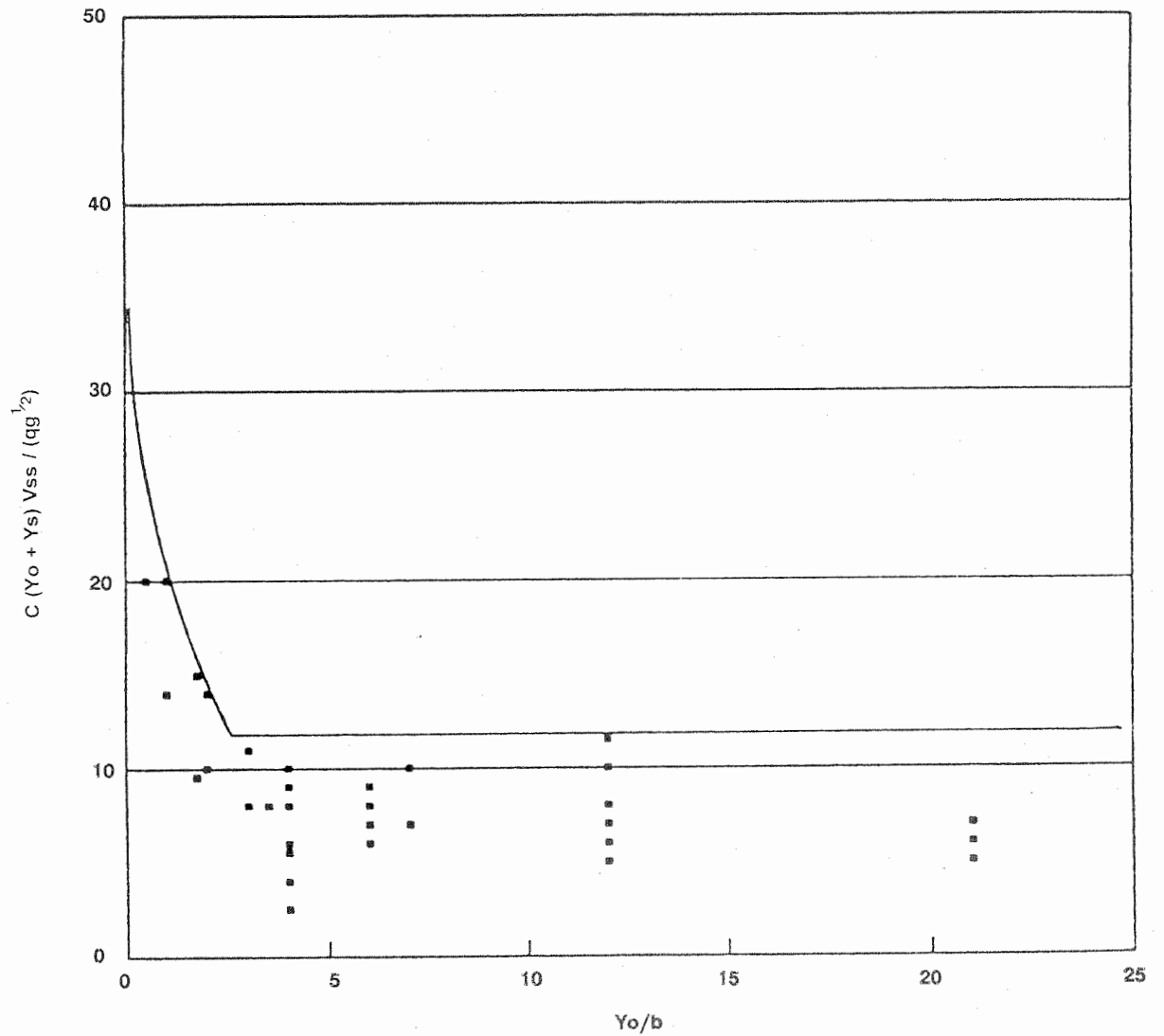


FIGURE 4.4 : ENVELOPE CURVE ENCLOSING MAXIMUM SCOUR DEPTHS
(ETTEMA MODEL DATA, 1980)

4.3.5 Conclusion

The simple relationship

$$\frac{C(y_o + y_s) V_{ss}}{q\sqrt{g}} = \text{Constant}$$

provides a useful estimate of the maximum scour depth which may be expected in terms of pier design. The value of the constant comes out at 0,5, if the surface grain size of sediments from the undisturbed river bed are used to calculate the representative settling velocity.

Not surprisingly, it has been found that scour depths which are calculated for Regional Maximum Flood discharges tend to coincide with the depth of unconsolidated bed material i.e. the "active" part of the bed. Such a check is useful in determining maximum foreseeable scour depths as well as for the interpolation of scour depths for lesser floods.

5. DESIGN OF BRIDGE WATERWAY OPENINGS

5.1 Introduction

Minimum dimensions of waterway openings have in the past generally been defined in terms of minimum spans (e.g. 7,5 m minimum where trees are brought down by floods) and standardized freeboard values (e.g. NTC Road Drainage Manual, Rooseboom *et al.* 1981).

Recent flood events have led to the conclusion that typical individual span lengths being used on major bridges are adequate. However as far as total opening widths and vertical freeboard are concerned, provision has often been inadequate.

The following chapters attempt to summarize the main conclusions to be drawn from the many case studies reported on in the literature.

5.2 Minimum opening widths

Lateral scour has been identified as the major cause of serious hydraulic problems at bridges in the USA (Brice 1983). In order to provide for lateral scour it is often necessary to provide additional spans to a bridge structure. Too often in the past the main channel(s) of a river as it was observed on a given day under low flow conditions was regarded to be "the river". As river reaches through erodible bed and bank materials tend to widen and deepen as well as change direction with increasing discharge (and vice versa) the total river must be considered when bridge openings are being designed.

Ideally, the total opening width should not be less than the equilibrium channel width corresponding with the design discharge (Section 4.1.3). The positions of bridge openings should also be in harmony with the flow patterns which prevail under design discharge conditions.

As the discharge increases, the major topographical features play an increasingly important role in determining flow patterns and channel positions and directions may change drastically as features such as large scale bends upstream begin to play a dominant role in the flow patterns.

The following tools can be used in assessing the extent to which channels may change their shape and direction under varying conditions:

- Bed material characteristics including particle sizes and Standard Penetration Test Values. As extreme scour depths could be in excess of 30 m in major South African rivers, this information may be required for large depths where rock surfaces are very deep.
- Overall channel characteristics with special consideration of bends upstream and the total widths of active floodplains.
- Historical records of changes in channel patterns. These records must be considered with circumspection as a river channel may appear to be "stable" in a given position for tens of years before undergoing drastic change. Historical records also tend to represent low flow conditions rather than the more important extreme flood events.

- Physical and mathematical models have the advantage that they provide insight into changes in flow patterns which are associated with changes in discharge. Although there can be doubts about the absolute values given by model scour studies the main patterns come out clearly.

Flow patterns at unfavourable bridge sites should be studied in detail **before** bridge openings are positioned. The bridge should fit in with the natural flow patterns in order to minimize construction and maintenance costs.

Moving a bridge a relatively small distance upstream or downstream or laterally can lead to substantial savings in costs. The one case where there is little merit in lengthening a bridge is on the inside of a bend in a river where sediment build-up tends to block the openings.

Opening widths between piers generally seem to be adequate in typical modern bridges and the only criterion which needs to be re-emphasised is that floating objects such as car wrecks and trees can pass through.

5.3 Vertical openings

Current practice generally provides for sufficient freeboard to pass floating objects, especially trees, through the structure when the design flood discharge is reached. Alternatively provision needs to be made for boats to pass underneath, where applicable.

No clear design standards have been developed for the case where floating debris is to pass over the bridge structure. From the limited information available it appears that the trend is to have the structure submerged by the time the five year return period flood is reached. Such bridges are provided with low parapets instead of rails in order to limit the trapping of floating debris. It is agreed that the flood level must be well above the deck when trees are being carried by the river and local evidence needs to be sought as to the level at which this happens in a given river, if possible.

In the case of bridges on critical routes and bridges across major rivers, consideration should be given to passing the regional maximum flood below the deck if this can be done at limited cost.

Additional freeboard needs to be provided where:

- Sediment build-up occurs along the upper reaches of a reservoir. Simple empirical relationships which express the equilibrium bed slope as a percentage of the original bed slope have proven to be unreliable. The final equilibrium profile can only be determined working backwards from the final sediment level at the dam wall by balancing the energy equation as well as equations which represent equilibrium sediment movement through the reservoir. It must be remembered that a substantial proportion of the sediment deposit caused by a dam is actually deposited upstream of the full supply level. This means that the sediment delta can increase flood levels above the initial levels determined by the dam far upstream of the full supply level.

- Whilst bed and bank scour tend to take place immediately downstream of a dam a dam can also cause sediment build up further downstream. This happens during periods when floods are significantly attenuated by the dam. Subsequent extreme floods which are not attenuated to the same extent will then be blocked and higher flood levels could ensue downstream of the dam.
- Sand spits or bars close off river mouths with increasing frequency due to increasing water utilization within catchments. Such bars across river mouths cause significant damming upstream before they are completely breached by floods. A reasonable method for calculating resulting maximum flood levels, is to assume the crest of a sand spit at its built up level and to calculate the flood level which coincides with a breached hole through the spit just big enough to let the flood pass through at a conservatively high velocity (= critical velocity for gravel (say) $\approx 1,8 \text{ m/s}$). This approach eliminates the need to predict the dimensions of the scour hole and has given realistic answers where the problem of sand transport lagging behind flood build-up has been tackled previously. Maximum water levels which will be reached before breaching takes place also need to be considered (Section 7.1.4).

5.4 Alignment of bridges

Typical equations for calculating local scour around bridge piers indicate that the depth of local scour is directly proportional to the projected width of the piers in the direction of flow.

Indications are that maximum scour depths under extreme flood conditions are not determined mainly by pier dimensions but rather by the magnitude of the discharge per unit width. Accordingly, the alignment of bridge piers influences depth of scour:

- Directly by causing local scour to increase proportionally to the projected pier area.
- Indirectly by affecting nett flow widths between piers and thus unit discharge whereby the depth of general scour is increased.

Forces which are exerted on bridge piers by flowing water also increase with increasing projected pier area. It is therefore of critical importance that bridge supports must be aligned with the flow direction.

It is thus advisable to limit the projected area of bridge piers by aligning the piers with the flow direction especially when the design discharge is approached. Flow direction can vary greatly with discharge and this should be considered. As local scour depths are determined by the projected pier widths at the levels where scour occurs these widths need to be used in calculations, and not the widths of the piers at higher levels, which often are much smaller.

5.5 Allowance for scour

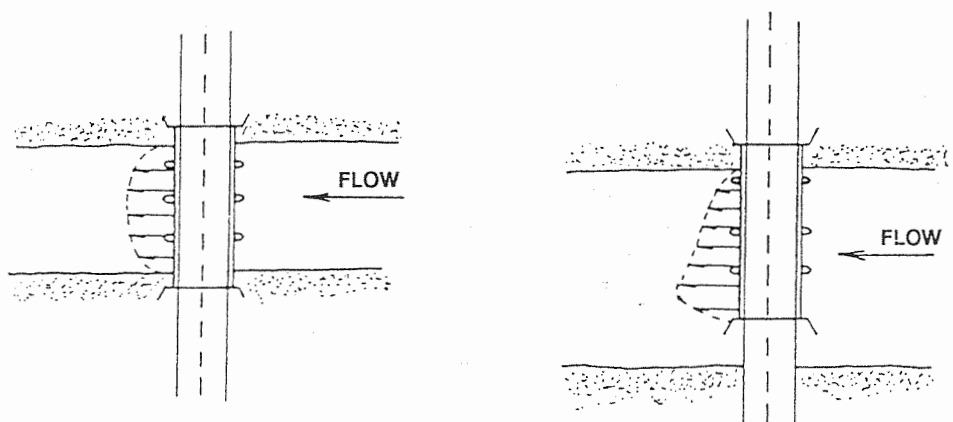
It would seem logical that in cases where severe scour is to be expected between bridge piers a smaller opening width would be required than in cases where no scour is to be expected. Investigation into damming recorded at bridges under extreme flood conditions Basson (1991) have shown that the calculation procedures recommended in Chapter 3 provide reliable estimates of damming height without allowance being made for scour. The conclusion is thus drawn that whether scour occurs or not, the

discharge capacity through a bridge opening is not greater than that indicated by the formulae in Chapter 3 even when scour occurs. The discharge capacity of bridge openings are therefore to be calculated as if no scour occurs.

