Guide to Bridge Technology Part 8

Hydraulic Design of Waterway Structures

Austroads

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Abstract

Austroads Guide to Bridge Technology provides bridge owners and agencies with advice on bridge ownership, design procurement, vehicle and pedestrian accessibility, and bridge maintenance and management practices. The Guide has eight parts.

Part 8 provides a guideline for issues related to the waterway design of bridges. It covers various topics such as design flood standards and estimation methods, general considerations in waterway design and design considerations of waterway structures. Design of new bridges for scour, scour countermeasures, as well as monitoring and evaluation of scour at existing bridge sites are also included.

Keywords

Waterway design, total waterway, flood frequency analysis, bridge hydraulics, open channel flow, hydraulic design, scour, stream stability, unsteady flow analysis, hydraulic forces on structures, bridge deck drainage, design storm, drainage of carriageway, bridge scour, contraction scour, local scour, scour countermeasure, scour monitoring, live-bed scour, clear-water scour, existing bridges, scour aggradation, scour degradation, types of scour, bridge crossing, estimation of design floods

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First edition published 1994

The latest edition of this Guide reinstates the superseded Austroads' 1994 *Waterway Design: A Guide to the Hydraulic Design of Bridges, Culverts and Floodways*. This version provides updated details and information based on the latest technology and incorporates the US Federal Highway Administration publications that are currently being used in Australian and New Zealand practices.

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About Austroads

Austroads is the peak organisation of Australasian road transport and traffic agencies.

Austroads' purpose is to support our member organisations to deliver an improved Australasian road transport network. To succeed in this task, we undertake leading-edge road and transport research which underpins our input to policy development and published guidance on the design, construction and management of the road network and its associated infrastructure.

Austroads provides a collective approach that delivers value for money, encourages shared knowledge and drives consistency for road users.

Austroads is governed by a Board consisting of senior executive representatives from each of its eleven member organisations:

- Roads and Maritime Services New South Wales
- Roads Corporation Victoria
- Queensland Department of Transport and Main Roads
- Main Roads Western Australia
- Department of Planning, Transport and Infrastructure South Australia
- Department of State Growth Tasmania
- Department of Infrastructure, Planning and Logistics Northern Territory
- Transport Canberra and City Services Directorate, Australian Capital Territory
- Australian Government Department of Infrastructure and Regional Development
- Australian Local Government Association
- New Zealand Transport Agency.

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1. Introduction

1.1 Scope

This publication provides guidance on the selection of the design floods required for the various aspects of the design of waterway structures, the hydraulic design of bridges, and the design of works required to protect these structures from the effects of scour. Readers are referred to the *Austroads Guide to Road Design Part 5B: Drainage: Open Channels, Culverts and Floodways* (Austroads 2013a) for guidelines for the design of culverts and floodways.

The hydraulic design of waterway structures (bridges, culverts and floodways) is of considerable economic importance, as these structures can consume up to 30% of the total road construction costs. The selection of the appropriate design flood and good practice in the design of these structures determines the initial cost, ongoing maintenance costs, the provision of the desired level of serviceability to traffic, and the safety of the road user.

Washouts at bridge approaches, in roads adjacent to culverts and at floodways can create hazards and delays to the road user. Major damage or failure of a bridge as the result of scour not only poses a safety hazard to motorists, but also causes large social impacts and economic losses over a long period of time. The appropriate design of these structures should minimise these impacts and reduce the cost of flood damage.

This publication provides guidance on the following:

- The average recurrence intervals of the design floods which should be utilised for the design of the various aspects of a stream crossing.
- The methods available to a design engineer for estimating design flood discharges. For details of the actual procedures, reference is made to *Australian Rainfall and Runoff: A Guide to Flood Estimation* (Ball et al. 2016).
- The basic factors which should be considered when designing a stream crossing, including geomorphic and hydraulic factors, environmental considerations and the factors to be considered in the selection and design of waterway structures.
- The hydraulic design of bridges.
- The estimation of scour at bridges and designing for the effects of scour.
- The design of works for the protection of bridges, culverts and floodways from the effects of scour.

Design procedures and worked examples are provided where appropriate.

Advanced methods for hydraulic design of structures are also introduced, including:

- the development of computer programs that can be used for the hydraulic design of bridges, culverts and floodways
- more sophisticated methods, such as two/three-dimensional hydraulic analysis, computational fluid dynamics and many more advanced hydraulics modelling
- approaches to solving more complicated problems, such as unsteady flow analysis.

Refer to Section ‘Glossary’ at the end of this Guide for the definitions used throughout the document.

1.2 Guide Structure

The *Guide to Bridge Technology* is published in eight parts and addresses a range of bridge technology issues, each of which is summarised in Table 1.1.

Table 1.1: Parts of the Guide to Bridge Technology

Part	Title	Content
Part 1	Introduction and Bridge Performance	• Scope of the *Guide to Bridge Technology* and its relationship to the bridge design standards. • Factors affecting bridge performance and technical and non-technical design influences. • Evolution of bridges, bridge construction methods and equipment and bridge loadings. • Specifications and quality assurance in bridge construction.
Part 2	Materials	• The full range of bridge building materials including concrete, steel, timber and non-metallic components. • Material characteristics including individual stress mechanisms.
Part 3	Typical Bridge Superstructures, Substructures and Components	• Superstructure and substructure components – namely timber, steel, wrought iron, reinforced and pre-stressed concrete. • Typical bridge types such as suspension, cable stayed and arched types. • Bridge foundations.
Part 4	Design Procurement and Concept Design	• Bridge design process procurement models, specification requirements, design and delivery management processes, design checking and review concepts, the use of standardised components, aesthetics/architectural requirements, standard presentation of drawings and reports, designing for constructability and maintenance. • Service life of the structure and components, mining and subsidence, flood plains, bridge loadings, and geotechnical and environmental considerations.
Part 5	Structural Drafting	• Detailed drawing aspects required to clearly convey to the consultant/construction contractor the specifics of the project. • Standards including details required for cost estimating and material quantities. • Reinforcement identification details.
Part 6	Bridge Construction	• Guidance to the bridge owner's representative on site. • Focuses on bridge technology, high-risk construction processes e.g. piling, pre-stressing, and the relevant technical surveillance requirements during the construction phase. • Bridge geometry, the management of existing road traffic and temporary works.
Part 7	Maintenance and Management of Existing Bridges	• Maintenance issues for timber, reinforced and pre-stressed concrete, steel, wrought and cast iron bridges. • Maintenance of bridge components including bridge bearings and deck joints. • Monitoring, inspection and management of bridge conditions.
Part 8	Hydraulic Design of Waterway Structures	• Waterway design of bridge structures • Design flood standards and estimation methods, general considerations in waterway design and design considerations of waterway structures. • Design of new bridges for scour, as well as monitoring and evaluation of scour at existing bridge sites.

2. Design Floods

The probability terms used in this Guide are the same as those adopted in the *Australian Rainfall and Runoff: A Guide to Flood Estimation* (Ball et al. 2016). They are 'average recurrence interval' and 'annual exceedance probability'. The definitions of these two terms are:

- **Average recurrence interval (ARI)** – the average time period between occurrences equalling or exceeding a given value.
- **Annual exceedance probability (AEP)** – the probability of an event being equalled or exceeded within a year.

The word 'average' is the key part of the definition of recurrence interval. This is because hydrological events are generally random occurrences and it cannot be inferred that a flood of a particular ARI will be exceeded at regular intervals. It is important that design engineers understand that the periods between exceedances are generally random, and that they convey and explain this to those who make decisions on the basis of their investigations and designs, and to members of the public who are affected by them.

ARI is usually expressed as the reciprocal of the AEP. For example, the probability that the 100 year ARI peak flow will be equalled or exceeded in any one year is 0.01 or 1%. The use of ARI and AEP was considered to be interchangeable. As this interchangeable use often resulted in confusion, the National Committee on Water Engineering has adopted the terms shown in Figure 2.1 and the suggested frequency indicators (Ball et al. 2016).

Figure 2.1: Australian Rainfall and Runoff preferred terminology

Frequency descriptor	EY	AEP (%)	AEP (1 in x)	ARI
Very frequent	12			
	6	99.75	1.002	0.17
	4	98.17	1.02	0.25
	3	95.02	1.05	0.33
	2	86.47	1.16	0.5
	1	63.21	1.58	1
Frequent	0.69	50	2	1.44
	0.5	39.35	2.54	2
	0.22	20	5	4.48
	0.2	18.13	5.52	5
	0.11	10	10	9.49
Rare	0.05	5	20	20
	0.02	2	50	50
	0.01	1	100	100
Very rare	0.005	0.5	200	200
	0.002	0.2	500	500
	0.001	0.1	1000	1000
	0.0005	0.05	2000	2000
Extreme rare	0.0002	0.02	5000	5000
			↓	
			PMP/ PMPDF	

Source: Ball et al. (2016).

2.1 Design Flood Standards

2.1.1 General

In designing stream crossings and associated waterway structures, there are several aspects of the design that may require the use of design floods with different average recurrence intervals. These various aspects of design are as follows:

- overall design of the total waterway of a stream crossing, including protection works to bridge abutments, culvert inlets and outlets, and floodways
- level of service to be provided to traffic
- serviceability limit states (SLS) for the bridge structure
- ultimate limit states (ULS) for structural strength and stability of the bridge structure
- environmental impact of the waterway structure on the stream and its environs.

AS 5100.1 specifies the ARI for flood immunity and SLSs. However, owing to the wide variation in conditions throughout Australia and the different standards adopted by the various road agencies, it is at the discretion of the relevant agency to select an appropriate ARI for a specific design.

It should be noted that the responsibility for urban watercourses/drains varies around Australia. In some cases, statutory authorities are responsible while, in others, it is the responsibility of local government. Checks should be made with the authority responsible for a particular urban watercourse/drain to see if they have any requirements for road crossings, such as the design standard of waterway openings and/or design flows.

2.1.2 Overall Design of Total Waterway

It is generally the practice in the design of a stream crossing that the total waterway is designed to pass floods with AEPs up to 2% or 1% (ARIs up to 50 or 100 years) without significant damage to the road and waterway structure(s), although the level of serviceability to traffic may provide for the crossing to be impassable with a lower ARI flood. For example, a bridge or culvert may be designed to pass a 20 year ARI (4.9% AEP) flood without interruption to traffic, but for larger floods the road will be impassable.

For clarity, the flood utilised for the overall design of the total waterway will be referred to as ‘the total waterway design flood’ in the remainder of this Guide.

The protection works to the fill around bridge abutments, to bridge approaches, and at culverts and floodways should be designed so that only minimal damage occurs with floods with ARIs up to and including that of the total waterway design flood.

Design engineers should also consider the effects on major stream crossings of extreme flood events that are larger than the total waterway design flood, and determine the resulting mode of failure as indicated in Section 3.5.1.

2.1.3 Serviceability Level

The level of serviceability to be provided to traffic at a stream crossing will be determined by a risk-based approach where the total road system is of main interest. The probability of closure of the total road system will be much greater than that at an individual site and is difficult to determine. In addition, the closure of a railway or major road due to flooding of any one of many stream crossings will cause closure of the route. As a result, it is normal practice to design each stream crossing on a road link for some predetermined level of serviceability (Ball et al. 2016).

It is worth noting, however, that techniques and data are now available to assess link flood performance in Australia. Queensland Department of Transport and Main Roads (TMR) has developed a methodology to determine probability of closure of a total road link – this is based on continuous simulation techniques of a long period of historical flows based on gridded rainfall data. This allows link upgrade strategies to be developed on a consistent link performance improvement/value for money basis. This approach has already been undertaken on the Flinders and Gore Highways, and are currently used to assess the whole of the Bruce Highway (from Brisbane to Cairns)1.

The selection of the level of serviceability to be provided at each waterway structure (as distinct from the stream crossing) on a road link is generally based on the following criteria:

- the level of serviceability expected by the community
- the availability of alternative routes and period of closure
- the importance of the road as access in emergency situations, such as to hospitals, airports, etc.
- the relationship between traffic density and composition, and the wet season, especially in northern Australia
- economic considerations, i.e. the cost and benefit of providing a higher level of serviceability.