6. HYDRAULIC FORCES ON BRIDGES

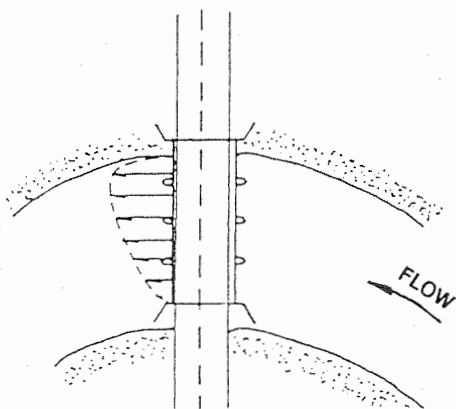
6.1 Problem statement

Apart from normal traffic loads recommended in TMH7 (1981), the stability of river bridges must also be checked for hydraulic forces caused by floods. Since, under flood conditions in South African rivers, the river bed, water levels and flow conditions are difficult to predict, it is even more difficult to determine the resulting hydraulic forces accurately and detailed hydraulic model tests may be required (Zwamborn 1981).



a) Bridge spanning total width of river.
Velocity distribution uniform.

b) Non-uniform velocity distribution due
to the asymmetric obstruction of the flow.



c) Non-uniform velocity distribution due
to helocoidal flow at the bend.

FIGURE 6.1 : THREE TYPES OF RIVER CROSSINGS

Depending on the situation and bridge layout, forces on bridge elements can be estimated theoretically. For simple bridge crossings (Figure 6.1a), the estimated forces can be calculated fairly accurately, but for asymmetric flow conditions (Figure 6.1b) and for bridges on a curve* (Figure 6.1c), these estimates become progressively less reliable. The following procedures are available to calculate the hydraulic forces using empirical coefficients provided in the literature (MTC 1983).

The following hydraulic forces may occur:

6.1.1 Horizontal forces

- Hydrostatic force due to difference in upstream and downstream water levels
- Dynamic forces (drag forces) due to the flow
- Forces due to debris caught by the bridge.
- Impact force due to floating debris.

6.1.2 Vertical forces

- Buoyancy forces
- Lift forces due to flow overtopping the bridge deck.

The hydraulic forces are by no means negligible. Particular attention must be paid to bridges built on long piles or caissons supported in erodible bed material which could scour tens of meters. As a result, the piles/caissons will be exposed and subjected to dynamic forces over a large portion of their length, providing a long moment arm and the possibility of pile vibrations.

6.2 Hydrostatic force

For economic reasons, bridges usually restrict river flows, resulting in a certain amount of backing-up so that there is a difference between the water levels upstream and downstream of the bridge (Chapter 3). To reduce both general and local scour at the abutments, the water level difference should be kept below a maximum of, say 0,5 m. However, when debris builds up against the bridge (which should be limited by adequate bridge span and freeboard design) the water level difference could increase.

The static force for a water level difference across the piers of $\Delta h = h_u - h_d$ as shown in Figure 6.2 is found from :

$$F_s = \frac{1}{2} \rho g (h_u^2 - h_d^2) w \quad (6.1)$$

where

- F_s = horizontal hydrostatic force acting on the pier (N)
- ρ = fluid density (kg/m^3)
- h_u = upstream water depth (m)

* This should be avoided, if possible (See Section 1.5)

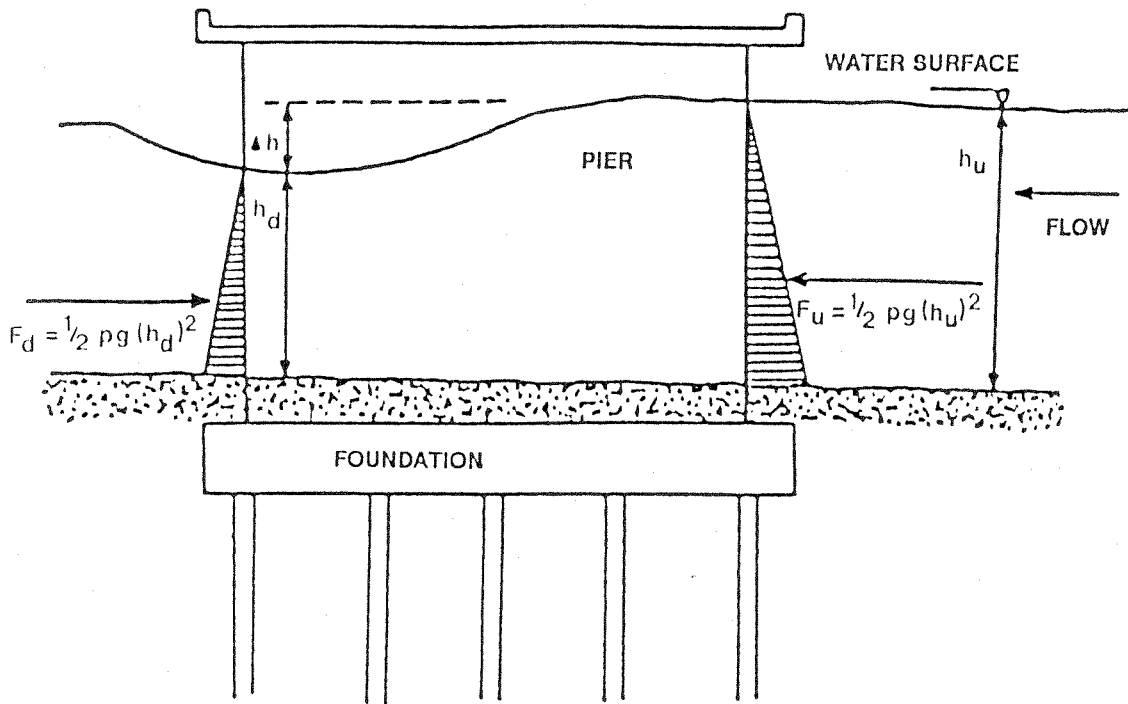


FIGURE 6.2 : HYDROSTATIC FORCE DIAGRAM

h_d = downstream water depth (m)
 w = width of the pier (m)

Thus for a bridge pier of 1,5 m wide, $h_u = 8$ m and $h_d = 7,5$ m:

$$F_s = \frac{1}{2} \times 1000 \times 9,8 (8^2 - 7,5^2) \times 1,5 = 57 \text{ kN}^{**}$$

F_s acts in the horizontal downstream direction. The hydrostatic force acts horizontally on about half the total submerged depth of the pier above the bed. This depth should include that part of the pier below the original bed level which will be exposed due to maximum scour (general river bed and local scour).

Strictly speaking, the water levels at the upstream and downstream ends of the pier will not necessarily be the same as the general upstream and downstream river levels. To determine these levels exactly, detailed model tests would be necessary; however, the above formula for F_s is considered sufficiently accurate for most cases.

^{**} 1 kN = 1000 N

6.3 Dynamic forces

Dynamic forces exerted on piers by a current can be separated into forces due to the flow component in the longitudinal direction of the piers and forces due to the lateral flow component, that is, the flow component perpendicular to the longitudinal pier axis (Figure 6.3). In general, the bridge piers should be aligned with the local flood flow direction to avoid lateral forces and minimize local turbulence and consequent scour. Different floods may cause different angles of approach and allowance must be made for the resulting angular approach.

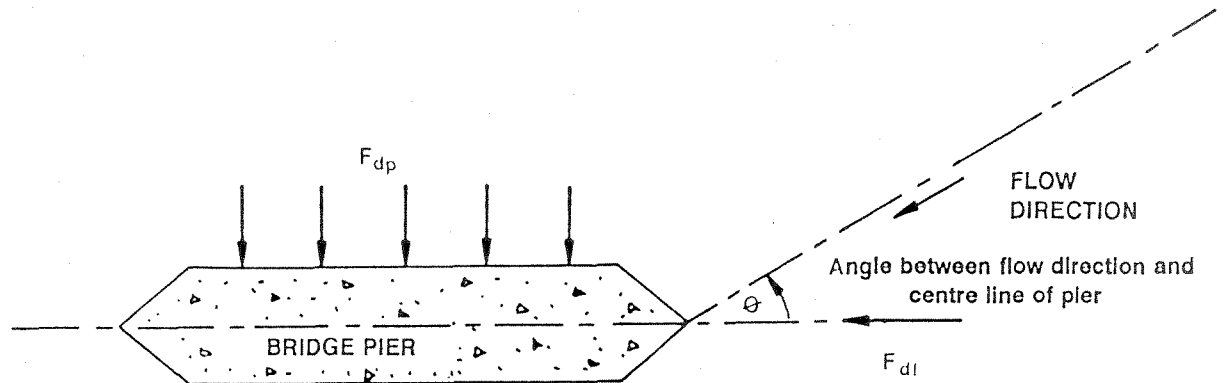


FIGURE 6.3 : DYNAMIC FORCES ACTING ON BRIDGE PIERS

The in-line drag forces are determined by the following equation:

$$F_{dl} = \frac{1}{2} C_{dl} A_1 \rho V^2 \quad (6.2)$$

where

- F_{dl} = horizontal force component in line with the pier axis (N)
- C_{dl} = longitudinal drag coefficient (given in Table 6.1)
- A_1 = submerged cross-sectional area of the pier and exposed part of the foundation projected normal to the flow direction (m^2)
- ρ = fluid density (kg/m^3)
- V = flow velocity through the constriction determined for the case of no scour (m/s)

Equation 6.2 applies for relatively small angles of the angle of approach, θ .

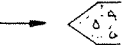
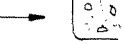
Type	Pier shape	C_{dl} ***
Semi-circular nose		0,7
Wedge-nose with nose angle < 90°		0,8
Square ended		1,4

TABLE 6.1 : DRAG COEFFICIENTS FOR DIFFERENT PIER SHAPES FOR FLOW IN THE LONGITUDINAL DIRECTION (MTC 1983)

For example, a bridge with wedge-nosed piers, with a width of 1,5 m each and a submerged height of 8 m, relative to the general scour level, subjected to a flow velocity of 5 m/s and directed 10° from the centre line of the pier, will experience a total in-line drag force on each pier which can be calculated from:

$$\begin{aligned}
 F_{dl} &= \frac{1}{2} \times 0,8 \times (1,5 \times 8 \times \cos \theta) \times 1000 \times 5^2 \\
 &= 118 \text{ kN}
 \end{aligned}$$

If the local flow direction is not in line with the bridge piers, the lateral drag forces, can be calculated from:

$$F_{dp} = \frac{1}{2} C_{dp} A_p \rho V^2 \quad (6.3)$$

where

- F_{dp} = horizontal force perpendicular to the pier axis (N)
- C_{dp} = lateral drag coefficient (given in Table 6.2)
- A_p = projected submerged lateral cross-sectional area of the pier and the exposed part of the foundation to general scour level (m²)
- V = flow velocity as determined for the longitudinal forces on the piers (m/s)

Angle between flood direction and pier centre line	C_{dp}
0°	0
5°	0,5
10°	0,7
20°	0,9
30°	1,0

TABLE 6.2 : LATERAL DRAG COEFFICIENTS FOR FLOW AT AN ANGLE TO THE CENTRE LINE OF THE PIER (MTC 1983)

*** These values agree with TMH7(1981), note that $C_{dl} = 2K$, e.g. "square ended" $C_{dl} = 2 \times 0,7 = 1,4$

For example, for a bridge with a pier length of 10 m and a height of 8 m, subjected to a flow velocity of 5 m/s at an angle of 10° to the centre lines of the piers, each pier will experience a lateral force:

$$\begin{aligned} F_{dp} &= \frac{1}{2} \times 0,9 \times (10 \times 8 \times \sin \theta) \times 1000 \times 5^2 \\ &= 159 \text{ kN} \end{aligned}$$

To determine the location of the drag forces on the bridge piers, it can be assumed that the velocity distribution is uniform over the upstream depth. This means that the force can be assumed to act at mid-water depth relative to the general scour level but the resulting moments on the bridge piers or pile groups have to be based on the expected local scour depth s (Figure 6.4), plus an allowance for fixity below the actual scour depth.