In addition, the requirements of local authorities, the Environment Protection Agency (EPA) and those responsible for navigation and flood control will also influence the size and type of waterway structures and, hence, impact on the level of service provided.

Where it is likely that a higher level of serviceability will be required on a road in the future, consideration should be given to staging the construction of the waterway structures at stream crossings. This can be achieved by designing the initial stage so that it can be upgraded without major structural changes.

Australia is a large continent with a wide range of climates, topography and density of population. Hence, the levels of serviceability provided on roads in Australia vary considerably from jurisdiction to jurisdiction and even within jurisdictions. Design engineers should, therefore, consult the relevant road agency to determine the level of serviceability required, prior to commencing the design of a road and any associated waterway structures.

There are two interrelated aspects to be considered when determining the level of serviceability:

- the frequency with which the road is closed to traffic
- the time of closure.

It should be noted that for a particular class of road, frequent closures of short duration may be acceptable, whereas, long duration closures of the same frequency may not be. Conversely, there are situations where long duration, very infrequent closures may not cause problems.

AS 5100.1 specifies the following levels of flood immunity and SLSs, subject to approval by the relevant authority (Table 2.1).

1 Comments from Chris Russell (TMR) as part of the Austroads Bridge Task Force (BTF) review (2017).

Table 2.1: AS 5100 flood immunity and SLSs

Austroads road classification	Flood immunity (1)	SLS
Controlled access highways Includes: motorways and freeways (National/State/Territory)	100 years ARI (1% AEP)	100 years ARI (1% AEP)
Arterial roads classes 1 and 2(2) Includes: highways and urban arterial roads (National/State/Territory)	50–100 years ARI (2%–1% AEP)	50–100 years ARI (2%–1% AEP)
Arterial road class 3(2) Includes: main roads (State/Territory)	50 years ARI (2% AEP)	50 years ARI (2% AEP)
Local roads classes 4 and 5(2)	10–20 years ARI (9.5–4.9% AEP)	20 years ARI (4.9% AEP)
Urban collector/distributor roads	10–50 years ARI (9.5–2% AEP)	20–50 years ARI (4.9–2% AEP)
Urban local roads	10 years ARI (9.5% AEP)	10 years ARI (9.5% AEP)

1 *Subject to approval by relevant authority.*

2 *For description of road classes, refer to Austroads (2015).*

Source: Based on AS 5100.1.

Risk of design flood being exceeded

Consideration of the risk of the design flood being exceeded during the design life of a structure can be used as an aid in the selection of the level of serviceability to be provided at a stream crossing. The probability of a flood flow of a particular ARI being equalled or exceeded within a specific period increases as the duration of the period increases. This relationship can be expressed as shown in Equation 1:

$$P = 1 - exp\left[\frac{-L}{Y}\right] \qquad 1$$

where

P = probability of one or more exceedances of the design capacity during the design life, L of a structure whose design flood ARI is Y

Examples of the risk of exceedance of a particular flood for a structure with a design life of 100 years are given on Table 2.2.

Table 2.2: Probability of design flood being exceeded during design life of 100 years

Design flood ARI (years)	AEP of design flood (%)	Probability of design flood being exceeded in 100 years (%)
5	18.10	100.0
10	9.50	100.0
20	4.90	99.3
50	2.00	86.5
100	1.00	63.2
2000	0.05	5.0

2.1.4 Serviceability Limit State

For the SLS, AS 5100.1 requires that bridges be designed for the effects of a flood that has an average recurrence interval as defined in Table 2.1, unless specified otherwise by the relevant authority. The probability of the design flood being exceed each year and during the design life of 100 years for corresponding design flood ARI is presented in Table 2.2. A structure is required to remain open (without damage) under various combinations of serviceability loads. The most relevant load combinations in this case are traffic loads, hydrodynamic flood forces, debris loads and log impact.

Another important serviceability measure is the provision of freeboard at bridges and culverts for debris clearance and avoiding pressure flow. Freeboard is an allowance made between the design water level and a specified point, e.g. bridge soffit, culvert obvert, etc. Freeboards for culverts and bridges should be as follows (Austroads 2013b):

- For culverts and bridges with a cross-section greater than 6 m^2, a freeboard of 0.3 m should be provided in order to prevent blockages of, or damage to, the culvert or bridge by debris.
- Culverts with a cross-section less than 6 m^2 are usually designed with submerged inlets that is without freeboard, unless it is known that debris will be carried down the catchment.

It should be noted that some road agencies do not generally mandate the freeboard allowance to the soffit of the bridge deck. Instead, the serviceability of the bridge is determined by the flood immunity of the bridge deck (i.e. overtopping flood)2. Refer to the relevant road agency for the specific requirements.

2.1.5 Ultimate Limit State

For the ultimate failure limit state, AS 5100.1 requires that a bridge shall be designed for the effects of a flood of a magnitude up to and including that with a 2000 year average return interval, whichever produces the most severe effect. Assuming a 100 year design life, then this equates to a flood with a 5% chance of being exceeded during the design life of the structure.

The bridge design code requires that bridges shall not fail catastrophically as a result of structural inadequacies and/or the loss of static equilibrium under the ultimate loads. Hydraulic considerations in the design of bridges are discussed in Section 4.2.

In most cases, it is not necessary to estimate the 2000 year ARI (0.05% AEP) flood; it is sufficient to determine, if it is likely that the bridge will or will not be overtopped.

If the bridge is overtopped, then the critical flood for the crossing is likely to occur when the flood is just about to overtop the bridge deck, assuming that there is no relief flow available on the road approaches. The photos in Figure 2.2 show the critical flood at Bowmans Crossing over Upper Hunter River (NSW), where the bridge and the road approach are at same level. The crossing was designed for 1 in 20 year ARI (4.9% AEP), in which all flood flow up to the bridge deck has to go under the bridge. The 2007 flood was just above the design flood. Significant scour on substructure occurred.

2 Comments from Chris Russell (TMR) as part of the BTF review (2017).

Figure 2.2: Scour due to an overtopped flood at Bowmans Crossing over Upper Hunter River (NSW) in 2007

Source: Roads and Maritime Services (Roads and Maritime) NSW (n.d.).

To avoid the large amount of work required to estimate the 2000 year ARI (0.05% AEP) flood, the 500 year ARI (0.2% AEP) flood can be estimated using the simple extrapolation procedure given in Section 2.2.6. If the bridge is overtopped with a flood with an ARI less than 500 years (or an AEP greater than 0.2%), then the bridge should be designed for this overtopping flood. Only when a bridge is not overtopped, with a flood less than or equal to the 500 year ARI (0.2% AEP) flood event, will an estimate of the 2000 year ARI (0.05% AEP) flood be required to determine the ULS design flood loading condition.

It should be noted that, with the release of the 2016 *Australian Rainfall and Runoff* and rainfall Intensity Frequency Duration (IFD) data, 2000 year ARI (0.05% AEP) IFD and temporal pattern are readily available and the 0.05% AEP design flood can be estimated directly.

For bridges in the north of Australia, in regions subject to tropical cyclones, where design flood estimates could be in error, as a result of poor data or a lack of data, it is recommended that bridges should be designed for overtopping, unless the design engineer is confident overtopping cannot occur.

The effect of debris, reducing the waterway opening and causing greater depths of scour, should be considered when evaluating the foundation design of a bridge in accordance with Section 4.7.

2.1.6 Environmental Impact

There is no published information available on the ARI of the flood that should be used for assessing and minimising possible environmental damage to a stream from the construction of a road crossing. Each site should be investigated for possible problems that might occur with a range of flood events, with emphasis on the more frequent events. The factors identified in Section 3.3 should be considered when assessing any potential environmental damage. These factors, which are not only applicable to bridges but any waterway structure, include:

- selection of a suitable site
- provision of an adequate waterway opening to limit backwater effects and excessive localised bed scour
- protection of banks from erosion resulting from the redirection of flow and turbulence, or from excessive increase in velocity
- protection of natural vegetation, especially where it protects or stabilises natural banks
- control of roadside drainage, where it enters the stream, to limit bank erosion
- provision of adequate waterway openings to maintain a natural supply of flood water to wetland areas

- provision of an adequate number of waterway openings, in wide flood areas in arid regions, to ensure that water is not prevented from reaching areas downstream from the road, which could lead to the death of vegetation.

Environmental impacts due to bridge work may include the instability of the riverbank, erosion of streambed, increase flooding upstream and changes in flow distribution. These impacts may be caused by a significant constriction of waterway, the abutment location too close to the riverbank, or the presence of a bridge substructure located at the transition zone (Figure 2.3).

Figure 2.3: Luskintyre Bridge over Hunter River (NSW) with a pier located too close to the river bank

Source: Roads and Maritime (n.d.).

2.2 Estimation of Design Floods

Peak discharge is the most commonly required input for river hydraulic models. For unsteady flow models the flow hydrograph is usually required. Statistical methods, regional regression equations and hydrologic modelling are of primary interest. Any one of these methods may provide the most reliable estimate of peak discharge based on basin conditions and availability of information.

The sources of peak discharge in general order of preference are:

- prior studies by authoritative sources
- statistical frequency analysis of gauge records at the site
- transferring a gauge analysis to a nearby, ungauged location
- applicable regional regression equations
- hydrologic models.

This section briefly describes the methods of flood estimation available to the design engineer for estimating design flood discharges and ULS floods.

The actual procedures used for design flood estimation should be in accordance with the recommendations of Book 3, Peak Flow Estimation of *Australian Rainfall and Runoff* (Ball et al. 2016). The Hydraulic Design Series (HDS) number 2 manual (McCuen, Johnson & Ragan 2002) also provides discussion and guidance on the analysis of aspects of the hydrologic cycle that are important to highway engineers.

Information on past floods is extremely valuable in the assessment of theoretically based design flood estimates. Design engineers should make every effort, therefore, to obtain any historical flood data that might be available. Valuable information is often available from landowners, Railway Authorities, Water and River Authorities, Local Government Authorities, and from published historical records, such as newspapers.

Flood flows can be estimated from flood levels using the techniques outlined in the subsequent sections. Where there is a combination of waterway structures operating in parallel, the flows through or over each individual structure are summed to give the total flood flow.

2.2.1 Methods Available

Rural catchments

Flood estimation procedures for rural catchments can be divided into two broad groups: those used for gauged catchments, and those used for ungauged catchments. The following methods are generally used to estimate design floods from gauged catchments:

- Flood frequency analysis – for catchments with long streamflow records: the recorded floods are statistically analysed to estimate design floods of a selected probability of exceedance.
- Unit hydrograph methods – for catchments with limited streamflow records: the recorded floods and associated rainfall are used to construct a unit hydrograph. Design storms less losses are applied to the unit hydrograph to obtain design flood hydrographs of the same ARI as the design storms.
- Runoff routing methods – for catchments with limited streamflow records: the recorded floods and associated rainfall are used to derive the catchment model parameters. Design storms less losses are applied to the model to produce design flood hydrographs of the same ARI as the design storms.

The following methods, which are commonly known as regional methods, are generally used for estimating design floods from ungauged catchments:

- Rational method –a probabilistic or statistical method in which a peak flow of a selected ARI is estimated from an average rainfall intensity of the same ARI.
- Regional flood frequency methods – such as the index flood method and multiple regression method.
- Synthetic unit hydrograph methods – using regional relationships for the parameters required to construct the unit hydrograph.
- Runoff routing methods – using regional relationships to estimate the model parameters.

As discussed in Book 3, Section 1.3 of *Australian Rainfall and Runoff* (Ball et al. 2016), the primary criterion for the selection of methods is that they should be based on observed data in the region of interest and have been peer reviewed by the profession. For most parts of Australia, data are available for the development of techniques that have been peer reviewed from both scientific and practical perspectives. The previous arbitrary methods based purely on engineering judgement have limitations that designers should be aware of, as use of these methods outside these limits can give poor or inaccurate results. For example, the rational method should only be used for small and simple rural catchments up to 25 km^2 and urban catchments up to 1 km^2.

The rational method is also not applicable for complex catchments, irrespective of size, such as:

- multiple streams
- branched catchments
- mixed land use catchments
- situations where a catchment may be inundated by another catchment
- situations where the catchment may overflow into an adjacent catchment
- catchments with significant storage capacity (dam, swamp and major retention/detention basin)
- irrigated land.