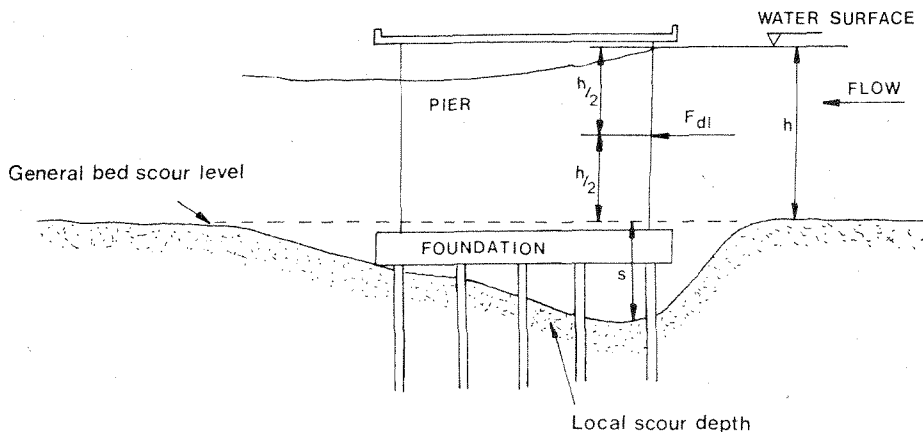


FIGURE 6.4 : POSITION OF IN-LINE DRAG FORCE ACTING ON THE PIER

6.4 Forces due to debris

Debris can cause additional forces on the bridge structure, directly, by additional drag forces and impacting bodies and indirectly, by decreasing the effective flow area and increasing the water level difference across the bridge (increased dynamic as well as static forces). The dynamic force could also be influenced by a change in velocity distribution resulting from blockage by debris.

It is virtually impossible to determine the effect of debris on forces accurately. Therefore, a blockage factor is assumed, based on experience with the river concerned. The vegetation of the catchment area is important in determining the type of debris to be expected, such as trees, sugar cane, weeds or grass. Reductions of up to 20 per cent in flow area have been observed in South Africa (Basson 1991).

In cases where the river is expected to carry large amounts of debris during floods, the horizontal forces can be calculated for the partly blocked bridge opening, using the dynamic force Equation 6.2 with $C_{dI} = 1,4$ and the static force Equation 6.1, with an upstream water level calculated for the partly blocked

bridge opening (Chapter 3). For the example given in Section 6.3, F_{dl} would increase from 118 kN to 207 kN, an increase of 89 kN.

Furthermore, because floating debris will collect near the water surface, the action line of the drag forces should be taken at a distance of one third of the upstream depth below the upstream water level.

6.5 Impact forces

Impact forces can occur as a result of debris colliding with the piers. For these forces to be calculated, the mass of the impacting body needs to be estimated. Therefore, the impact forces (e.g. of tree trunks) on any one pier are usually taken at between 90 and 180 kN (depending on the maximum size of possible debris) acting on the flood water level TMH7(1981). If no information on the size of debris is available, the higher value of 180 kN acting horizontally at the upstream water level, should be used.

6.6 Alternative method

Basson (1991) has investigated the hydraulic conditions at a number of bridges in South Africa under extreme flood conditions. By applying the momentum equation, using recorded peak discharges as well as flood level marks, he found that:

- Recorded differences in levels upstream and downstream of bridges were in reasonable agreement with those obtained by means of the calculation procedures quoted in Chapter 3.
- Total in-line drag forces, including the effect of debris, can be represented by Equation 6.2, adopting a drag coefficient of $C_{dl} = 3$ instead of the coefficients given in Table 6.1.

This method appears to be an appropriate and easy method of determining the order of magnitude of the total hydrodynamic forces on a bridge structure.

For example, for the abovementioned bridge, the total in-line drag force on each pier becomes:

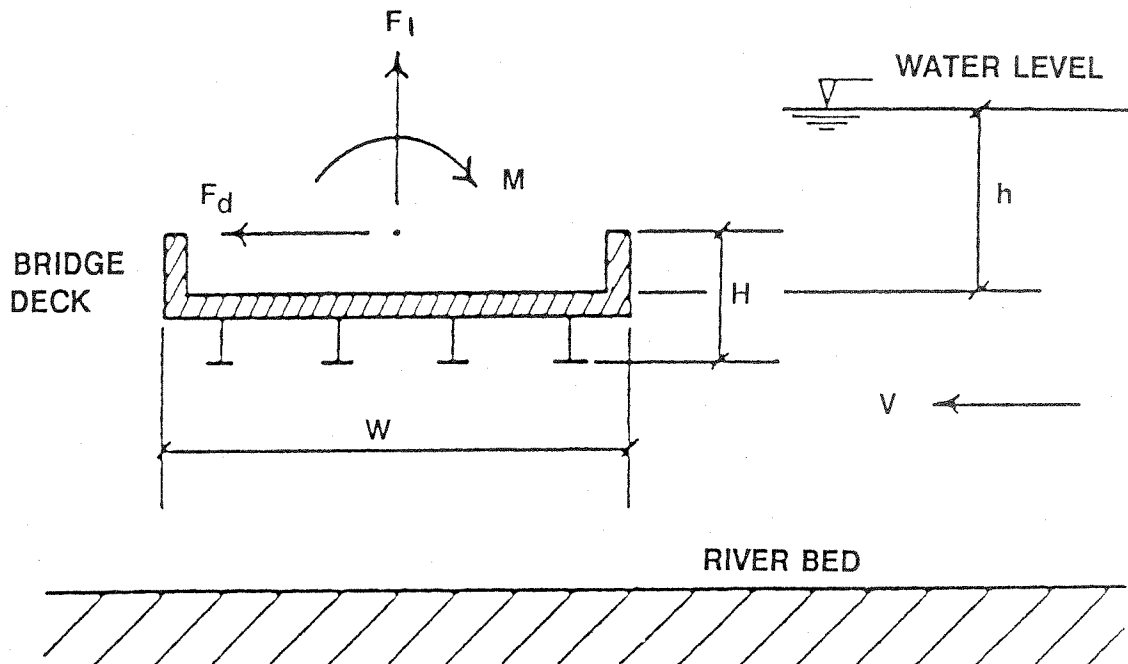
$$\begin{aligned} F_{dl} &= \frac{1}{2} \times 3 \times (1,5 \times 8 \cos 10^\circ) \times 1000 \times 5^2 \\ &= 443 \text{ kN} \end{aligned}$$

Compared with the sum of the static force of 57 kN (Section 6.2) and the dynamic force of 118 kN (Section 6.3) or 207 kN, with debris, the overall force calculated with the alternative method is 2,5 times greater (443/(57+118) or, in the case of debris 1,6 times greater (443/(57+207)).

6.7 Forces on overtopped bridges

In the case of low-level/overtopped bridges, considerably larger horizontal forces occur. Hydraulic forces on the bridge piers are dealt with as previously described in Section 6.2. This section deals with the possible forces on the bridge deck during overtopping and is based on some experimental work by the Mississippi University (MWRI 1982).

The three main forces on a bridge deck when inundated under flood conditions, are drag force F_d , lift force F_l , and overturning moment M (Figure 6.5).



Note : Arrows indicate positive directions for forces and moments

FIGURE 6.5 : DYNAMIC FORCES ON BRIDGES DURING OVERTOPPING

The drag force per unit length of bridge deck can also be determined according to the method presented in Section 6.2, from:

$$F_d = \frac{1}{2} C_d H \rho V^2 \quad (6.6)$$

where

- F_d = drag force per unit length of bridge deck (N/m)
- C_d = drag force coefficient, in the order of 2,0 to 2,7 (MWRRI 1982), or even up to about 3 (Basson 1991)
- H = total projected height of the superstructure, normal to the flow direction (m)

For example, for a bridge deck of total projected height of 0,6 m and a river flow velocity of 5 m/s, F_d , in an extreme case, becomes:

$$\begin{aligned} F_d &= \frac{1}{2} \times 2,7 \times 0,6 \times 1000 \times 5^2 \\ &= 20 \text{ kN/m} \end{aligned}$$

The total drag force is determined by integrating the unit drag forces over the length of the inundated bridge deck.

Similarly the lift force per unit length of the bridge deck can be determined from:

$$F_l = \frac{1}{2} C_l W \rho V^2 \quad (6.7)$$

where

$$\begin{aligned} F_l &= \text{lift forces per unit length of the bridge deck (N/m)} \\ C_l &= \text{lift force coefficient} \\ W &= \text{total width of bridge deck (m)}. \end{aligned}$$

Very little information is available on the value of C_l , which should be a function of the overtopping, the deck shape and the height of the bridge deck above the river bed. The only data on C_l found in the literature are published by MWRRI (1982). The reported C_l values of up to five, however, appeared unrealistic and research was initiated to check on these values for South African conditions.

Initial results from a comprehensive test programme, supported by the Department of Transport, showed mainly negative lift forces or **downward** forces on the bridge decks tested, opposite to the MWRRI results. These downward forces were found to **increase** with increased velocity and a decrease in clearance (bridge height). They were found to **decrease** with increased depth of overtopping. Values for the downward force coefficients of the order of 3 to 4 ($C_l = -3$ to -4) were found for typical South African bridge decks and river conditions which means that allowance has to be made for an additional vertical load due to overtopping in the design of the bridge foundations.

The model test results will be analysed to establish more reliable design data (force coefficients) for overtopping bridges.

The rolling moment per unit length on the deck due to the lift force is:

$$M = \frac{1}{2} C_m W^2 \rho V^2 \quad (6.8)$$

where

$$\begin{aligned} M &= \text{rolling moment on the deck (Nm/m)} \\ C_m &= \text{coefficient of rolling moment} \\ W &= \text{width of the bridge (m)} \end{aligned}$$

The above remarks about C_d apply equally to C_m . Values of $C_m = 0,2$ to $0,7$ are given again by MWRRI (1982) but these should be used with care until more data, from model force measurements, become available.

6.8 Overall design approach and load combinations

The above sample calculations show that the hydraulic forces play a significant role in the overall bridge-stability design, that is, the combined horizontal static and dynamic force on a pier 1,5 m wide and 8 m (submerged) high, could be between 175 kN and 443 kN, depending on the method used, compared with the accidental skidding force of 250 kN recommended in TMH7 (1981), in any direction, or the allowance for collisions against the balustrades of 50 kN.

For an initial design, the empirical "alternative method" (Section 6.6) could be used. However, for a more economical design, the forces should be calculated from the actual water levels and flow conditions. For more complicated situations, for instance a semi-submerged bridge, the forces on the bridge elements could be measured in a hydraulic model.

According to TMH7 (1981), the design forces due to flood action must be applied as follows:

Limit State	γ_{fl} to be considered in combinations :		
	1	2	3
Ultimate	-	1,05	1,3
Serviceability		0,85	1,0

TABLE 6.3 : LIMIT STATE FACTOR γ_{fl} TO BE CONSIDERED IN DESIGN

The load factors, γ_{fl} , are determined by the variation in the flood forces and the probability that these forces occur simultaneously with other forces (loads). Considering the five different hydraulic forces, namely:

- Static force
- Dynamic force
- Forces due to debris
- Impact force
- Overtopping force

and the uncertainties attached to the determination of these forces, it may be necessary to re-assess the values of the above partial load factors and possibly to determine separate factors for the different forces to arrive at a more realistic and economic design. The hydraulic model tests mentioned above should provide data on the expected variation in these forces and thus the values of the load factors.

7. SPECIAL ASPECTS

7.1 Tidal crossings

7.1.1 Problem statement

When a bridge is built across a tidal channel or estuary, allowance must be made in the design, not only for the effects of the normal river flow, but also for typical tidal effects, which include (Figure 7.1):

- Water level variations due to the tidal variations in the sea (up to 2 m in South Africa),
- Tidal flow in both directions with corresponding scour potential, particularly under spring tide conditions,
- Ocean wave action in the case of a wide estuary mouth.