Several road agencies recommend use of rational method in certain applications, such as the design of simple longitudinal drainage, appropriate urban applications, or as a check method.

A brief description of each method is given in the following sections, but for a detailed description of each method, reference should be made to *Australian Rainfall and Runoff* (Ball et al. 2016).

Urbanised catchments

Flood estimation procedures for catchments that contain areas of urbanisation or are completely urbanised are the same in principle as those for rural catchments. Urban catchments, however, are typically more complex. The unit hydrograph method and runoff routing methods can all be used where suitable data are available. There are also specialised urban catchment models available. However, the approach and constraints for urban catchments is significantly different to rural catchments in some aspects of flood hydrology (Ball et al. 2016).

At present, there is a lack of data for the testing and calibrating of urban hydrological models or for the development of regional procedures for determining flow rates. Hence, rainfall-runoff models using statistical design rainfall are generally used for estimating design floods from urban catchments (see Section 2.1.1 for comment on the ARI of the design flood).

Reference should be made to Ball et al. (2016) for details of flood estimation for urbanised catchments.

2.2.2 At-site Flood Frequency Analysis

Flood frequency analysis (FFA) refers to procedures that use recorded and related flood data to identify underlying probability model of flood peaks at a particular location in the catchment. They can then be used to perform risk-based design and flood risk assessment, while providing input to regional flood estimation methods.

At-site flood frequency analysis is used if there is a long-term stream gauge located near the bridge site. If the gauge is located close to the bridge site, say with a difference in catchment area of less than 10%, the gauge can be used directly, with the calculated discharges modified for the small differences in catchment area. If the gauge is located further away, the flood frequency analysis can be used to help estimate parameters for a runoff routing model. Flood frequency analysis is usually the most reliable method for calculating design flood discharges, but it only provides peak discharges, not flood hydrographs. If flood hydrographs are also needed, say for calculating times of closure, these must be calculated in some other way. The two methods are by scaling recorded hydrographs or by using a runoff routing model. Even if the nearest stream gauge is located some distance from the bridge site or even in a neighbouring catchment, flood frequency analysis on this station may be of value to obtain local regional flood estimates to compare with general regional values.

Advantages

- Peak flood records represent the integrated response of the storm event with the catchment. They provide a direct measure of flood exceedance probabilities. As a result, FFA is not subject to the potential for bias, possibly large, that can affect alternative methods based on design rainfall (Kuczera et al. 2006).
- Comparative simplicity and capacity to quantify uncertainty arising from limited information.

Disadvantages

- The true probability distribution family is unknown. Unfortunately, different models can fit the flood data with similar capability, yet can diverge in the right-hand tail when extrapolated beyond the data.
- Short records may compromise the utility of flood estimates. Confidence limits inform the practitioner about the credibility of the estimate.
- It may be difficult, or impossible, to adjust the data if the catchment conditions under which the flood data were obtained have changed during the period of record, or are different to those applying to the future economic life of a structure or works being designed.
- The accuracy/reliability of FFA relies on the presence of an accurate rating curve at the gauge (many of them are inaccurate because of extrapolation and other reasons). Therefore, if the rating curve is not accurate this will skew the results of an FFA.

2.2.3 Regional Flood Methods

Estimation of peak flows on small to medium-sized rural catchments is required for the design of culverts, small- to medium-sized bridges, causeways, soil conservation works and for various planning and regulatory purposes. Typically, most design flood estimates for projects on small- to medium-sized catchments are on catchments that are ungauged or have little recorded streamflow data. In these cases, peak flow estimates can be obtained using a Regional Flood Frequency Estimation (RFFE) approach, which transfers flood frequency characteristics from a group of gauged catchments to the location of interest. Even in cases where there is recorded streamflow data it is beneficial to pool the information in the gauged record with the RFFE information. An RFFE technique is expected to be simple, requiring only readily accessible catchment data to obtain design flood estimates relatively quickly.

The RFFE method described in this section ensures that design flood discharge estimates are consistent with the gauged records and with results for other ungauged catchments in a region. It is recognised that there will be considerable uncertainty in estimates for ungauged catchments because of the limited number of gauged catchments available to develop the method and the wide range of catchment types that exist throughout Australia.

The RFFE technique presented in this section has been incorporated into a software tool referred to as RFFE Model 2015. The current version of the RFFE model can be accessed from the *Australian Rainfall and Runoff* website (Australian Rainfall and Runoff n.d.). The model requires the basic inputs for the catchment of interest to generate design flood estimates for six annual exceedance probability (AEPs) including 50%, 20%, 10%, 5%, 2% and 1%.

Australian regional flood frequency (ARFF) method

This methodology provides a regional flood peak estimate for small- and medium-sized catchments in Australia. It also applies for any size catchment up to 1000 km^2, including small catchments. The data requirements are catchment area and design rainfall data with specific parameters defined for different regions of the state. As with at-site flood frequency analysis, this method only provided flood peak discharges. If flood hydrographs are required, as they often are, a runoff-routing model should be ‘calibrated’ to the flood peaks from the ARFF and then used to calculate flood hydrographs. This procedure also calculates uncertainty limits which are useful for assessing risk in the design.

2.2.4 Unit Hydrographs

As unit hydrograph approaches are no longer widely applied in Australia, only a brief introduction is given here. A more detailed description of the unit hydrograph method and worked examples can be found in Ball et al. (2016).

Unit hydrographs represent an advance over time-area procedures because, rather than constructing a time-area curve from isochrones of travel time, which requires assumptions regarding the travel times from all points on the catchment, the unit hydrograph represents the actual flood response of the catchment to rainfall, and can be directly determined or estimated from recorded rainfall and streamflow data. As a consequence, the resulting unit hydrograph incorporates the effects of both translation and attenuation, and so reduces the assumptions needed in the time-area approaches.

2.2.5 Runoff Routing Model

This is a continuous simulation model for flood estimation, applied to design flood estimation. Types of runoff-routing models used to simulate the flow can be varied and depend upon the complexity required to provide unbiased simulation of the hydrologic process in the catchment.

Runoff routing model is a rainfall runoff model where flood hydrographs are calculated from rainfall using one of a range of different software packages. The most common packages used for bridge design include RORB (Laurenson, Mein & Nathan. 2012), XPRAFTS (Xpsolutions 2017) and URBS (n.d.), but there are several others that can also be appropriate. Rainfall inputs are calculated using the *Australian Rainfall and Runoff* method (Ball et al. 2016). Model parameters can be calibrated using recorded streamflow data if available, or can be calculated using the formulae for ungauged catchments given in Ball et al. (2016). The model parameters can also be calibrated using design flood peaks calculated using the *Australian Rainfall and Runoff* method (Ball et al. 2016). Runoff routing analysis gives both flood peak discharges and hydrographs.

When design floods have been calculated, they should be checked for consistency. This check can be with nearby similar catchments that have been previously analysed or with the regional graph of AEP 1% floods prepared by the relevant agency, such as Department of Natural Resources and Mines (2015).

Once the design floods are decided, this data is available for the next process, the hydraulic analysis.

2.2.6 Estimation of ULS Floods

2000 year ARI (0.05% AEP) flood

The 1 in 2000 year ARI (0.05% AEP) flood is derived by interpolation between the probable maximum flood (PMF) and the 1 in 50 ARI (2% AEP) and 1 in 100 year ARI (1% AEP) floods, as described in Ball et al. (2106). The probable maximum flood is generally derived by applying a runoff routing method to a range of estimates of the probable maximum precipitation for various durations. An assumption or estimate of the AEP to be assigned to the PMF must be made. Book 3 of Ball et al. (2016) offers guidance on the selection of the assigned probability for the probable maximum event. The 1 in 50 ARI (2% AEP) and 1 in 100 year ARI (1% AEP) floods may be estimated using either flood frequency analysis, or a runoff routing method or a unit hydrograph method.

500 year ARI flood (0.2% AEP)

The 1 in 500 year ARI (0.2% AEP) flood can be estimated by simply extrapolating the flood frequency curve from the 1 in 20 ARI (5% AEP), 1 in 50 ARI (2% AEP) and 1 in 100 year ARI (1% AEP) flood estimates with a line of best fit on log-probability paper.

Both the 0.2% and 0.05% AEP events can be estimated directly for the catchment in question using the rainfall IFD, temporal patterns and other techniques from Australian Rainfall and Runoff (Ball et al. 2016).

2.2.7 Field Observation of Floods

General

The importance of collecting and recording flood and associated rainfall data cannot be over emphasised. The availability of such data allows hydrological investigations and hydraulic design to be carried out with a greater level of confidence than if design flood estimates are based on theoretical estimates alone.

The responsibility for collection of elementary hydrological data rests as much with Local Government Authorities, as with State and Federal Government Departments and Instrumentalities. All these authorities should establish procedures for the systematic collection of flood data, such as flood debris levels, as such work is usually simple and inexpensive.

Information to be acquired

The information collected during flooding should include:

- peak levels
- flood gradients
- rates of rise and fall of flood
- rainfall data.

Measurements of velocities, depths and directions of overbank flow are sometimes of equal importance, but are often neglected. Information on these aspects should also be obtained if possible.

Particular attention should be given to any evidence of scour at bridges, as this may have resulted in increased waterway area and discharge through the structure during flood conditions.

It is important that flood data should be gathered either during or as soon as possible after a flood event.

Collection of data

The majority of flood height data is usually collected after the event by observation of debris lines or water marks left by the flood, or by information provided by the community. Care must be exercised in evaluating information offered by individuals, who:

- may consider the highest levels that they have observed personally to be the peak of the flood
- may considerably overestimate the flow depths and velocities, as a result of inexperience or the drama of the occasion
- will often be unaware of the actual nature of an event which has occurred overnight
- may, as a result of personal interest, exaggerate or understate flood levels.

Flood levels upstream and downstream from road crossings are relatively easily obtained, and apart from providing a basis for estimation of discharge through structures, will give an indication of the adequacy of existing waterways.

It should be noted that, as indicated in Section 4.10, the difference in water surface elevation between the upstream and downstream side of bridge approach embankments is not the backwater produced by the bridge. The maximum backwater is some distance upstream of the bridge as indicated in Section 4.11 and normal stage is not re-established until some considerable distance downstream, as shown on Figure 4.4 (A). As a guide to the measurement of backwater, it is recommended that upstream flood levels should be taken at least a bridge length away from the bridge opening and downstream levels at least four bridge lengths downstream from the bridge.

Flood levels and gradients can be established readily by photography, and in particular by aerial photography.

Similarly, rates of rise and fall can be assessed by observing the times at which recognisable features or objects become submerged or are uncovered.

Flow velocities should preferably be measured with flow meters, but when this cannot be done surface velocities in main streams and over flood plains can be obtained by timing the movement of floating objects. Directions, speeds and depths of overbank flow can be measured by field parties in boats using reference marks or survey controls to define the locations at which observations are made.

Aerial photographs are a most useful means of establishing patterns and concentrations of flow in wide and otherwise inaccessible waterways. It is important to record the dates and times at which photographs are taken.

Any particular features which affect flow characteristics or flood levels should be recorded. These may comprise backwater effects from constrictions downstream, changes in course of the stream, breakout flow from channels, tidal effects, etc. Awareness of such factors may be essential to an understanding of particular aspects of flood behaviour.

3. General Considerations in Waterway Design

3.1 Introduction

As already discussed in Section 2.1.3, Australia is a very large continent with wide variations in climate, topography and population density. Hence, standards and practices vary widely from jurisdiction to jurisdiction. It has not been possible, therefore, to include recommendations that cover all situations and for this reason, this publication is intended as a guide and not a code of practice. This will allow design engineers to adopt local practice, where it differs from the recommendations made later in this Guide.

The philosophy taken in the preparation of this Guide was that the frequently-used methods and procedures utilised by the road agencies for the hydraulic design of bridges, culverts and floodways should be included in detail, with reference being made to other material less frequently used.

3.2 Rivers and River Crossings

3.2.1 General

Fluvial geomorphology and river mechanics are subjects that engineers engaged in waterway design should be familiar with. However, it is beyond the scope of this Guide to cover fluvial geomorphology and river mechanics in detail and reference should be made to the publications referred to below for a detailed discussion on these subjects.

The most relevant Australian publications on the subject of fluvial geomorphology are as follows:

- *Fluvial Geomorphology in Australia* (Warner 1988) – this collection of specialist papers provides background into the geomorphology of rivers and related phenomena in Australia.
- *The Geomorphology of Australia's Fluvial Systems: Retrospect, Perspect and Prospect* (Tooth & Nanson 1995) – provides a comprehensive summary and bibliography on the subject.

The following US Federal Highway Administration (FHWA) publications provide detailed discussion on fluvial geomorphology and river mechanics as they apply to stream crossings:

- *Highways in the River Environment: Floodplains, Extreme Events, Risk, and Resilience* (FHWA 2016).
- *Stream Stability at Highway Structures* (Lagasse et al. 2012).

It should be recognised that although the geomorphology of Australian rivers is different from that of rivers in North America, the basic concepts of fluvial geomorphology presented in these publications apply to any river. In addition, the following reference provides an overview of fluvial geomorphology in terms of natural channel forms and fluvial processes:

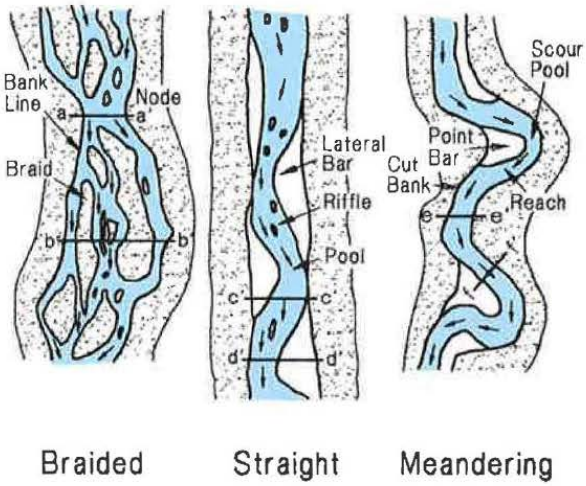
- *Applied Fluvial Geomorphology for River Engineering and Management* (Thorne, Hey & Newson 1997).

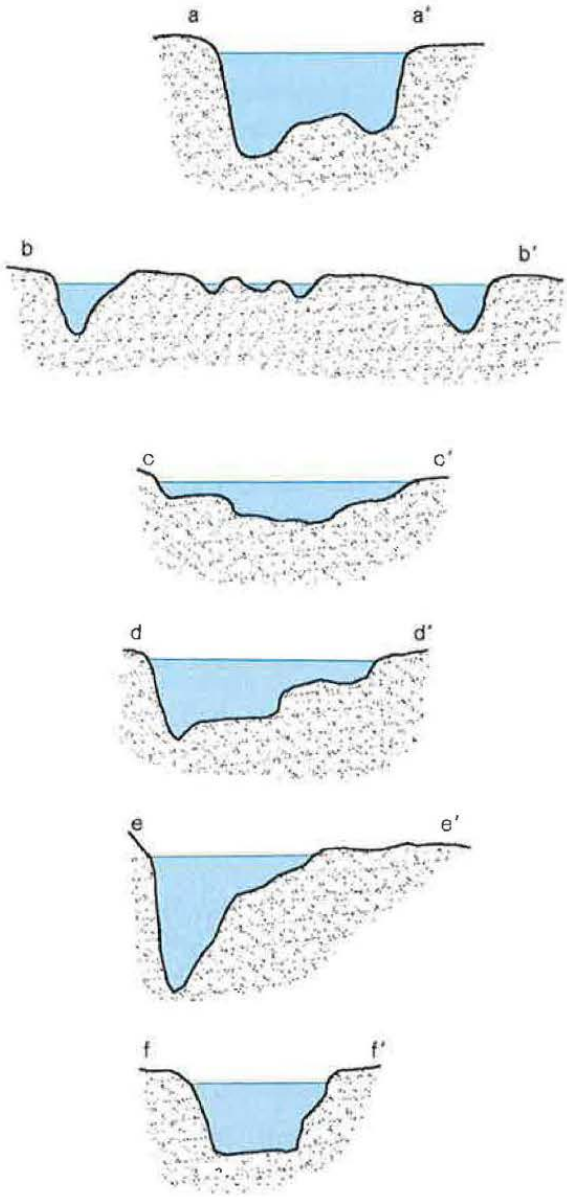
3.2.2 Types of Rivers

Rivers can be divided into those with floodplains and those without. Floodplains are usually not the direct result of large flood flows, but rather the result of lateral movement of the river from one side of the plain to the other over geological time. By definition, the floodplain is low enough to be completely inundated by floods with fairly low return periods.

Rivers can be further classified as either braided, straight or meandering, whether they have floodplains or not. The character of each classification is shown in Figure 3.1. Braided channels can, in some circumstances, be extremely unstable, with the channels shifting with each sharp change in discharge. This has resulted in many crossing failures.

Figure 3.1: River channel patterns





As seen in Figure 3.1, even straight rivers are to some degree sinuous. The sinuosity is a measure of this meandering feature. The sinuosity is defined as the ratio of the river's thalweg to the length of the valley proper. The thalweg is the path of deepest flow. Rivers with sinuosity less than 1.5 are usually considered straight.

Meandering rivers are commonly associated with erodible floodplains, although very regular and highly developed meanders have occurred in rivers incised in solid rock.

3.2.3 Dynamics of Natural Rivers

Frequently, environmentalists and hydraulic engineers consider a river to be static, i.e. unchanging in shape, dimensions and pattern. However, an alluvial river continually changes its position and shape as a consequence of hydraulic forces acting on its bed and banks. These changes may be slow or rapid and may result from natural environmental changes or from changes by man's activities.

A river that may appear stable may be relatively unstable because of the slow but implacable shift of a channel through erosion and deposition at bends, the shift of a channel to form chutes and islands, and the cut-off of a bend to form oxbow lakes. These changes are dependent on flood events, bank stability, permanence of vegetation on banks and floodplain land use.

Experience has shown that modifications to perennial rivers can be quite extensive, however, the dynamic effects of rivers in arid and semi-arid regions, and especially ephemeral (flowing occasionally) stream channels can be even more extensive and dramatic. When an engineer modifies a river channel locally, this local change can frequently modify channel characteristics both up and down stream. The response of a river to man-induced changes often occurs in spite of attempts by engineers to keep the anticipated response under control.

All rivers are governed by the same basic forces which determine the response to man-induced changes. The design engineer should have an understanding of these forces which include:

- geological factors, including soil conditions
- hydrological factors, including possible changes to runoff and the hydrological effects of changes in land use
- geometric characteristics of the stream, including the probable geometric alterations which may occur as a result of man's actions
- hydraulic characteristics such as depths, slopes and velocity and what changes may be expected in these characteristics in space and time.

River hydraulics

As just stated, rivers are dynamic and respond to changed environmental conditions. The extent of the change depends on the forces acting on the system. The mechanics of flow in rivers is a complex subject, the major complicating factors of which are:

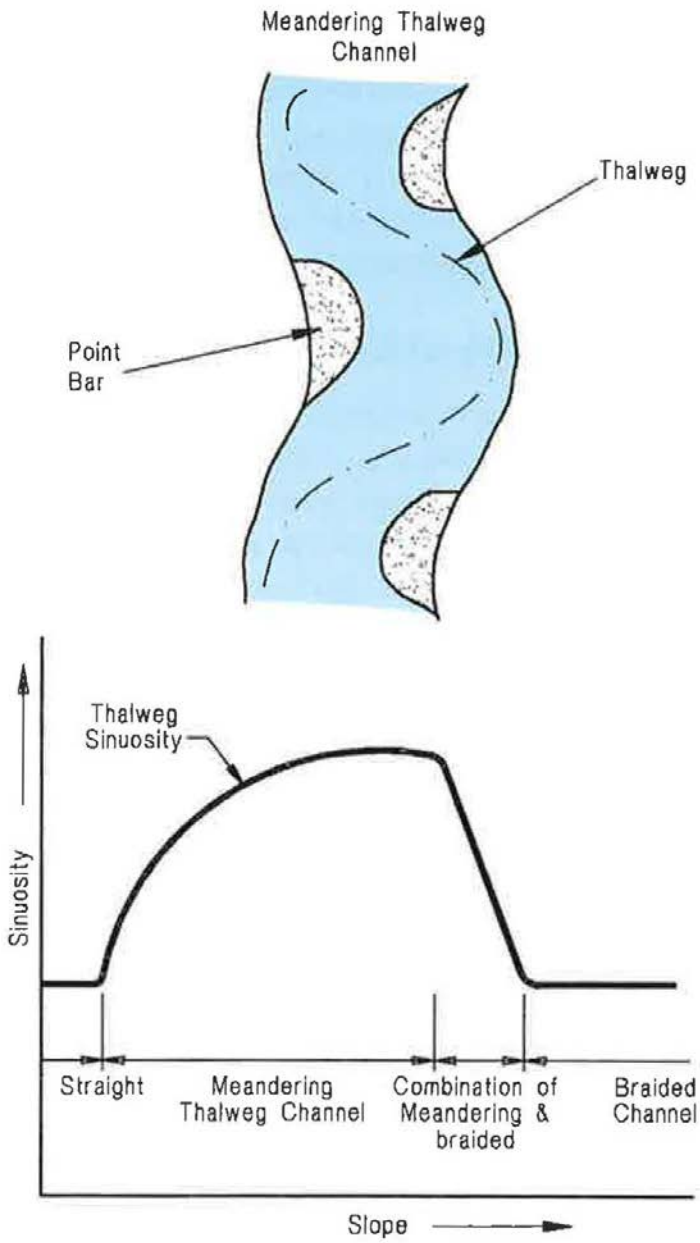
- the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a river system
- the continual evolution of river channel patterns, channel geometry, bars and forms of bed roughness with changing water and sediment discharge.

In order to understand the responses of a river to the actions of man and nature, a few hydraulic and geomorphic concepts are now presented.

As stated in Section 3.2.2, river forms are broadly classified as straight, meandering, braided or some combination of these classifications. However, any changes that are imposed on a river may change its form. The dependence of river form on the slope, which may be imposed independent of the other river characteristics, is illustrated schematically in Figure 3.2. By changing the slope, it is possible to change the river from a meandering one that is relatively tranquil and easy to control to a braided one that varies rapidly with time, has high velocities, is subdivided by sandbars and carries relatively large quantities of sediment. Such a change could be caused by the natural or artificial shortening of the length of the stream. Conversely, it is possible that a slight decrease in slope could change an unstable braided river into a meandering one.

The significantly different channel dimensions, shapes and patterns associated with different quantities of discharge and amounts of sediment load indicate that as these independent variables change, major adjustments of channel form can be anticipated.

Figure 3.2: Sinuosity versus slope with constant discharge (channel patterns are illustrated on Figure 3.1)



An insight into the direction of change, the magnitude of change, and the time involved to reach a new equilibrium can be gained by studying the river in a natural condition; having knowledge of the sediment and water discharge; being able to predict the effects and magnitude of man's future activities; and applying to these a knowledge of geology, soils, hydrology and hydraulics of alluvial rivers.

3.2.4 Types of Encroachment

Stream crossings generally impose some degree of encroachment of the river or floodplain. In some instances, particularly in mountainous regions or in river gorges, river crossings can be accomplished with absolutely no encroachment on the river. The bridge and approaches can be located far above and beyond any possible flood stage. More commonly, the economics of crossings may require substantial encroachment on the river and its floodplain. The encroachment can be in the form of earth fill embankments over the floodplain or into the main channel itself, reducing the required bridge length.

3.2.5 Geometry of Bridge Crossings

The geometric properties of stream crossings are dependent on the conditions at the site. Abutments may be of the spill through type or vertical wall type. The approaches may be skewed or normal to the direction of flow, or one approach may be longer than the other, producing an eccentric crossing.

Abutments may be set back from the channel banks to provide room to pass the flood flow or simply to provide access under the bridge, or the abutments may extend up to the banks or even protrude over the banks, constricting the low flow channel. Piers, dual bridges for multi-lane highways, channel bed conditions, and guide banks add to the list of geometric classifications.