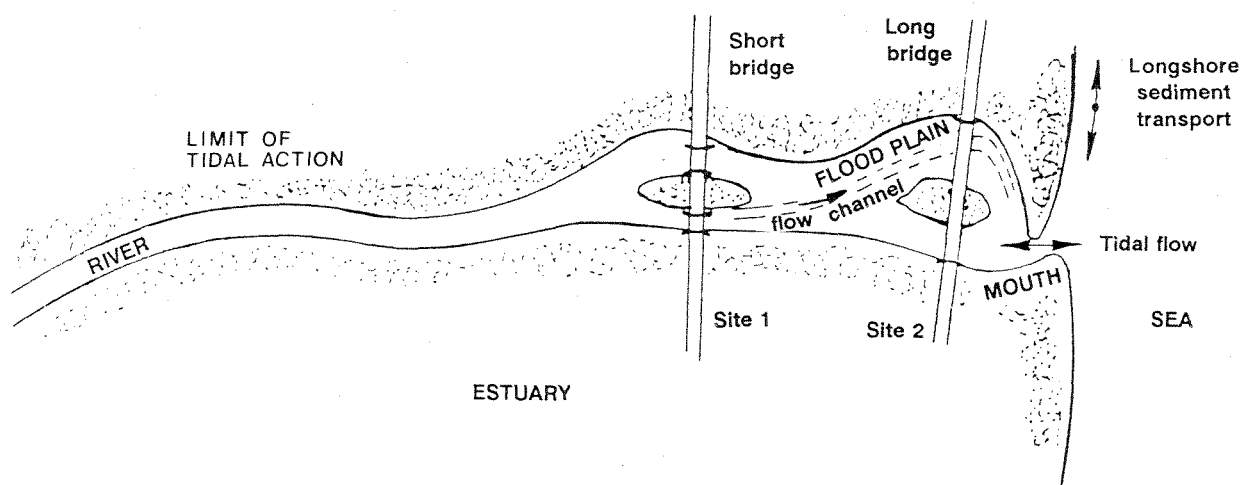


FIGURE 7.1 : TYPICAL SOUTH AFRICAN ESTUARY

As a result of the strong littoral sand transport along most of the South African coastline, estuaries tend to close up during the dry season, resulting in a reduced tidal effect. During the wet season, when the rivers come down in flood, the estuary mouths are scoured out, resulting in a much higher tidal range and associated larger tidal currents through the bridge openings. Thus, although peak river discharges will in most cases determine the hydraulic design of the bridge, tidal flow may also become important, particularly in the case of wide open estuaries and relatively small rivers.

Estuaries generally constitute ecologically sensitive wetland areas which should be interfered with as little as possible. Although a relatively small bridge opening combined with causeways may be adequate hydraulically in many instances ecological requirements will dictate a limited length of causeway across the wetlands and a larger bridge length to span the more important wetland areas. The bridge opening must also be large enough not to inhibit the incoming tidal flow through the bridge.

Bridge opening requirements with regard to recreational uses of estuaries are included in Section 7.2.2.

7.1.2 Tidal hydraulics

(i) Sea water levels

The range of the astronomical tides in the open sea around the South African coastline is about 2,0 m for an extreme spring tide, about 1,4 m for an average spring tide and about 0,6 m for an average neap tide. The mean water level of the open sea is about 1,1 m above Chart Datum (CD), while Chart Datum is 0,90 m below the national land-survey datum, called MSL (mean sea level). This means that the mean water level in the open sea is about 0,2 m above MSL (Figure 7.2).

For the calculation of extreme flood water levels the sea water level should be taken at high water spring tide plus a possible 0,5 m for storm set-up, that is, at +1,7 m MSL (worst case). For the calculation of extreme flow velocities the sea water level should be taken at low-water spring tide minus 0,5 m for set-down, that is, -1,3 m MSL. The sea tides form the driving force of water flowing into and out of an estuary as a result of the water level gradient.

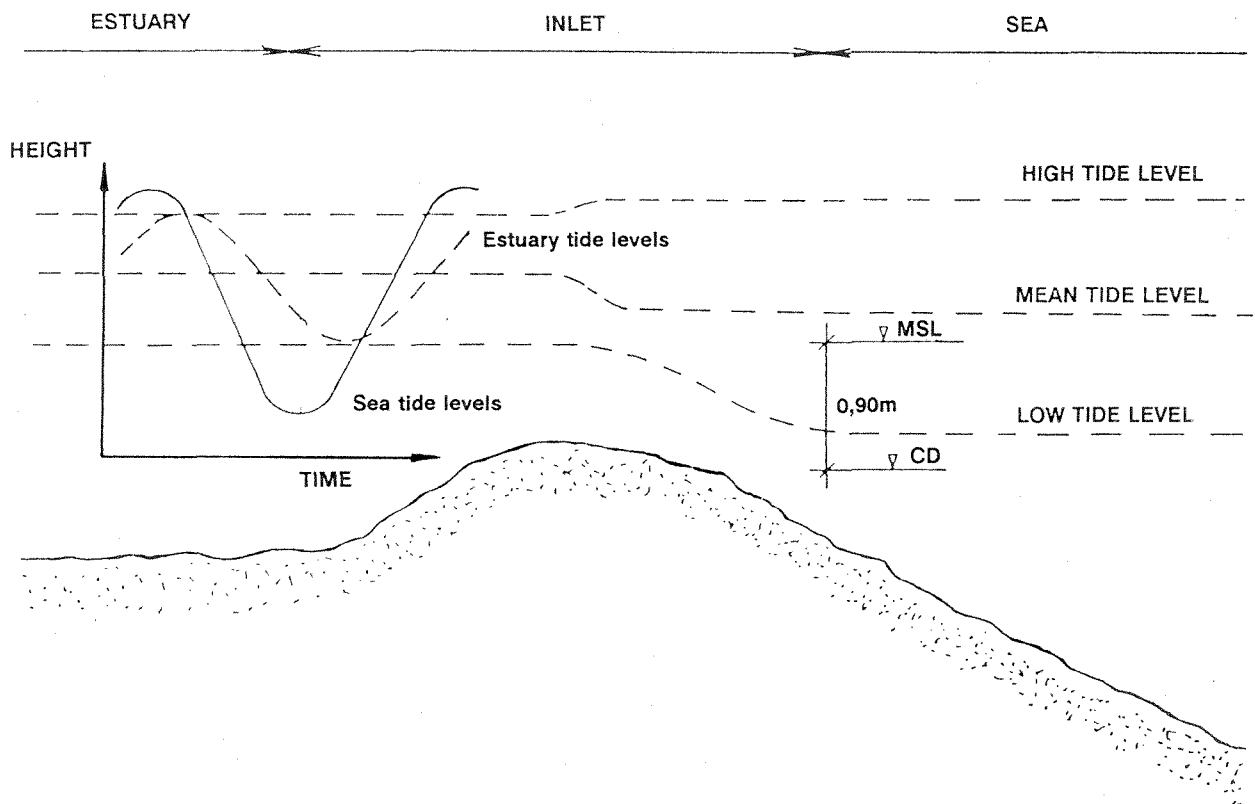


FIGURE 7.2 : WATER LEVELS AT EXTREME SPRING TIDE

(ii) Tidal flow through estuaries

The relationship between the maximum discharge through a tidal restriction and the volume of water stored upstream of the restriction per tidal cycle (the tidal prism) is:

$$Q_{\max} = \pi P/T \quad (7.1)$$

where

$$\begin{aligned} Q_{\max} &= \text{maximum discharge (m}^3/\text{s)} \\ P &= \text{tidal prism (m}^3) \\ T &= \text{tidal period (44 700 s)} \end{aligned}$$

The tidal prism is the volume of water between the high-water level and the low-water level in the estuary (Figure 7.3). In the case of additional river discharge, Q_r :

$$Q_{\max} = \pi P/T + Q_r \quad (7.2)$$

In the example given in Figure 7.3 the maximum tidal prism at the bridge site is 437 658 m³ for zero river discharge. If the river discharge is 6 m³/s then the tidal flow becomes:

$$Q_{\max} = \pi \times 437\,658 / 44\,700 + 6 = 37 \text{ m}^3/\text{s}$$

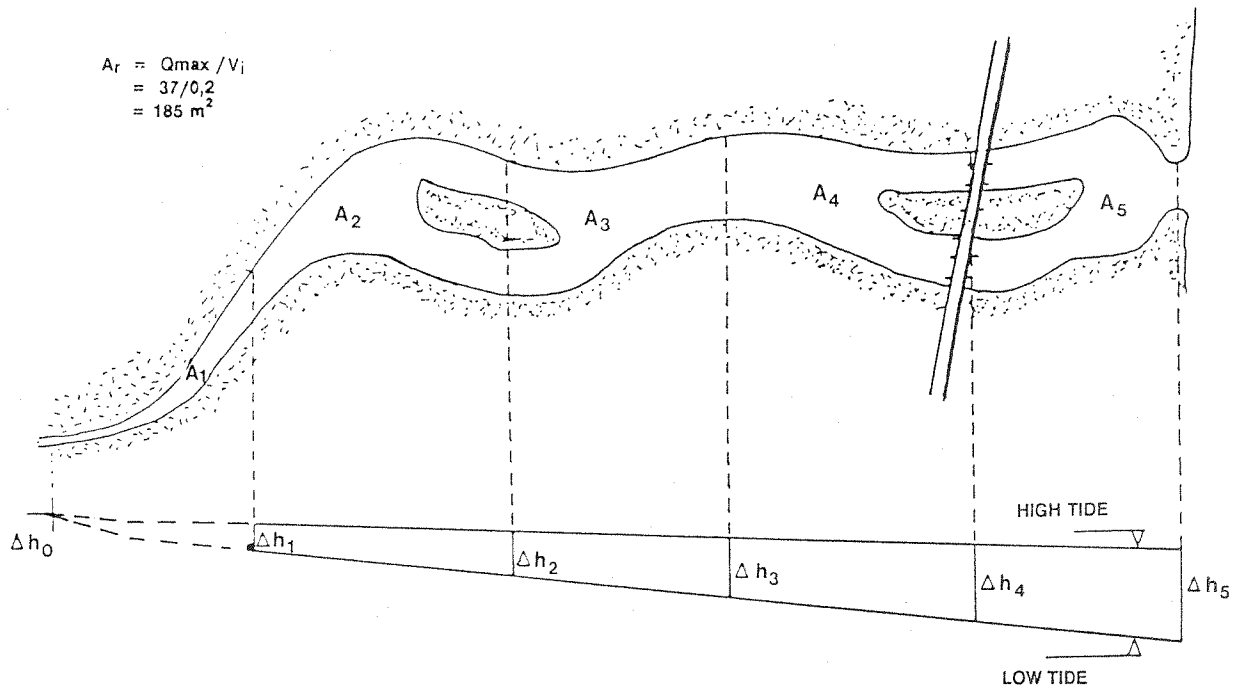
The computed discharge will form the basis for calculating an equilibrium flow profile for the bridge. For example, if the velocity for the initiation of motion of the local sediment is $V_i = 0,2 \text{ m/s}$ (Figure 7.4) and, if no scour due to tidal flow is to take place, the required flow area is:

$$\begin{aligned} A_r &= Q_{\max} / V_i \\ &= 37 / 0,2 \\ &= 185 \text{ m}^2 \end{aligned}$$

Since the maximum river flow during floods is normally an order of magnitude larger than the tidal discharge, the hydraulic design must in such a case be based on the river flood, ignoring tidal flow.

The above method can be used to give a rough indication of the tidal discharges. If the estuary area is more complicated or if the tidal water levels cannot be determined, a mathematical tidal model can be used to calculate the discharge, the flow velocities and the water levels (CSIR 1975). This is also recommended when flow restrictions, such as old bridges will be removed, changing the flow and the upstream tidal range, or when the depth or width of the estuary will be changed by the bridge crossing.

The tidal flow is mainly concentrated in the channels. The flood plain mainly acts during tidal conditions as a storage area, where the flow velocities are relatively small.



WATER LEVELS AT EXTREME SPRING TIDE

Section n	Δh_n (m)	$\bar{\Delta h}_n$ (m)	A_n (m ²)	$\Delta P_n = \bar{\Delta h}_n \cdot A_n$ (m ³)	ΣP_n (m ³)
0	0	0,10	100 000	10 000	10 000
1	0,21	0,27	308 460	83 284	93 284
2	0,33	0,41	231 870	95 067	188 351
3	0,48	0,57	437 380	249 307	437 658
4	0,66	0,78	398 340	310 705	748 363
5	0,89	TOTAL	1 476 050	748 363	

bridge mouth

FIGURE 7.3 : EXAMPLE OF COMPUTATION OF TIDAL PRISM

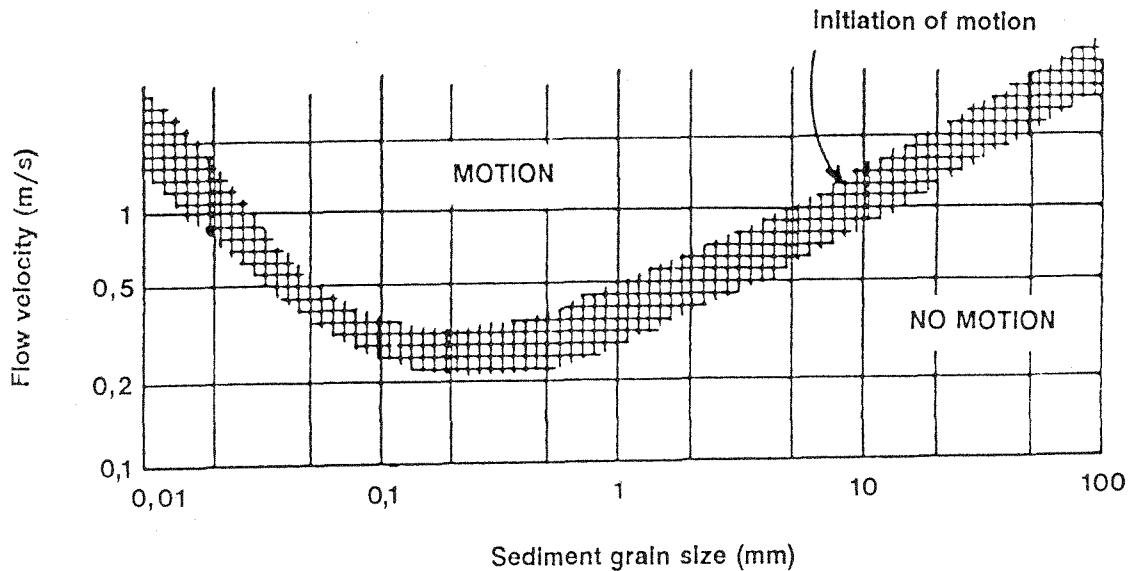


FIGURE 7.4 : INITIATION OF MOTION (Also see Figure 4.1)
(Schematized from ASCE, 1966)

7.1.3 Wave action

(i) Wind-generated waves

In addition to currents, waves are present in most estuaries. Wave action should, therefore, be considered at tidal crossings, mainly to determine the stability of the embankments and to check on scour and deposition.

For a bridge crossing close to the sea the source of local wave action may be sea waves propagating through the inlet into the estuary. However, most South African tidal inlets are so shallow and restricted that sea waves will normally not propagate through the inlet into the estuary. Waves that cause damage to estuary or lagoon embankments are mostly generated by local winds or by motor boats. Direct wave forces on the bridge piers only have to be considered if sea waves higher than 0.5 m can reach the bridge site. Smaller waves or wind waves generated in the estuary, which have small periods, exert forces on the bridge structure which are negligible relative to the forces due to currents.

The development of wind-generated waves depends on the velocity, duration and direction of the wind, as well as on the fetch and water depth. Design wind for the area concerned must preferably be based on local wind data. From this data the, say, once-in-10-year wind speed-duration combinations should be derived from which the significant wave height and period can be estimated using empirical relationships.

Such empirical relationships have been prepared by the Coastal Engineering Research Center (1984) to produce sets of forecasting curves. Three of these curves for shallow water conditions have been reproduced as Figure 7.5.1 for 1,5 m water depth, Figure 7.5.2 for 3,0 m water depth and Figure 7.5.3 for 4,5 m water depth. The nomogram for deep water has been reproduced as in Figure 7.6. Extensive areas with an average water depth of more than 5 m are, however, very rare in South Africa estuaries.

As an example, the wave conditions for an area with an average water depth of 3,0 m are computed, using Figure 7.5.2. For the wind fetch, a length of $F = 3\,000$ m is taken and a wind speed of $U = 20$ m/s. The wind stress factor U_A has to be computed from the wind speed, using the equation:

$$U_A = 0,71 U^{1,23} \quad (7.3)$$

where

$$U = \text{wind speed (m/s)}$$

For $U = 20$ m/s it follows that $U_A = 28,3$ (Y-axis on Figure 7.5.2). The minimum period for this wind to blow is indicated on Figure 7.5.2 as $t = 19$ min. In this case the wave height will be $H = 0,65$ m and the period $T = 2,5$ s. For a wind fetch of less than 1 000 m the generated waves will be very small.

Digital or processed wind data are not readily available in South Africa. A study has been made by the CSIR to determine extreme wind speeds at four major harbours (CSIR 1989). It appears that for the whole of South Africa's coastline the safe maximum hourly average wind speed with a 10 year return period is 25 m/s. This value could be used as a first estimate if no local data is available.

(ii) Wave forces on bridge elements

Breaking sea waves at the bridge structure could cause a threat to the bridge stability. To determine the breaking wave height at a structure, the water depth is taken at half a wave length away from the structure (the wave length of a shallow-water wave is computed from $L = T\sqrt{gd}$, where T is the wave period and d is the water depth). As a first estimate, the height of the breaking wave can be taken as 0,8 times this depth, which should be taken at high tide.

The forces, acting at about the water level, due to waves breaking perpendicular against a flat vertical surface, can be estimated conservatively with the Minikin method (CERC 1984):

$$F_w = 660 H^2 d / L \quad (7.4)$$

where

$$\begin{aligned} F_w &= \text{total pressure force per unit width (kN/m)} \\ H &= \text{wave height (m)} \end{aligned}$$

Thus, for $d = 3$ m, $H = 0,8 \times 3 = 2,4$ m and $L = 30$ m it follows that $F_w = 380$ kN/m.

Wave forces on rounded piles are much smaller.