3.2.6 Effects of Bridge Construction on River Systems

Bridge construction can have significant general and local effects on the geomorphology and hydraulics of river systems. Hence, it is necessary to consider induced short-term and long-term responses of the river, the impact on environmental factors, the aesthetics of the river environment and short-term and long-term effects of erosion and sedimentation on the surrounding landscape and the river.

Immediate responses

Constrictions generally lead to general and local scour (see Section 5.2.7), and the sediments removed through this action are usually dropped in the immediate reach downstream. In addition, the development of crossings and the constrictions of river sections may have a significant effect on the water level in the vicinity and upstream of the bridge.

As a consequence of the construction of bridges and their approaches, areas adjacent to the bridge site become highly susceptible to erosion. Rainfall causes surface runoff and the accompanying erosion can significantly increase the sediment yield to the river unless careful control is exercised. Fine sediments are easily transported and generally pollute the river downstream.

Delayed responses

Often it is necessary to employ training works in connection with bridges to favourably align flow with the bridge openings. When such training works are used, they can straighten the channel, shorten the flow line and increase the velocity within the channel. Any such changes made in the system that cause an increase in the gradient may cause an increase in velocities. The increase in velocity increases local and general scour with subsequent deposition downstream where the channel takes on its normal characteristics.

3.3 Environmental Considerations

3.3.1 General

As the preservation of environmental quality has become a matter of national interest and priority (National Committee on Environmental Engineering, Institution of Engineers, Australia, 1992), the decisions made on the location, design and construction of river crossings should, as far as possible, avoid or minimise the adverse effects on scenic, natural, historical, archaeological, recreational and social values and resources of the project area. To do this, the engineer must have sufficient knowledge to recognise potential problems.

Environmental impact must be included in the investigation of all bridge sites. To achieve this the following steps should be taken:

1. Several alternative bridge sites should be identified based on both environmental and technical considerations. An inventory should be made of the environmental resources at each site. The inventory should check the area for its uniqueness in supporting specific vegetation, wildlife and aquatic life, especially rare and endangered species. Where necessary relevant specialists should be consulted to provide advice and evaluate the response of the species to stress.
   a. Areas of archaeological significance should be identified and included in the inventory. Existing and future uses of the sites should also be noted.
   b. Archaeological and ethnographic Aboriginal sites should also be identified and the traditional Aboriginal land owners consulted to determine the significance of the sites and what impact a bridge might have on them, and any special requirements that might be required regarding location of piers and abutments.
2. If necessary, an interdisciplinary study should then be made of the impact, at each site, of construction on the environmental factors identified in the first step. The impact of alternative forms of construction should also be included in this study.
3. Preliminary plans and cost estimates that provide the best compromise between functional cost and environmental impact should then be prepared for each site. These plans should include measures to rehabilitate the sites and ameliorate any expected harmful impacts. The final choice can then be made with an understanding of the impact of the crossing.

Specific considerations pertinent to site selection and bridge construction are described in more detail below.

3.3.2 Site Selection

While the site selection of a bridge is usually closely dictated by the location planning of the proposed road, the site must be selected with full knowledge of its environmental impact.

To minimise the environmental impact the site should be located where:

- satisfactory geological and soil conditions exist
- it is away from the reaches of highly-unstable channels
- possible adverse effects on other existing bridges and hydraulic structures can be avoided
- it is possible to minimise the hazards from floods, landslides, cyclones, earthquake and subsidence
- river banks are stable
- ecological impact is acceptable
- aesthetic considerations are favourable.

3.3.3 Construction Effects

Although construction may be of short duration as compared to the operating life of the project, some changes during construction could have long-term damaging effects. The impact of each of the construction activities on the environment should be assessed and measures taken to minimise such impact. Erosion control and other pollution control measures, the impact on water supplies, and restoration of the landscape after completion of construction must be considered.

3.4 Hydraulics of Open Channel Flow

3.4.1 General

A brief review and commentary is provided in this section on the hydraulics of open channel flow and its application in the hydraulic design of bridges, culverts and floodways. For more detailed information, reference should be made to standard texts on the subject, such as:

- *Open-channel Hydraulics* (Chow 2009)
- *Hydraulics of Open Channel Flow* (Chanson 2004).

3.4.2 Types of Flow

The classification of flow according to change in depth with respect to space and time is shown in Figure 3.3.

The simplest way to classify open channel flow is to examine changes in flow depth with respect to time and space. Flow in an open channel is said to be steady if the depth of flow does not change with time. The flow is unsteady if the depth changes with time. Examples of unsteady flow are traveling surges and flood waves in an open channel.

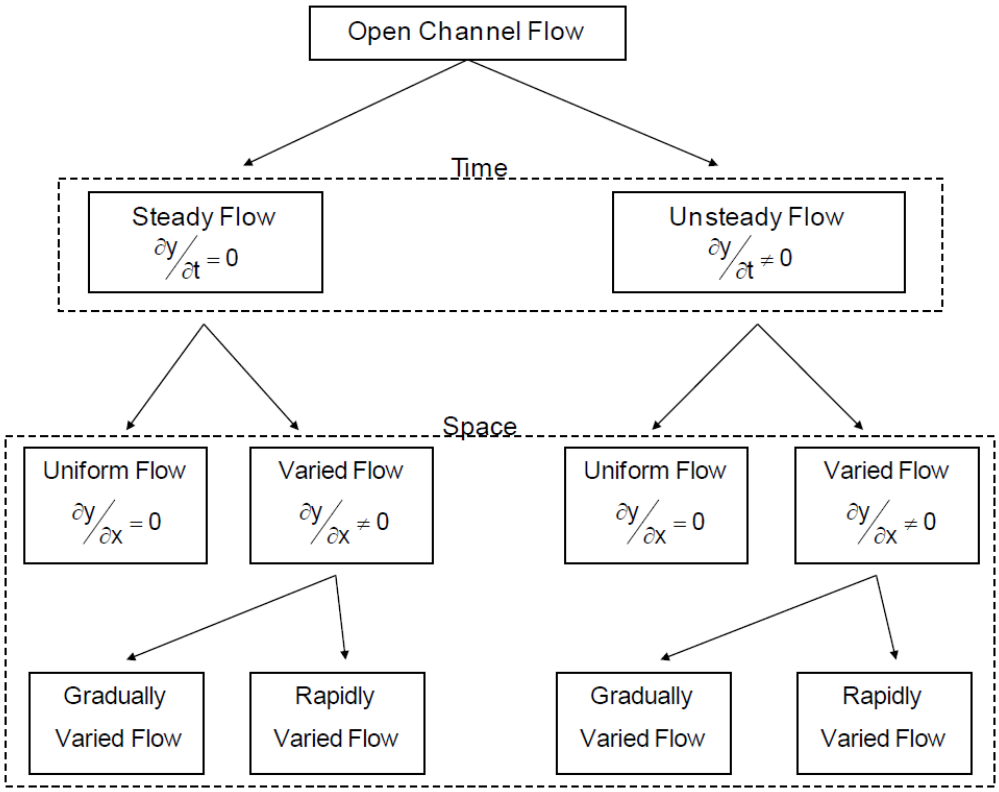
Open channel flow is said to be uniform if the depth of flow is the same at every section of the channel. This is only possible where the channel cross-section, roughness and slope are constant. Flow is non-uniform if the depth of flow varies along the channel. This occurs when channel properties vary from section to section.

Non-uniform flow can be either rapidly or gradually varied. The flow is rapidly varied if the depth changes abruptly over a comparatively short distance such as a hydraulic jump. Otherwise, it is gradually varied.

Whether laminar or turbulent flow exists in an open channel depends upon the Reynolds Number, R_e, of the flow. Reynolds Number is the ratio of viscous forces to inertial forces. Laminar flow occurs when viscous forces are predominant compared with the inertial forces, and turbulent flow occurs when the inertial forces are great compared with forces of viscosity. Laminar flow in open channels occurs very infrequently, except with special liquids such as oils or extreme concentrations of sediment mixtures. Further clarification of laminar and turbulent flow in open channels may be found in Chow (2009) and Munson et al. (2016).

The effect of gravity upon the state of flow is represented by the ratio of inertial forces to gravitational forces, called the Froude Number, F_r. If $F_r = 1$, the flow is critical; if $F_r < 1$, the flow is subcritical and if $F_r > 1$, the flow is supercritical. Subcritical and supercritical flows exist only with a free surface or interface.

Figure 3.3: Classification of flow according to change in depth with respect to space and time



Source: Zevenbergen et al. (2012).

There are three types of flow that may be encountered in a bridge waterway design. These are labelled Types I, II and III as shown in Figure 3.4. The long dashed lines shown on each profile represent normal water surface, or the stage the design flow would assume prior to placing a constriction in a channel. The solid lines represent the configuration of the water surface on the centre line of the channel in each case, after the bridge is in place. The short dashed lines represent critical depth, or critical stage in the main channel (y_{1c} and y_{4c}) and critical depth within the constriction, y_{2c}, for the design discharge in each case.

Since normal depth is shown essentially the same in all four profiles, the discharge and slope of the channel must increase in passing from Type I to Type IIA, to Type IIB, to Type III flow.

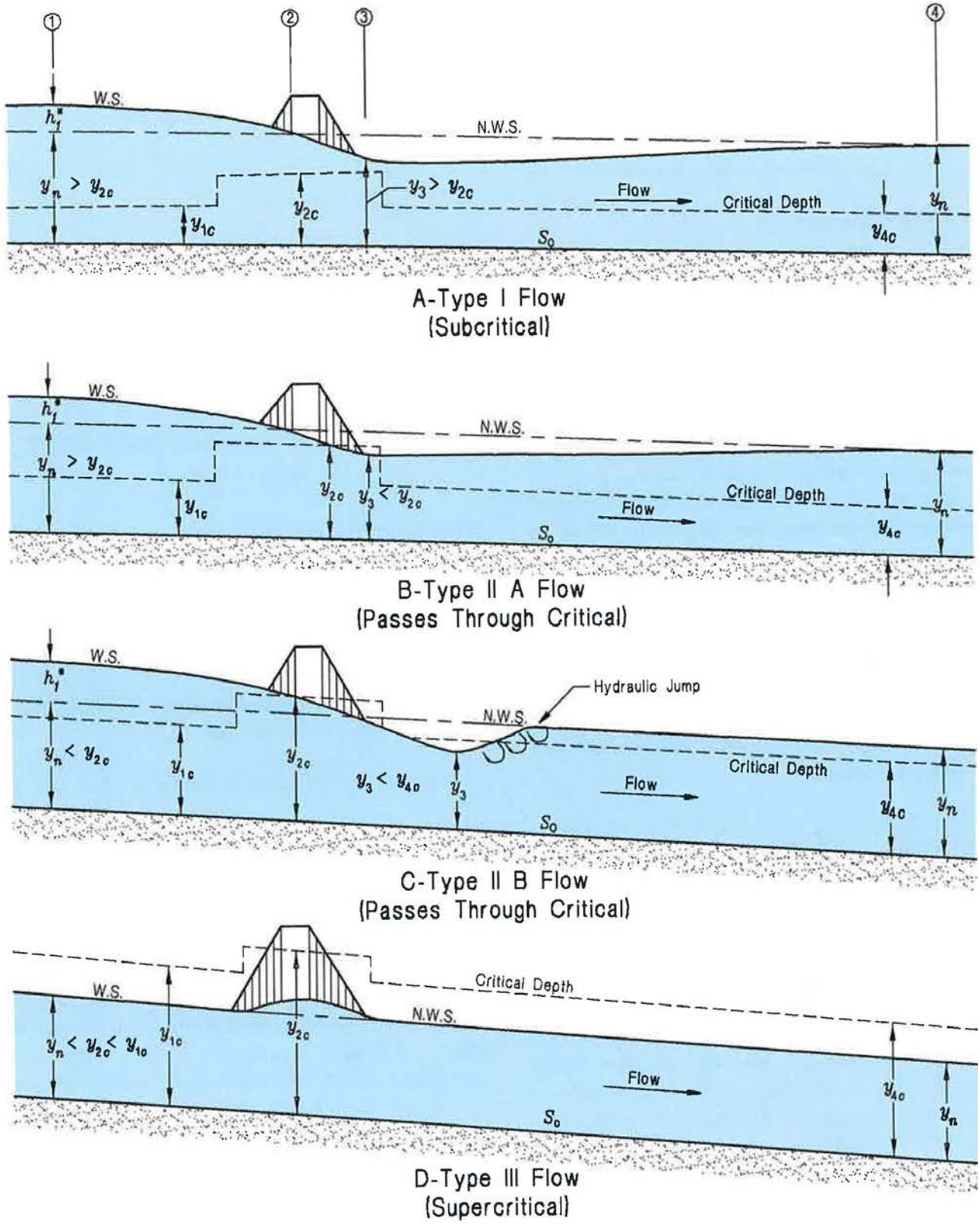
Type I flow – referring to Figure 3.4-A, it can be seen that normal water surface is everywhere above critical depth. This has been labelled Type I or subcritical flow, the type usually encountered in practice. The backwater expression for Type I flow is obtained by applying the conservation of energy principle between sections 1 and 4.