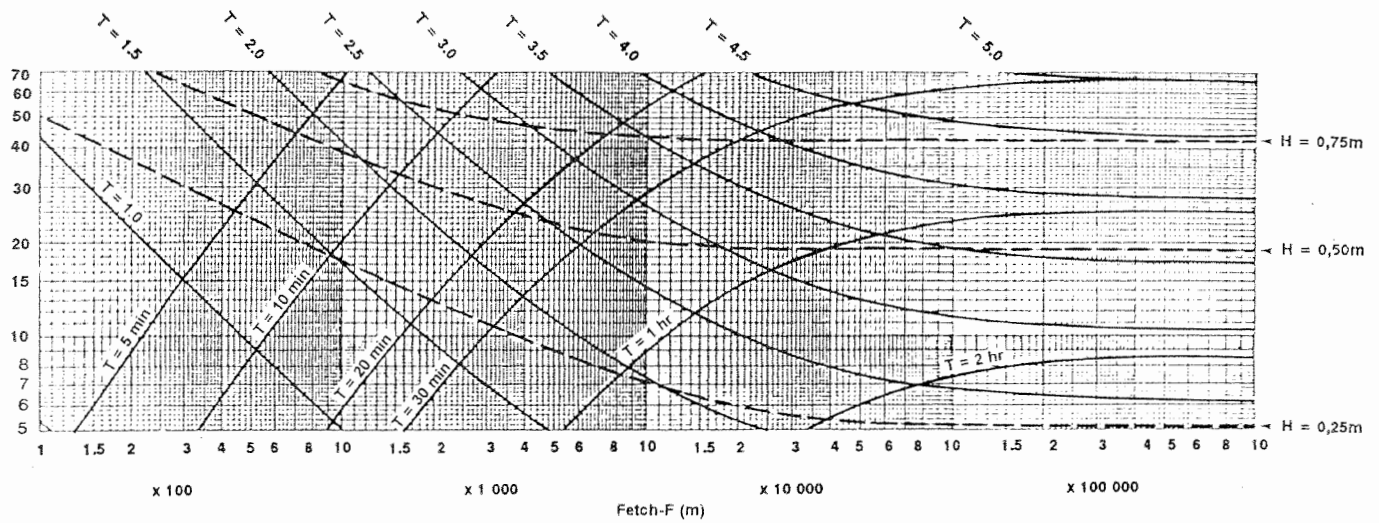


FIGURE 7.5.1 : FORECASTING CURVES FOR SHALLOW-WATER WAVES; CONSTANT DEPTHS = 1,5 m

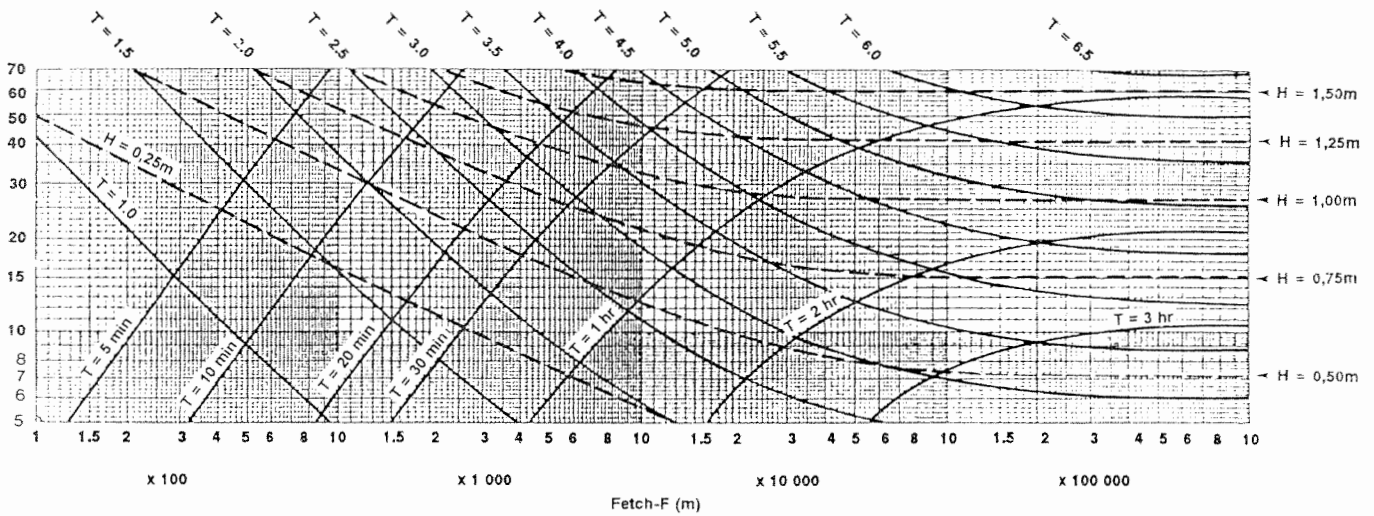


FIGURE 7.5.2 : FORECASTING CURVES FOR SHALLOW-WATER WAVES; CONSTANT DEPTHS = 3 m

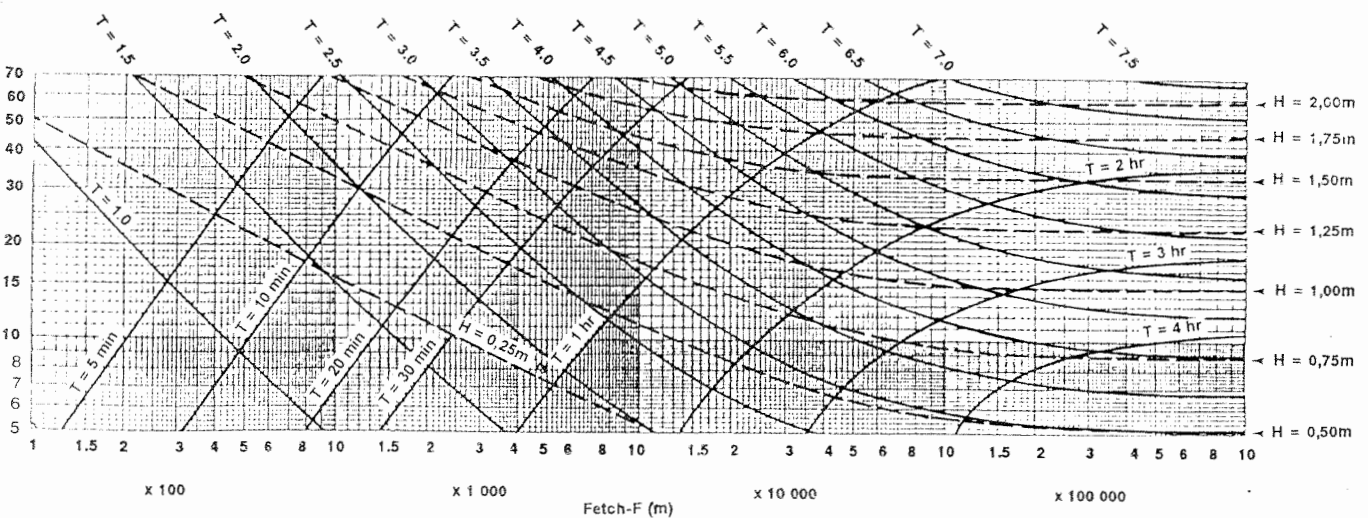


FIGURE 7.5.3 : FORECASTING CURVES FOR SHALLOW-WATER WAVES; CONSTANT DEPTHS = 4,5 m

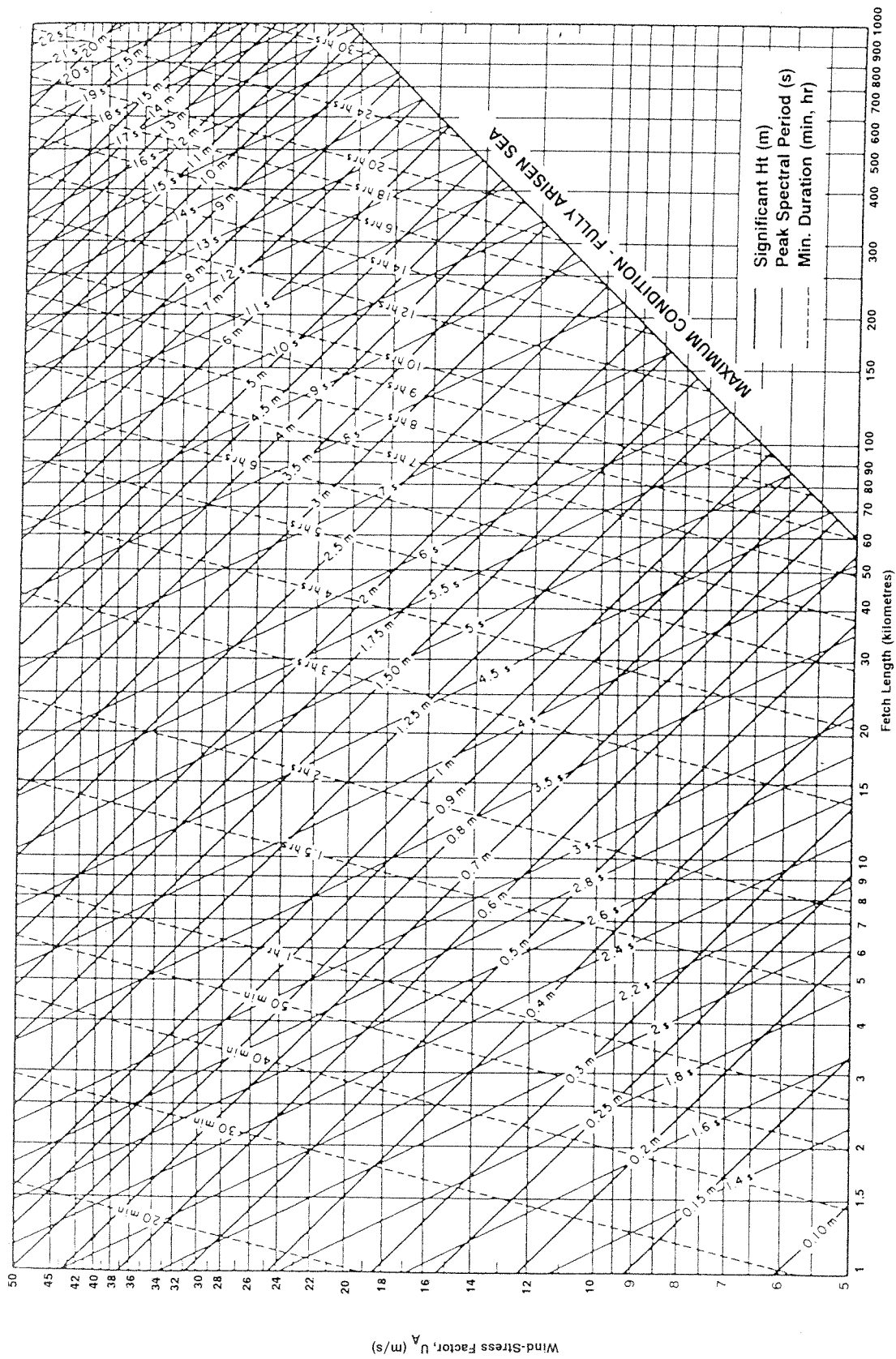


FIGURE 7.6 : NOMOGRAM OF DEEPWATER SIGNIFICANT WAVE PREDICTION CURVES AS FUNCTIONS OF WINDSPEED, FETCH LENGTH, AND WIND DURATION

(iii) Embankment armouring

Embankments should be armoured against wave action and the Hudson formula can be used, as a first estimate to determine the required weight of armouring (CERC 1984):

$$W = \frac{\rho_s g H^3}{K_D \Delta^3 \cot \alpha}$$

where

- W = minimum weight of the armouring unit (N)
 ρ_w = mass density of water (kg/m^3)
 ρ_s = mass density of rock (kg/m^3)
 Δ = $(\rho_s - \rho_w) / \rho_w$
 H = wave height (m)
 K_D = stability coefficient (values are listed in Table 7.1)
 α = embankment slope

The thickness of the cover layer can be calculated from:

$$t = nC \left(\frac{W}{\rho_s g} \right)^{1/3}$$

where

- n = number of layers of armour units, normally $n = 2$ (Table 7.1)
 C = shape factor
 = 1,0 for smooth rock
 = 1,1 for rough rock

No-damage criteria and minor overtopping							
Armour	n	Placement	Structure trunk		Structure head		
	Numbers of layers		K _D		K _D		Slope
			Breaking Wave	Nonbreaking Wave	Breaking Wave	Nonbreaking Wave	Cot θ
Quarystone							
Smooth rounded	2	Random	1.2	2.4	1.1	1.9	1.5 to 3.0
Smooth rounded	> 3	Random	1.6	3.2	1.4	2.3	
Rough angular	1	Random		2.9		2.3	
Rough angular	2	Random			1.9	3.2	1.5
			2.0	4.0	1.6	2.8	2.0
					1.3	2.3	3.0
Rough angular	> 3	Random	2.2	4.5	2.1	4.2	
Rough angular	2	Special	5.8	7.0	5.3	6.4	
Parallelepiped	2	Special	7.0 - 20.0	8.5 - 24.0	-	-	

TABLE 7.1 : SUGGESTED K_D VALUES FOR USE IN DETERMINING ARMOUR UNIT WEIGHT

The total number of armour units N for a specific area A is calculated by:

$$N = A nC \left(1 - \frac{p}{100}\right) \left(\frac{\rho_s g}{W}\right)^{2/3}$$

where

p = porosity of stone in the layer (about 0,4)

The size of the bed material underneath the armour layer must be 1/10th the size of the armouring. Because material may be washed out from below the armouring layers of the embankment as a result of the water level fluctuations, it is good practice to place a filter layer and/or filter fabric below the armour material.

The run-up of waves on embankments can be reduced by using flatter slopes or making the slope surface rough. Flatter slopes would also improve the stability of the embankment.

Because South African estuaries are relatively small, wind set-up is limited to about 0,2 m. This is insignificant compared to other water level variations so that wind set-up can normally be ignored.

7.1.4 Sedimentological aspects of estuary crossings

The main effects of constricting a tidal channel are:

- Increases in flow velocity through the constricted waterway which implies greater scour potential;
- A change in the local flow pattern; and
- Increases in the local water level gradient.

The discharge through the opening of a constricted waterway can be readily calculated in cases where the openings are nonerodible (Section 7.1.2). In the case of an erodible bed in the bridge openings, the calculations become more complicated and less reliable. A new equilibrium will occur when the cross-sectional area has scoured to such an extent that the velocities become equal to those for the initiation of motion of the local sediment (Figure 7.4). However, this is a time-related process and conditions may have changed before the new equilibrium is reached. Scour studies can be undertaken using physical or mathematical models.

The South African coastline is subject to sediment transport for most of its length. This means that tidal inlets and river estuaries are always in danger of blockage by beach sand, with the formation of a bar across the mouth. An opening can only be maintained if there is sufficient tidal flow through the inlet or if there is sufficient river discharge. Only a few river estuaries in South Africa are permanently open to the sea.

This could mean that a sand bar exists across the mouth at the beginning of a flood. When this bar completely closes the inlet the flood level will rise to just above the crest level of the bar, when the bar will overflow and erosion of a breach will start. The height of the sand spit across the mouth area is

usually about +2,5 m MSL. Because natural breaching will occur if the water level is about 0,2 m higher, a maximum water level at the mouth of +3 m MSL appears to be a safe general value.

When more detailed information is available on the crest level of the sand spit and the spit is left to breach naturally a maximum water level of this crest level plus 0,5 m can be accepted for the area close to the spit. If the sand spit is always breached artificially (usually when a specified water level is reached), this water level plus 0,5 m should be taken as the maximum water level for the area close to the spit.

The rate of lowering of the water level in the estuary basin after breaching, from which the velocities could be determined, depends on the width of the sand spit, the amount of river inflow into the basin and the size of the basin. Velocities in the breaching channel of up to 2 m/s can occur, but further inland, where the basin usually widens, these will be much lower. If the breaching occurs during a river flood, the flow velocities will be determined by the total river discharge, increased by the rate of drainage of the basin upstream of the bridge.

Shortly after breaching, the flow regime through the breaching channel will be super-critical. This means that the tidal water level variation in the sea is not propagating into the basin. Tidal water level variations and associated flow velocities will be present in the basin only when the flow regime has become sub-critical.

Where the inlet area is supplied with longshore sediment (Figure 7.1) it should be taken into account that the longshore sediment transport rate could change in future as a result of developments at the coastline adjacent to the inlet. An increase in longshore transport may result in (more frequent) closure of the inlet. A reduced rate of transport, without change in transport potential, may result in local erosion and retreat of the coastline.

The hydraulic aspects of bridges at tidal crossings can be studied effectively with the aid of hydraulic models (Section 7.4). Sedimentological aspects are more complicated, but in these cases a small-scale model can also be a useful tool in studying areas of erosion and sedimentation.

7.2 Inland basin crossings

7.2.1 Problem statement

In the case of inland basin crossings, conditions are similar to those for estuary crossings (water level variations, two-way flow through the bridge opening and wave effects), but there are some significant differences which must be taken into account in the hydraulic design of these crossings.

These differences are:

- Water level variations are caused by wind set-up/set-down, instead of tidal action,
- The flow through the bridge is the result of different wind directions (which are not periodic),
- Only locally generated waves can occur on an inland basin (no ocean waves).

The bridge opening will again be mainly determined by river flood flow, assuming that a river does discharge into the basin. If this is not the case, the flow is limited to that resulting from the water level differences between the windward and leeward sides of the bridge crossings, or from wind-generated water oscillations.

If the basin is ecologically sensitive, the design of the bridge openings may also be influenced by conservation criteria in addition to the hydraulic requirements, that is, limiting the velocities under the bridge with respect to possible scour.

7.2.2 Hydraulic design aspects

The following factors must be considered in the hydraulic design:

- The bridge opening must be able to cope with the design river flow (if applicable).
- Allowance must be made for possible scour due to the wind generated flow,
- Proper bank protection on the leeward side banks against wind generated wave attack, must be provided.
- Clearance of the superstructure must allow for river floods and water level fluctuations due to wind effects, as well as for small-craft navigation (if applicable).

Freeboard for small craft should be provided as follows:

■ Canoes and rowing boats	1 m
■ Ski boats	2 m
■ Motor boats	3 to 5 m
■ Sailing boats	8 to 10 m

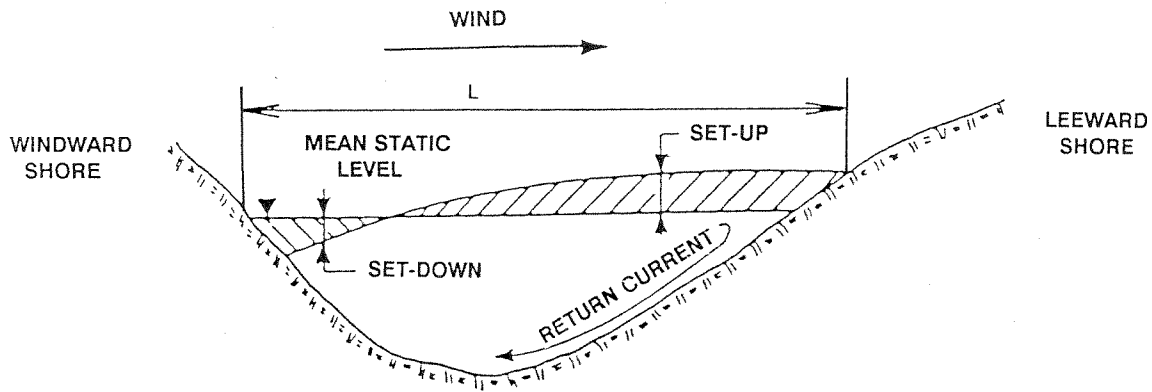
7.2.3 River flow

If a river discharges into the basin, the flow must be routed through the basin to determine the maximum discharge through the bridge with the corresponding water levels (Section 5.1).

Where records are available for water levels in the basin during floods, these can be used for the design water levels. However, if there is no or little information available, numerical analysis of the in- and outflow may be necessary to determine the design water levels in the basin as related to floods.

7.2.4 Wind effects

Wind set-up and set-down are fluctuations in water level above and below the static water level (Figure 7.7). The wind exerts a shear stress on the water surface causing the movement of the surface water, while a small return flow is generated at the bed of the basin.



NOTE: VOLUME SET-DOWN = VOLUME SET-UP

FIGURE 7.7 : WIND SET-UP AND SET-DOWN IN INLAND BASIN

The wind set-up and set-down for an enclosed area of water are determined from the design wind velocity (Section 7.1.3.(i), wind direction and fetch length by using the following formula (Bretschneider, 1967):

$$h_u = \frac{C U^2 L N}{2 g d} \quad (7.5)$$

where

- h_u = wind set-up above the static water level (m)
- U = the over-water design wind velocity (m/s)
- L = fetch length i.e. distance of water affected by wind (m)
- g = acceleration due to gravity (m/s^2)
- C = a calibration constant = $3,3 \times 10^{-6}$
- d = depth of flow for maximum static water level (m)

$$N = \frac{2 (2b_o + b)}{3 (b_o + b)} \text{ plan form factor}$$

- b_o = width of windward shore (m)
- b = width of leeward shore (m)

In the case of a large reduction in cross-sectional area by narrows, shoals or bridge crossings, a difference in water level is possible between the downwind and upwind sides of the constriction. This difference in water levels may result in significant flow velocities through the constricted area.

To determine the water levels at a constriction, the basin can be considered as two basins, separated by the constriction. The wind set-up and set-down at the constriction can be determined by applying Equation 7.5 to the basins at both sides of the constriction.

For example, for a basin as in Figure 7.8 and for a wind velocity of 30 m/s and water depth of 1,5 m the following water level differences can be determined:

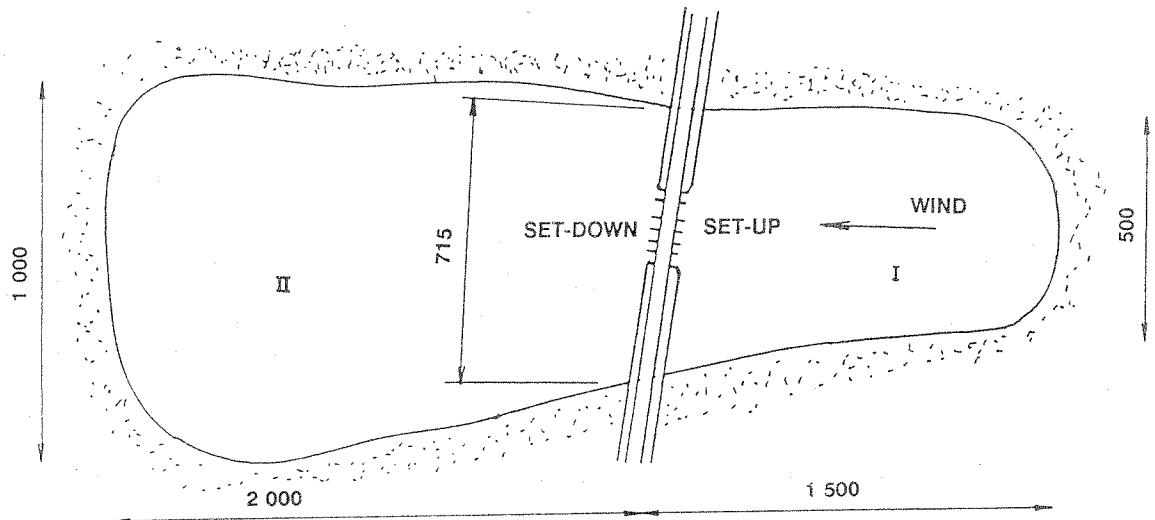


FIGURE 7.8 : EXAMPLE OF INLAND BRIDGE CROSSING

(i) Basin I

$$N = \frac{2 (2 \times 500 + 715)}{3 (500 + 715)}$$

$$= 0,94$$

Wind set-up :

$$h_u = \frac{3,3 \times 10^{-6} \times 30^2 \times 1500 \times 0,94}{2 \times 9,8 \times 1,5}$$

$$= 0,14 \text{ m}$$

(ii) Basin II

$$N = \frac{2 (2 \times 715 + 1000)}{3 (715 + 1000)}$$

$$= 0,94$$

Wind set-down :

$$h_u = \frac{3,3 \times 10^{-6} \times 30^2 \times 2000 \times 0,94}{2 \times 9,8 \times 1,5}$$

$$= 0,19 \text{ m}$$

Thus, the difference in water level at the bridge crossing, is 0,33 m.

The discharge and velocity through the constriction can be determined through methods described in Chapter 3.

It should be noted that wind set-up and set-down are time-related processes. After some time (a few hours) of constant wind conditions an equilibrium water level will be established in the basin; the effect of the causeway and bridge then becomes negligible and the basin can be considered as one. The maximum set up at the windward shore is then that calculated for the whole basin, ignoring the bridge. This comes to 0,31 m in the above case.

7.2.5 Wave action

Similarly to the estuary crossings, embankments across inland waterways could be attacked by locally generated wind waves and bank protection may have to be provided.

Most natural inland basins or vleis are shallow and the shallow-water wave predictions given in Section 7.1.3.(i) can be applied.

In the case of artificial (water storage) basins the water depth could be considerably more and in cases where the water depth is more than 5 m, the deep water wave prediction procedures can be used, as given in Figure 7.6 (CERC 1984).

The stability of the required armouring can be determined according to the procedures given in Section 7.1.3.(iii).

7.3 Hydraulic model studies

7.3.1 Application of hydraulic models

Although analytical and/or empirical techniques can be used to provide a first design of a bridge crossing, the hydraulic model still provides the most versatile design tool, particularly in the case of the more complicated situations where the flow conditions are difficult to predict (near river bends, constrictions, confluences and bifurcations) and/or where river bank erosion and bottom scour play an important role.

Such models can be and are used to determine, inter alia:

- The optimum location of the bridge crossing
- The position and extent of the general river bed change and bottom scour (contraction and local scour)

- The required bridge length
- A safe soffit level for the bridge deck
- The best pier alignment, configuration and abutment layout
- The need and extent of training works
- Special means to reduce debris collection (pier spacing)
- The hydrodynamic forces on the bridge structure
- Solutions to troublesome maintenance problems for existing bridges.

Many bridge model studies have been successfully completed and have resulted in considerable savings and increased safety (Zwamborn 1966, Zwamborn 1981 and Zwamborn and Swart 1988) while the cost of such model studies was usually only a small percentage of the construction and possible repair/replacement cost in the case of flood damage.

7.3.2 Types of models

There are two main types of hydraulic models which can be used for bridge crossing studies, namely, fixed-bed models and movable bed models. In the latter case, the model bed consists of mobile material, such as sand, crushed coal or polystyrene.

Fixed bed models are ideal to study flow distributions during low to maximum flood conditions, flow lines, velocity distributions, bridge alignment, backing-up (back water curves) and free board. Fairly large scales have to be used to enable measurement of the hydrodynamic forces in such models.

A movable bed model can be used to determine general river bed changes, optimum bridge pier alignments, river guiding works and bridge lengths, scour depths and bank erosion protection. It usually consists of a concrete base covered with a layer of model sediment in those areas of the model where changes in the topography are expected to take place. In these models, distortion and the use of lightweight model sediment, such as crushed anthracite, are essential in most cases to keep the model size within practical limits. Although it is possible to determine an approximate time scale for sediment changes from the geometric scales of the model (Zwamborn 1969), it is preferable to determine this time scale on the basis of a comparison between model results and observed prototype data.

In the case of river models, minimum scale ratios of 1:10 to 1:100 have to be used. Since this could result in very large models, a distorted model scale is often used, that is, a horizontal scale ratio of 1:100 to 1:1 000 resulting in distortions of as high as 10. The distortion should, however, be kept as small as possible.

In most cases of river crossing, a three-dimensional model will have to be built but in certain cases, such as local scour around a bridge pier or for force measurements on a bridge element, the tests can be done two-dimensionally, that is, in a suitable test flume.

7.3.3 Model design

For the design of an effective hydraulic model, the following information/data is required:

- A layout plan with contours of the area concerned far enough upstream and downstream to include topographical features which may influence flow patterns at the bridge site
- Design flow rates and related flood levels along the stretch of river covered by the model (numerical flood calculations could assist with this)
- Layout and design (foundations) of the proposed/existing bridge
- Data on river course changes, bank erosion and river bed scour (aerial photographs and borehole results at different times, preferably before and after a flood)
- Size distributions of the bed material.

The scales to which the model is built are dependent on many practical factors such as the available laboratory space, pump capacity, extent of river area to be modelled, construction time and costs. Although different types of models have different requirements, generally speaking, the scales should be as large as possible and the distortion as small as possible to reduce scale effects which could distort the model results.

A review of the basic hydraulic model scaling techniques is given in Zwamborn (1969), for both types of models, namely those with a fixed-bed and those with a movable bed.

For fixed-bed models, the horizontal and vertical scales must be chosen to satisfy the well known Froude similarity criterion :

$$\left(\frac{V}{\sqrt{gh}} \right)_r = 1$$

whereas for a sediment movement model the above scales plus the sediment scales (with regard to density and size) must also satisfy the following :

$$\left(\frac{V_*}{V_{ss}} \right)_r = 1$$

and

$$\left(C \sqrt{\frac{S}{g}} \right)_r = 1$$

where

V	=	flow velocity
g	=	acceleration due to gravity
h	=	water depth
r	=	model to prototype ration
V_*	=	shear velocity = $\sqrt{ghS}$
V_{ss}	=	settling velocity
C_i	=	grain roughness coefficient
S	=	water surface slope

Practical model scales used are, for instance,

- horizontal scale $L_r = 1:250$
- vertical scale $h_r = 1:100$
- velocity scale $V_r = (g_r h_r)^{1/2} = 1:10$
- flow rate scale $Q_r = L_r h_r^{3/2} = 250\ 000$
(this means, a 10 000 m³/s flow is reproduced by 40 l/s in the model)
- anthracite with a density of 1,35 and 170 μm mean diameter is needed to reproduce the 230 μm prototype sand (accroding to the above criteria).

7.3.4 Possible scale effects

River models are built in accordance with the Froude similarity law which ensures that inertial and gravity forces are correctly scaled. Particularly in the case of distorted models, the model roughness must be adjusted to achieve the correct water-level slopes in the model (this often requires artificial roughness to be added).

Although the viscosity forces in such a model are not correctly scaled, provided the model flow is turbulent (Reynolds number greater than 600), the model will be representative of prototype flow conditions. The larger the scale of the model, the less chance there is for significant deviations, or scale effects.

Significant scale effects can, however, occur in movable bed models, if not designed by experts in the correct way. This is clearly demonstrated by the results of model tests with a movable bed, of the Notchwan river bridge (Zwamborn 1966). Two models were built, one at scales $L_r = 1:120$ and $h_r = 1:36$ (small model) and the other, large model at $L_r = 1:24$ and $h_r = 1:16$, the distortions being 3,3 and 1,5 respectively. Both models were tested with 280 μm sand which is in accordance with the scaling laws given in Section 7.3.3. The resulting maximum-river bed scour was identical for the two models (Figure 7.9), confirming the correctness of the model scaling technique. However, when the small scale model was tested with incorrectly scaled model sediment (400 μm anthracite instead of 620 μm) the scour found was significantly larger (Figure 7.9). Also, when the large model was remoulded according to the regime theory (Blench 1957), with $L_r = 32$, $h_r = 10$ and using 780 μm sand (prototype size) the scour was far to small (Figure 7.9).