Type IIA flow – there are two variations of Type II flow, which will be described here as Types IIA and IIB. For Type IIA flow (Figure 3.4-B), normal water surface in the unconstricted channel again remains above the critical depth throughout, but the water surface passes through critical depth in the constriction. Once the critical depth is penetrated, the water surface upstream from the constriction, and thus the backwater, becomes independent of conditions downstream (even though the water surface returns to normal at section 4). Thus the backwater expression for Type I flow is not valid for Type II flow.

Type IIB flow – The water surface for Type IIB flow (Figure 3.4-C), starts out above both the normal water surface and the critical depth upstream and passes through the critical depth in the constriction. It then dips below the critical depth downstream from the constriction and then returns to normal. The return to normal depth can be rather abrupt as shown on Figure 3.4-C, taking place in the form of a poor hydraulic jump, since normal water surface in the stream is above the critical depth. A backwater expression applicable to both Types IIA and IIB flow has been developed by equating the total energy between section I and the point at which the water surface passes through critical stage in the constriction.

Type III flow – in type III flow (Figure 3.4-D), the normal water surface is everywhere below the critical depth and the flow throughout is supercritical. This is an unusual case requiring a steep gradient. However, such conditions do exist, particularly in mountainous regions. Theoretically, backwater should not occur for this type, since the flow throughout is supercritical. It is more than likely that an undulation of the water surface will occur in the vicinity of the constriction, however, as indicated on Figure 3.4-D.

Figure 3.4: Types of flow



Open channel flow can be classified in many ways. Flow can be classified as either steady or unsteady. Flow can also be classified as uniform or non-uniform (varied). Non-uniform flow can be further classified as gradually varied or rapidly varied. The flow can also be subcritical or supercritical, and depending upon the turbulence of the flow field, the flow can be classified as laminar (low R_e) or turbulent (high R_e). As just discussed, there are four classifications needed to describe the type of flow in an open channel:

- uniform or non-uniform
- steady or unsteady
- laminar or turbulent
- subcritical or supercritical.

One from each of these four types must exist. Because the classifying characteristics are independent, 16 different types of flow can occur. In most open channel flow problems related to the hydraulics of bridges, culverts and floodways, it is sufficient to study flow behaviour under steady flow conditions only. Although flow in natural channels is almost always unsteady, it is often analysed in a quasi-steady state, unless special circumstances prevail. This is due to the fact that, while flow may vary over time, it varies over a long enough time that it can be treated as steady flow.

3.4.3 Basic Equations of Open Channel Flow

The basic equations of flow in open channels are derived from the three conservation laws (Zevenbergen et al. 2012) including: (1) the conservation of mass, (2) the conservation of energy, and (3) the conservation of linear momentum.

The conservation of mass is another way of stating that (except for mass-energy interchange) matter can neither be created nor destroyed and it can be stated as shown in Equation 2. A control volume is a volume which is fixed in space or moving with the fluid and through whose boundary matter, mass, momentum, energy can flow:

$$\left|\begin{matrix}Mass\ flux\ out\ of \\ the\ control\ volume\end{matrix}\right| - \left|\begin{matrix}Mass\ flux\ into\ the \\ control\ volume\end{matrix}\right| + \left|\begin{matrix}Time\ rate\ of\ change\ in\ mass \\ in\ the\ control\ volume\end{matrix}\right| = 0 \qquad 2$$

The conservation of energy uses the first law of thermodynamics which states that energy must at all times be conserved. This law accounts for energy entering, leaving and accumulating in either a system or a control volume. The conservation of energy is used in one-dimensional models to account for horizontal velocity variation across a cross-section (channel versus floodplains) and is used by some two-dimensional models to account for vertical velocity variation at a point. Refer to (Zevenbergen et al. 2012) for full details.

The principle of conservation of linear momentum is based on Newton's second law of motion which states that a mass (of fluid) accelerates in the direction of and in proportion to the applied forces on the mass. The statement of conservation of linear momentum is presented in Equation 3:

$$\left|\begin{matrix}Flux\ of \\ momentum \\ out\ of\ the\ control \\ volume\end{matrix}\right| - \left|\begin{matrix}Flux\ of \\ momentum \\ into\ the\ control \\ volume\end{matrix}\right| + \left|\begin{matrix}Time\ rate\ of \\ change\ of \\ momentum\ in\ the \\ control\ volume\end{matrix}\right| = \left|\begin{matrix}Sum\ of\ the\ forces \\ acting\ on\ the\ fluid \\ in\ the\ control \\ volume\end{matrix}\right| \qquad 3$$

The forces affecting the body fall into two classes: surface forces and body forces. Depending on the situation, other external forces need to be applied, such as a surface force due to wind blowing over the control volume. This force plays a significant role when analysing currents during a hurricane storm surge.

3.4.4 Flow Resistance and Other Hydraulic Issues

Flow resistance

Resistance to flow can be divided into shear resistance and resistance due to the difference in pressure from the upstream side to the downstream side of an object (form drag). The Chezy and Manning Equations are the two most commonly used equations for the computation of steady flow in natural channels. Flow velocity in the channel depends on its cross-sectional shape (among other factors), whilst the resistance to the flow depends upon the shear stress acting over the channel boundary, the wetted perimeter. For uniform flow, the flow resistance equations are developed based on the Manning Formula and quantified forces acting on a control volume. It is important to estimate correctly the Manning resistance coefficient 'n' in order to determine either the flow given the depth or determine the depth given the flow in a natural channel. Different methods to estimate the Manning resistance coefficient are presented in Section 3.4.8.

Drag force

Resistance is due to the difference in pressure from the upstream side to the downstream side of an object (form drag). When flow passes over an object a resistance to flow is created that depends upon the shape or form of the boundary of the object. The shape of the boundary (i.e. a bridge pier) causes a deflection of the streamlines and local acceleration of the fluid. Consequently, a change in pressure takes place from the upstream to the downstream side of the boundary, which is also referred to as a normal stress. The summation of the forces over the surface results in a drag force on the boundary and a pressure resistance against the fluid. Drag forces can be caused by bridge components such as piers and a submerged superstructure, as well as additional debris.

Weir flow

A weir may be described as any regular obstruction over which water flows. The two types of weir applications that relate to bridge hydraulics are a broad-crested weir and an ogee spillway structure. A more complete coverage of weir flow may be found in Chow (2009). Diversion structures are used to divert water from an existing natural watercourse into an off-channel conveyance system. Very often the shape is that of an ogee weir. The flow over a broad-crested weir will occur at critical depth for an ideal fluid flow. When flow overtops a weir and there is significant tail-water the discharge will be reduced due to the submergence of the weir.

3.4.5 Normal Depth and Normal Velocity

For a given flow in a channel of constant cross-section, roughness and slope, there exists only one depth, h_n,(the normal depth) that satisfies the Manning Equation (Equation 4):

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \qquad 4$$

where

A	=	area (m^2) of cross-section of flow
R	=	hydraulic radius = A/WP
WP	=	wetted perimeter (m) of the cross-section of flow
S	=	hydraulic slope (m/m)
n	=	Manning roughness coefficient

As both A and WP depend on the depth of flow, an iterative solution is required to calculate the normal depth, h_n. The normal velocity is then, $V_n = Q/A$.

The Manning Equation (Equation 4) will only give an accurate estimate of normal depth if the flow, channel cross-section, roughness and slope are constant over a sufficient distance to establish uniform flow conditions. Such conditions seldom, if ever, occur in natural streams because channel sections change from point to point. However, the Manning Equation (Equation 4) can be applied to most stream flow problems where the channel sections, roughness and slope are reasonably uniform.

It should be recognised that the value of Manning's ‘n’ utilised for estimating flow in natural streams is a composite value accounting for all the energy losses due to bed and bank roughness, vegetative retardance and turbulence at channel bends.

Initial estimates of the hydraulic slope for natural streams can be obtained by measuring bed slope. However, better estimates of the hydraulic slope can generally be obtained from measurements of water levels of a stream in flood or from debris after the event.

3.4.6 Critical Depth

The magnitude of critical depth depends only on the flow and shape of the channel, and is independent of the slope or channel roughness. Thus, for any given size and shape of channel, there is only one critical depth for a particular discharge.

Critical depth is an important value in hydraulic analysis because it is a control in reaches of non-uniform flow, whenever flow changes from subcritical to supercritical. Typical situations in which critical flow occurs are:

- at a constriction, such as the entrance to a culvert on a steep slope
- at the crest of a weir, such as a floodway on a road
- at the outlet of culvert discharging with a free outfall or into a relatively wide channel.

3.4.7 Non-uniform Flow

With subcritical flow, a change in channel shape, slope, or roughness affects the depth of flow upstream. If sufficiently severe, the depth of flow can differ significantly from the normal depth as calculated by the Manning Equation (Equation 4).

For example, given subcritical flow, a constriction caused by a structure, such as a bridge or culvert, causes an increase in flow depth upstream of the structure. The water surface profile upstream of the structure will be asymptotic to the upstream normal water surface profile. This water surface profile is called a backwater curve and the flow is said to be under downstream control.

Similarly, at a sudden channel enlargement such as a culvert flowing with subcritical flow and not flowing full, the flow depth at the end of the culvert is at critical depth. The water surface profile upstream of this point will be asymptotic to the normal water surface profile in the culvert. This water surface profile is called a drawdown curve and again the flow is under downstream control.

In such cases the use of the Manning Equation (Equation 4) is no longer directly applicable. If information on the depth of flow is required, detailed water surface profile calculations have to be undertaken.

With supercritical flow the influence of changes in channel shape, slope or roughness cannot be reflected upstream. However, the change may affect the depth of flow downstream. In such cases the flow is said to be under upstream control and in a similar manner to downstream control, detailed water surface profile calculations have to be undertaken, if information on the depth of flow is required.

3.4.8 Channel Rating

It is important that the normal stage height of a water course, for a design flood discharge, be determined as accurately as possible at the site of the stream crossing (i.e. the bridge, culvert, or floodway). This may be accomplished from stream gauging records, or if these are unavailable, from a theoretical approach such as the slope area method (see below), utilising records of peak floods as a check where these are available.

When using peak flood levels to check stage heights, derived theoretically, care should be taken to ensure that the flood levels do not include the effects of backwater.

Slope area method

The slope area method divides an irregular stream cross-section at a particular stage height into smaller, roughly rectangular sections, where the Manning Equation (Equation 4) is used to calculate the discharge for each sub-section separately, and then summed to calculate the overall discharge. This can be repeated for other stage heights and a stage-discharge rating curve drawn.

Channel roughness

The selection of the Manning roughness coefficient, n, is of prime importance in evaluating waterway discharge. In practice, tables of Manning factors are published in a variety of sources such as Chow (2009). Such sources will often include pictures of calibrated streams (streams where the ‘n’ values have been determined from flood data).

In general, Manning’s ‘n’ values should be calibrated whenever observed water surface elevation information (gauged data, as well as high water marks) is available. When gauged data is not available, values of ‘n’ computed for similar stream conditions or values obtained from experimental data should be used as guides in selection ‘n’ values (Brunner 2016).

The selection of an appropriate value for Manning’s ‘n’ is very significant to the accuracy of the computed water surface elevations. The value of Manning’s ‘n’ is highly variable and depends on a number of factors including: surface roughness; vegetation; channel irregularities; channel alignment; scour and deposition; obstructions; size and shape of the channel; stage and discharge; seasonal changes; temperature; and suspended material and bedload.

Value of Manning factors for natural channels and floodplains can range from 0.025 (channel, clean, straight, no deep pools) to 0.2 (floodplain, dense trees, straight) (Zevenbergen et al. 2012). Values for artificial channels are typically lower, especially if concrete lined (0.011 to 0.027), but can have similar values if vegetation lined or poorly maintained (up to 0.14) (Brunner 2016).

Refer to Appendix A for a full list of Manning’s ‘n’ values.

3.5 Design Considerations

3.5.1 General

Stream crossings can be achieved by waterway structures comprising bridges, culverts or floodways, or a combination of these structures. The selection of the waterway to be provided will depend on the size of the stream to be crossed and the level of serviceability to be provided to traffic.

As stated in Section 2.1.2, the crossing should be designed to pass the total waterway design flood with minimal damage. However, there can be considerable uncertainty involved in estimating design floods and the risk of larger floods occurring during the life of waterway structures is quite large. It is desirable, therefore, that the mode of failure with these larger floods be considered and crossings designed to ensure that damage is kept to a minimum should these larger floods eventuate. This can be achieved by providing a low or sacrificial section of road adjacent to the bridge or culvert, which will pass larger floods without the waterway structures being overtopped. Where this is not possible a conservative estimate of the design flood should be utilised for design.

3.5.2 Factors Affecting Selection of Waterway Structures

The factors to be considered in determining the type and size of waterway structure to be provided at a stream crossing are as follows:

- the level of serviceability to be provided to traffic and the magnitude of this design flood
- the magnitude of the total waterway design flood where a lower level of serviceability is to be provided to traffic
- road alignment and geometric standards
- the topography at the site and geometric characteristics of the stream
- the hydraulic aspects of the stream such as stage/discharge and velocity/discharge relationships, and flow patterns at the site
- limits on backwater imposed by the relevant waterway authority or resulting from development upstream
- requirements for navigation
- soil conditions and potential for scour at the site
- the incidence and nature of debris carried by the stream during floods
- construction and maintenance considerations
- environmental considerations.

Figure 3.5 below shows an example where there was no relief flood structure provided on the road approaches (left hand side image), although a waterway opening was provided at the creek crossing. As a result, erosion occurred at the bridge opening as shown in the right hand side image.

Figure 3.5: Conjola Creek, NSW

Source: Roads and Maritime (n.d.).

Generally, it is obvious that either a bridge or culvert is the appropriate waterway structure for a specific site. Occasionally, however, waterway openings can be provided by either a culvert or bridge. Estimates of costs and risks associated with each will indicate which structure alternative should be selected on the basis of economics. Other considerations which may influence selection of structure type are given in Table 3.1.

In addition, the stream classification where fish passage and low flow is required where bridge is in favour of culvert.

Table 3.1: Advantages and disadvantages of bridges and culverts

Structure	Advantages	Disadvantages
Bridges	• Less susceptible to clogging with debris • Waterway increases with rising water surface until water surface begins to submerge superstructure • Scour increases waterway opening • Minimal impact on aquatic environment and wetlands • Widening does not usually affect hydraulic capacity	• Require more structural maintenance than culvert • Spill slopes susceptible to erosion and scour damage • Piers and abutment susceptible to failure from scour • Buoyant, drag and impact forces are hazards to bridges • Susceptible to stream channel migration
Culverts	• Changes to vertical geometry and road width can generally be accommodated by extending culvert ends • Require less structural maintenance than bridges • Capacity increases with stage • Usually easier and quicker to build than bridges • Scour is localised, more predictable and easier to control • Storage can be used to reduce peak discharge	• Silting may require periodic cleaning • No increase in waterway as stage rises above soffit • Susceptible to clogging with debris • Susceptible to scour at outlets • Susceptible to abrasion and corrosion damage • Extension may reduce hydraulic capacity • Inlets of flexible culverts susceptible to failure by buoyancy • Rigid culverts susceptible to separation at joints • Susceptible to failure by piping

Floodways are generally provided where traffic volumes are low, under the following circumstances:

- where flow across the road will be infrequent or of short duration
- in conjunction with bridge or culvert, where the bridge or culvert is designed to pass a lesser flood than the total waterway design flood; the bridge or culvert may be designed such that it will not be overtopped or be designed to be submerged as part of the floodway
- where it is impractical or uneconomic to construct a bridge or culvert.

Refer to Austroads (2013a) for more information on the selection of culverts and floodways.

3.5.3 Channel Modification

A primary objective in the design of a stream crossing should be to minimise disturbance to the stream. Channel modifications should only be made where it is necessary to avoid multiple or highly-skewed stream crossings.

Channel modifications associated with road crossings generally involve only short reaches of stream. However, environmental concerns for stream velocity, flow depth and factors important to the stream ecosystem, and hydraulic concerns for stream bed and bank stability make it advisable not to undertake channel modifications unless there is no practical alternative.

Figure 3.6 shows the failure of a buried corrugated steel culvert in NSW, where the depth of fill was large. The combination of blockage and pavement failure could be catastrophic.

Figure 3.6: Failure of a buried corrugated steel culvert in NSW

Source: Roads and Maritime (n.d.).

3.5.4 Energy Dissipation Structures

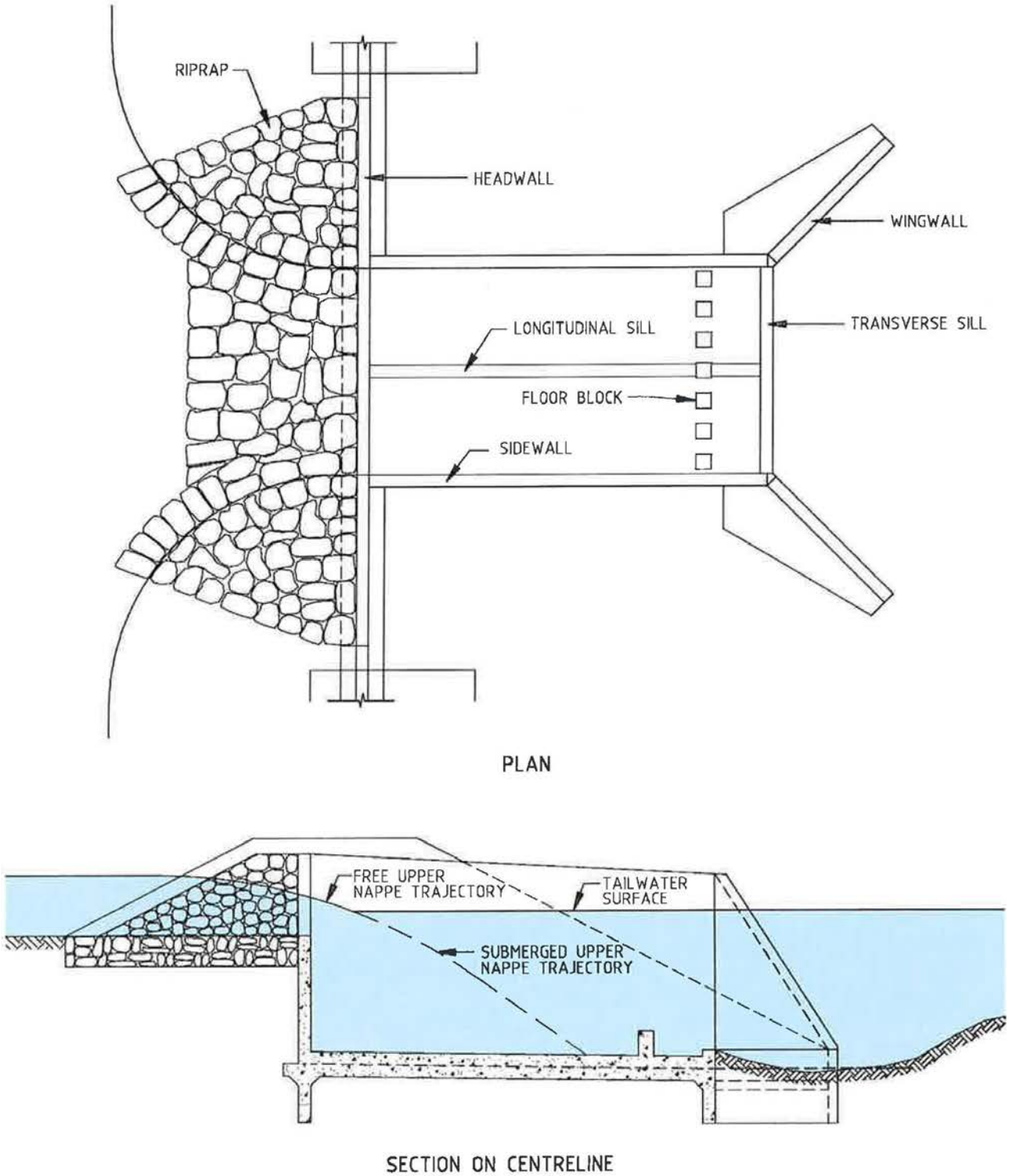
As indicated in Section 3.5.3 direct human interference with natural channels, such as in river realignment or stabilisation works associated with stream crossings can initiate channel changes which may propagate through the river system. In these situations, energy dissipation structures can be employed along channels and at culvert inlets or outlets for the purpose of controlling channel grades and for preventing or stabilising erosion of the channel bed and banks. A typical drop structure is shown on Figure 3.7.

Guidelines for the selection and design of energy dissipation structures can be found in the following references:

- *Stream Stability at Highway Structures* (Lagasse et al. 2012)
- *Hydraulic Design of Energy Dissipators for Culverts and Channels* (Thompson & Kilgore 2006)
- *Energy Dissipation in Hydraulic Structures* (Chanson 2015).

Where more unusual or complex situations exist, specialist advice and hydraulic model studies may be required for the design of these structures.

Figure 3.7: Basic elements of the straight drop structure



3.5.5 Data Requirements and Sources

There is a wide variety of information that is pertinent to waterway design and analysis. Table 3.2 provides a summary of the various types of information, their use, and sources. Although all of the data listed in Table 3.2 can be important in a bridge hydraulic study, geometric data are the greatest source of uncertainty and error. If the geometry is incorrect, then the flow, velocity, depth or water surface elevation must be incorrect. Geometric accuracy includes elevation, reach lengths, and bridge and roadway geometry.

Table 3.2: Data used in hydraulic studies

Type of information	Use	Possible sources
Floodplain topography	Hydraulic model geometry	Local and state governments, Geoscience Australia
Channel geometry and bathymetry	Hydraulic model geometry	
Current/recent aerial photography	Land use and roughness, channel boundaries	Local and state governments, Geoscience Australia
Historic aerial photography	Land use and roughness, channel boundaries, channel migration	Local and state governments, Geoscience Australia
Existing structure information	Hydraulic model geometry	Local and state governments, asset owners
Flood insurance and other flood hazard studies	Hydrology, flood history, channel and floodplain roughness information, flood profiles, coastal flooding range lines	
Flood maps	Floodplain delineation, base flood elevations, floodway boundaries	Government agencies
Existing hydraulic models	Hydraulic modelling	
Bore logs	Sediment size, scour analysis, erodibility assessment	Australian Soil Resource Information System, CSIRO
Core samples	Sediment gradation, scour analysis, erodibility testing	Australian Soil Resource Information System, CSIRO
Floodplain and channel roughness	Hydraulic model Manning Equation ‘n’ determination	
Bed and bank sediment surface and near-surface samples	Sediment gradation, scour analysis	Water and environmental agencies
Coastal hydrographic survey maps, data and coastal elevation models	Tidal hydrodynamic model geometry	Government agencies
Existing bridge inspection reports	Channel stability assessment	Local and state governments, asset owners
Gauge data	Flood frequency analysis, historic flooding, hydraulic model calibration and validation	Water authorities
Tide gauge data	Astronomic tide, water surface elevation frequency analysis	Bureau of Meteorology

Source: Zevenbergen et al. (2012) and Chapter 4, part 1 Ball et al. (2016).