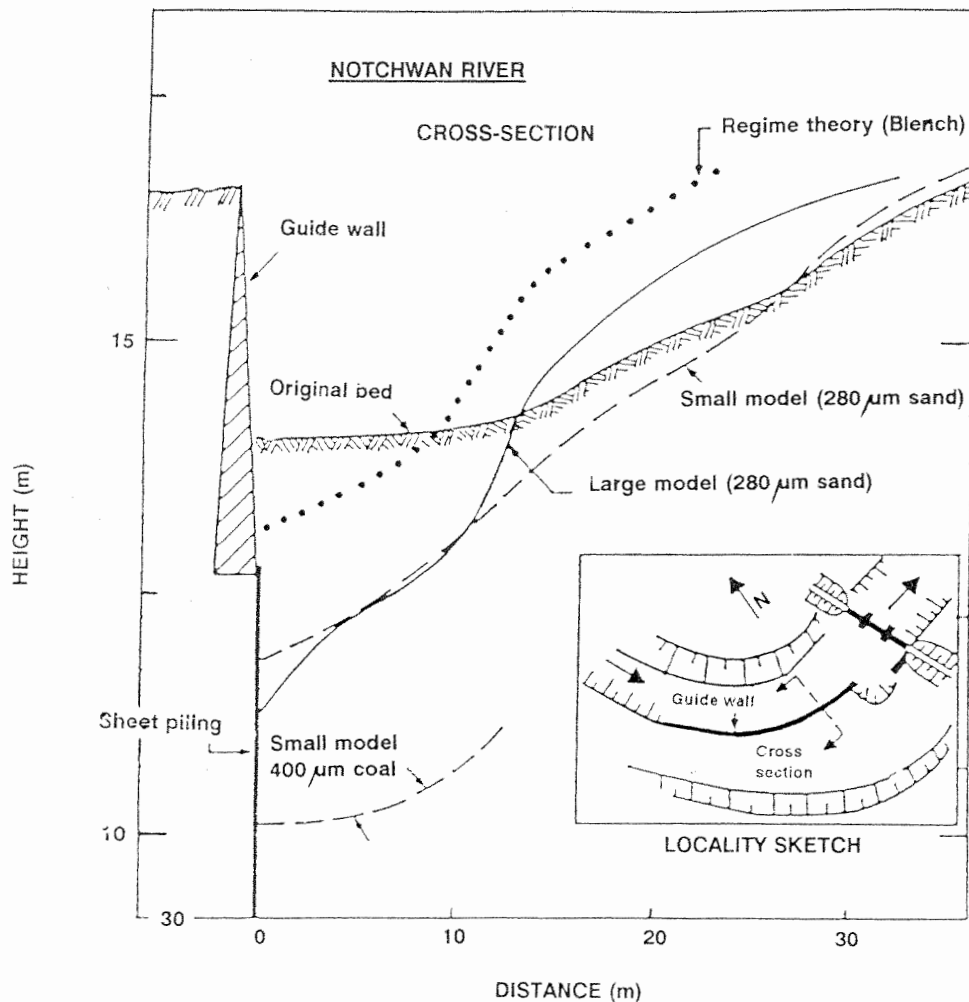


FIGURE 7.9 : SCOUR DEPTHS NOTCHWAN RIVER

Although these results give confidence in the reliability of the scale laws, especially in the case of movable bed models, it is very useful if the model can be calibrated with known prototype conditions, in particular to determine the sediment time scale experimentally.

7.3.5 Model study results

There are many examples of successful model studies but only one example will be described in some detail (Zwamborn and Swart 1988).

Umfolozzi bridge model studies

A movable bed hydraulic model was built for the following reasons :

- To determine the best river crossing with regard to scour
- To determine the minimum bridge length, and

- To determine the optimum bridge pier-spacing and orientation

The model was designed according to the similitude criteria for movable bed models described in Zwamborh (1969). It had a horizontal scale of 250, a vertical scale of 100. Crushed anthracite with a specific density of 1,35 and a mean grain size of 170 micron to represent the 230 micron prototype sand was used as sediment. The 100 year design flood was 5 600 m³/s but the model was also tested with 50 per cent (8 500 m³/s) and 100 per cent (11 300 m³/s) larger floods.

During the early stages of construction, in July 1963, a major flood occurred, estimated at (8 500 m³/s) and exceeding the design flood by 50 per cent. The scour depth determined from borings after the flood, provided proof of the reliability of the model scour predictions (Figure 7.10).

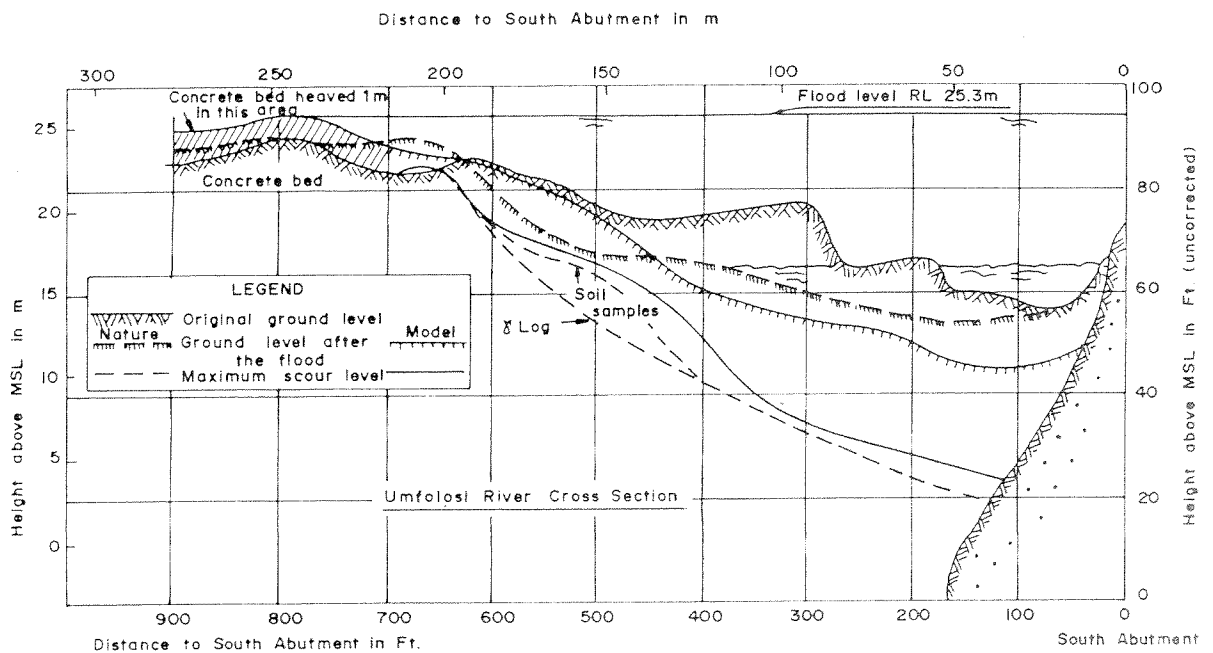


FIGURE 7.10 : SCOUR DEPTHS AT THE UMFOLOZI RIVER BRIDGE

8. ECOLOGICAL CONSIDERATIONS

8.1 Introduction

Bridges can have a significant impact on the environment if they impinge significantly on the habitats of aquatic species. Areas where special care needs to be taken are estuaries and wetlands. The following comments relate especially to the impacts which bridges might have on estuaries.

8.2 Estuarine ecological considerations

An estuary is a unique and important part of the aquatic environment and is essentially a body of water where a river mixes with and measurably dilutes sea water. It therefore represents a transition zone, retaining some characteristics of both fresh water and the marine environment, but it also has unique properties of its own.

Characteristically, abiotic conditions are highly variable within an estuary, particularly fluctuations in salinity. It is not unusual for salinity changes to be sudden and dramatic. Such environmental stresses have resulted in specialized adaptations by the flora and fauna. Relatively few plant and animal species have adapted to the conditions imposed, but those that have adapted attain high levels of abundance and biomass.

Nutrients are typically rich in estuarine waters. These are utilized by the plants, and following their death, are recycled through decomposition. This recycling of nutrients coupled to high plant production and associated decomposition processes result in high detritus (biogenic material in various stages of decomposition) levels in estuaries which in turn, support an abundant animal community. Estuaries are known to be amongst the richest ecosystems (an organic community of plants and animals viewed within a physical environment) on earth and have been ranked with intensive agriculture, coral reefs, kelp beds and areas of strong upwelling. By comparison, much of the open sea is 10 - 20 times less productive (Branch & Branch 1981). It is not the attached plants themselves which directly nourish herbivores, but the decomposing fraction together with the associated microbial community involved in that process which provide the rich source of nutrition.

Other sources of nutrition include phytoplankton (floating microalgae) and benthic (bottom living) microalgae. Large amounts of organic material, primarily plant detritus, are also derived from the sea or rivers. Collectively, plants provide a rich source of food in estuaries and this material sustains a prolific animal community, primarily invertebrates (animals without backbones - for example, shrimps, prawns and zooplankton). Over 80% of these invertebrates feed on detritus, and most of this is of plant origin.

Water circulation and mixing is of crucial importance in maintaining estuarine viability through transportation and redistribution of material. Constructions or obstructions which alter water quality or reduce water movement may negatively impact on these systems. Included are floodplains and channel areas. For example, bridge and causeway construction across the Swartvlei estuary complex reduced tidal flow into the lake by 40% and the resultant reduction in salinity permitted another submerged plant (*Potamogeton pectinatus*), to encroach above the construction (Howard-Williams & Liptrot 1980). This species is less tolerant to fluctuations in salinity and reflects an encroachment of low salinity conditions

in a region which was previously more saline. Hence former estuarine characteristics above the causeway were permanently altered.

In Transkei and Natal (and further northwards), mangroves dominate the estuarine plant community. These are the only trees adapted to living in salty tidal waters and are extremely vulnerable to changes in water level. Aerial roots (pneumatophores) ensure an adequate supply of air and if inundated for too long, mass mortality of the mangal occurs. An increase in water level may be due to natural causes (mouth closure) or through anthropogenic structures, or a combination of the two factors. Elevated water levels as a result of bridge construction across the estuary at Sodwana Bay was probably the main reason for the disappearance of a formerly rich mangrove system in the estuary (Bruton & Appleton 1975).

Construction of bridge abutments across estuarine tidal flats can also have a detrimental effect on the biotic community.

The saltmarsh represents a specialized plant assemblage which is specifically structured across the intertidal gradient. At the highest level of the tide, plant species are adapted to relatively long periods of aerial exposure and short periods of tidal inundation. The relationship between these two parameters changes progressively towards the low tide level where plants are exposed for short periods only. Subtidal plants are adapted to permanent inundation and are consequently species quite different to those higher up the intertidal gradient. Predictably, there is an associated change in animal species linked to plant zonation.

Substrate characteristics are also important in determining the community structure along the intertidal gradient. The presence of a particular plant species may influence sediment dynamics, particle size and chemistry, all of which contribute to the ecology of the saltmarsh and impact specific characteristics to the biotic community.

The information presented above, serves to illustrate the complex and unique nature of plants and animals inhabiting estuaries. In the case of bridge abutments, it is not only the presence, but the orientation of the abutment which may be problematic. Inter alia:

- Transportation and redistribution of nutrients and detritus may be restricted through changes in water velocity and flow patterns in areas influenced by the construction.
- Areas of the saltmarsh adjacent to the abutment may experience elevations in temperature which could lead to changes in chemical characteristics of the sediment as well as subject biota to additional physiologically stressful situations.
- At times of fluvial flooding, restrictions and alterations to the seaward flow of silt-laden water may result in the deposition of fine sediment across areas of the saltmarsh. Besides the smothering effect, there is likely to be a progressive change in the intertidal gradient as a result of localized sediment accumulation over time. Biotic zonation will change in direct response to the changing gradient. The saltmarsh community at the top of the intertidal zone may ultimately be replaced by terrestrial vegetation as a result of tidal waters no longer reaching former localities. A similar effect will occur if the tidal range is reduced as a result of hydraulic restrictions in the main channel.

- Biotic productivity is also variable along the intertidal gradient. A 10% reduction of a particular plant species may ultimately have far greater consequences for overall saltmarsh productivity.

The nett effect of saltmarsh destruction will ultimately impact negatively on higher trophic levels. There has, for example, been considerable concern expressed in the northern hemisphere with respect to the future of migratory shorebird populations. These birds breed in their northern summer grounds, but numbers appear to be on the decline as a result of loss and degradation of intertidal feeding habitats in southern latitudes where they spend their non-breeding season. Habitat loss in estuarine areas of southern Africa (and elsewhere) will result in an increase in bird density and there will come a point at which the rate of prey intake (due to increased competition for food) will decline to seriously low levels and ultimately, increased mortality rates and fewer birds returning northwards (Goss-Custard 1989).

Estuaries are specialized habitats, and although damage to any part may sometimes be considered minimal, the collective impact of anthropogenic changes (reduced riverine inflow, pollution, sedimentation, changes to water-flow patterns etc) is having a serious and negative effect on our estuarine ecosystems.

In conclusion :

- Estuaries are highly productive systems, sustaining a rich plant and animal community.
- High levels of production are usually associated with species unique to estuaries.
- Effective dispersion of material (e.g., nutrients, suspended food particles) is directly dependent on efficient water circulation.
- Reduced water circulation as a result of, Inter alia. man-made features, is likely to impact negatively on total estuarine productivity.
- A reduction in saltmarsh area (due to: direct smothering, increase in the intertidal gradient, etc.) will reduce the contribution made by saltmarshes to estuarine productivity. In extreme cases, species unique to estuaries may dissappear as a result of habitat loss.

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