3.6 Computer Modelling

With the development of modern computing and programming languages, more complex hydrologic and hydraulic modelling has become possible, as well as making it possible to rapidly complete trial and error solutions for more standard analysis. Some examples of what can be modelled include:

- steady vs unsteady flow analysis
- drag force on piers and submerged superstructures
- wave forces
- wind effects
- bridge scour
- sediment transportation
- turbulence modelling.

These are typically in addition to one or two-dimensional hydraulic analysis to model the flow effects caused by bridge or culvert construction over a waterway. There is a wide variety of commercial and open-source software for hydrology and hydraulic analysis, with many examples listed in Table 3.3. The most commonly used hydraulic models used in Australia include HEC-RAS (both 1D and 2D) developed by the US Army Corps of Engineers (US Army Corps of Engineers n.d.), TUFLOW (2D), MIKEFLOOD (1D or 2D), SWMM (1D or 2D), SMS and SOBEK/DELFT.

Table 3.3: Common hydrology and hydraulic analysis models and their uses

Computer Model	Uses
ANSYS CFX/FLUENT (CAE Associates 2017)	Computation fluid dynamics software capable of modelling fluid flow and other related physical phenomena
AFFLUX (Main Roads Western Australia 2008)	Surface water analysis based on Manning's equation to compute backwater at a bridge structure
CULVERTMASTER (Bentley Systems 2017)	Culvert hydraulic analysis and design software
FLOW-3D (Flow Science 2017)	Computational fluid dynamics software for modelling physical flow processes
HEC-RAS (US Army Corps of Engineers n.d.)	Hydraulic analysis program capable of performing steady flow water surface profile computations, 1D and 2D unsteady flow simulation, movable boundary sediment transportation computations and water quality analysis
ICFD++ (Metacomp Technologies 2017)	Simulates compressible and incompressible fluids and flows, unsteady and steady flows, large range of speed regimes including low speeds through subsonic, transonic, supersonic and hypersonic speeds, laminar and turbulent flows, various equations of state
MIKEFLOOD (1D or 2D) (MIKE Powered by DHI n.d.)	Specialised 1D and 2D flood modelling software
OpenFOAM (ESI Group 2017)	Open source computation fluid dynamics software
RFFE (Australian Rainfall and Runoff n.d.)	Statistical model for estimating design floods for a given catchment based on gauge data in the catchment region
RORB[(1)] (Laurenson, Mein & Nathan 2012)	Hydrological analysis of a catchment area based on runoff routing concepts with the capacity to simulate reservoir storage
SMS (Aquaveo 2016)	Comprehensive surface water modelling system with modules for one and two-dimensional analysis of riverine and coastal systems, sediment transportation and other hydraulic analyses
SOBEK/DELFT (Deltares n.d.)	1D and 2D modelling software for irrigation and drainage systems, open channel hydraulics, river systems, sewerage and urban drainage systems, inundation and flooding scenario simulations, and the design and optimisation of control systems for canal and waterway automation
SWMM (1D or 2D) (Environmental Protection Agency 2017)	Used for planning, analysis and design related to stormwater runoff, combined and sanitary sewers, and other drainage systems in urban areas
TUFLOW (TUFLOW 2015)	1D and 2D modelling software to simulate the flood and tidal wave propagation
URBS[(1)] (URBS n.d)	Hydrological modelling program combining a rainfall runoff and runoff routing model to simulate catchment storage and runoff response
WBNM[(1)] (Boyd, Rigby & VanDrie 1996)	Advanced hydrological model for simulation catchment runoff which incorporates the ability to directly model the changes in flow associated with urbanisation and diversions of flow at structures
XP-RAFTS[(1)] (Xpsolutions 2017)	Simulation of runoff hydrographs at defined points throughout a watershed for a give set of catchment conditions and rainfall events

1 *Hydrology models.*

4. Hydraulic Design of Bridges

4.1 Introduction

This section is based on material published in *Hydraulic Design of Safe Bridges* (Zevenbergen et al. 2012).

The impacts of bridge design and construction on the economics of highway design, safety to the traveling public, and the natural environment can be significant. An economically viable and safe bridge is one that is properly sized, designed, constructed, and maintained. In general, although longer bridges are more expensive to design and build than shorter bridges, they cause less backwater, experience less scour, and can reduce impacts to the environment. Increased scour from too short a bridge can require deeper foundations and necessitate countermeasures to resist these effects. A properly designed bridge is one that balances the cost of the bridge with concerns of safety to the traveling public, impacts to the environment, and regulatory requirements to not cause harm to those that live or work in the floodplain upstream and downstream of the bridge.

The hydraulic analysis of a bridge opening is a complicated undertaking. Decisions must be made regarding the type of model computational methods, model extent, and amount of topographic data that needs to be collected. An assessment of flow resistance caused by channel and floodplain conditions needs to be made and the impacts on flow due to different seasonal conditions also needs to be evaluated. An understanding of flow type, historic flow conditions, and flooding at the site also provides valuable insight into the approaches that need to be employed.

Once the preliminary data has been collected and an understanding of the flow complexity at the bridge opening is obtained, a decision must be made regarding the type of hydraulic model that should be used at the hydraulic crossing. Some situations call for a one-dimensional gradually-varied steady-state flow model while others require the use of unsteady flow models, or two-dimensional steady or unsteady flow models to more fully understand the flow conditions at the hydraulic crossing. Some situations call for a more sophisticated modelling approach because of other factors. These can include the need for a more complete understanding of the flow conditions because of bridge scour or bank stabilisation.

There are also regulatory requirements that must be adhered to. The state and local road agencies, and others have requirements that must be considered when determining the best overall approach for evaluating the flow through a bridge opening and its impact on adjacent land owners, the environment and economic concerns. These types of issues must be considered when developing the best approach for analysing the flow through a bridge opening or reach of river.

4.2 Design Considerations

Hydraulic factors that should be considered in the design of bridges include:

- bridge opening and road grade
- bridge location selection
- scour and stream stability
- bridge design specifications and design criteria.

These factors are now briefly described.

4.2.1 Bridge Opening and Road Grade Design Considerations

The bridge waterway width is directly associated with the bridge length, from abutment to abutment. Hydraulic capacity should be a primary consideration in setting the bridge length. The bridge must provide enough capacity to:

- avoid excessive backwater in order to prevent adverse floodplain impacts
- prevent excessive velocity and shear stress within the bridge waterway.

In general, given a particular design discharge at a given crossing, the shorter a bridge the more backwater it will create. This same smaller bridge will also have higher velocities through the bridge opening and an increased potential for scour at the bridge foundation. A longer bridge at this same crossing will generate a smaller amount of backwater and will have lower velocities and potential for scour. Policy considerations and economics require an understanding of the impacts that the bridge could have on the flow of water in the floodplain and impacts it might have on adjacent properties (Zevenbergen et al. 2012).

Particularly, hydrologic and hydraulic studies must be conducted for each bridge site that includes:

- assessing the impact of afflux on adjacent land upstream and downstream of the bridge for floods up to the 100 year ARI (1% AEP)
- checking the stability of the adjacent road embankment for floods up to the 100 year ARI (1% AEP)
- assessing the impact of the ULS event (1 in 2000 year ARI or 0.05% AEP flood event) on the bridges, major drainage structures and major retaining walls
- assessing the effects of the road on regional flooding for the PMF event
- estimating the flow velocity scour depth for all flood events.

It is worth noting that, as part of the environmental considerations, the PMF event needs to be considered during a flood study. For some road agencies, however, the 2000 year ARI (0.05% AEP) is considered sufficient for the assessment of extreme events, therefore, it is not necessary to assess the road for the PMF event.

For bridges, the freeboard requirement is associated with a particular design recurrence-interval event, which is usually the 50- or 100-year event. Rural, low-traffic routes often allow a lower recurrence interval for establishing hydraulic capacity and freeboard.

The road profile can have a significant effect on bridge crossing hydraulics. Even if a bridge is designed to provide freeboard above a 100-year flood, the approach roadways may be overtopped by that same flood. When the overtopping occurs over a long segment of roadway, the associated weir flow is an important component of the overall hydraulic capacity of the crossing. In such a case, raising the road profile will have the potential to increase backwater unless additional capacity is provided in the bridge waterway to compensate for the lost roadway overtopping flow capacity.

The design of the piers and abutments has an effect on the bridge hydraulic capacity. Although this effect is small compared to the bridge length and road profile, it can still be important. For example, a bridge that crosses a regulatory floodway must be shown to cause no increase in backwater over existing conditions. In such a case the energy losses that are affected by the number of piers and their geometry can be significant. Spill-through abutments, set well back from the tops of the main channel banks, are advisable when bridge hydraulic capacity must be optimized.

Frequently the bridge waterway design includes subtle changes to the channel cross-section under the bridge and for a short distance upstream and downstream of the bridge. These changes are intended to enhance channel stability and, in some cases, to improve hydraulic efficiency. Channel stability can be enhanced, for instance, by grading the channel banks to side slopes of 2H:1V or flatter, and by providing channel bank revetment. Capacity can be improved by a moderate widening of the channel bottom in the immediate vicinity of the bridge, with appropriate width transitions upstream and downstream.

There are several potential bridge opening and road grade considerations that impact hydraulic capacity and upstream flood risk, especially when a road is upgraded and the bridge is replaced. These include bridge length, deck width, abutment configuration (spill through or vertical wall), number and size of piers, low chord elevation, freeboard, and road grade. If a crossing with a 25-year level of service is improved to a 50-year level of service, the road elevation may need to be increased. To avoid increased flood risk, the replacement bridge may need to be considerably longer and higher than the existing bridge. If there is inadequate freeboard, debris may collect along the deck and reduce flow conveyance.

4.2.2 Selection of Bridge Location

Where practicable, the alignment of the bridge should be chosen to avoid unstable sections of a watercourse channel, such as sharp or obviously mobile channel bends. When it is considered necessary to realign a waterway channel as part of a bridge design, the following issues and concerns should be investigated and appropriately addressed (Austroads 2013b):

- potential environmental impacts of sediment run-off from the construction of the new channel
- erosion potential of the downstream channel in response to the proposed realignment
- possible changes to existing bed conditions, including pool-riffle systems, within the channel
- the need for rock protection of the channel bed and banks given the potential to adversely affect the continuity and health of riparian vegetation and consequently the quality of the wildlife corridor
- any reduction in the length of the main channel and a consequential increase in the hydraulic gradient and erosion potential
- the form, condition and location of the low-flow channel; where practical, all of these should be maintained.

The location of the low-flow channel can have a significant effect on channel stability and aquatic habitat values. It can also meander within the bed of the main channel and the form, condition and location of the low-flow channel can vary from flood event to flood event.

The selection of bridge waterway openings also take into consideration the size and type of debris from upstream (Figure 4.1). Factors to be considered include:

- hydrodynamic forces without debris
- forces due to debris mats
- forces due to log impact
- urban debris, e.g. shipping containers and vehicles.

Where large logs, trees and urban debris can be anticipated, consideration shall be given to increasing both the span length and the freeboard to permit passage of debris.

Piers and abutments shall be designed to minimize their effects on water flows and avoid the trapping of debris where this is considered likely, as well as remain stable after the effects of scour. If piers must be located within the channel, and if a pool is likely to form within the channel at the bridge location, then the foundation design must allow for future bed erosion.

Abutment slopes and the underlying material shall be designed for stability and shall be protected against erosion effects for the design flood velocities.

Figure 4.1: Debris accumulated upstream of a bridge at Tumbarumba (NSW) after a flood

Source: Roads and Maritime (n.d.).

4.2.3 Scour and Stream Stability Consideration and Guidance

Another critical component of the design and/or evaluation of a bridge opening is to design the bridge to be stable from scour at the piers, abutments, and across the contracted opening. From a hydraulic perspective, the magnitude of local scour at a pier is a function of depth and velocity of flow, alignment of the pier with flow, and pier type and location (Zevenbergen et al. 2012). Depending of foundation costs and complexity it will be necessary to balance the number and size of piers, length and height, and anticipated total scour depth against increased costs of the superstructure associated with longer spans (girder type and allowable span) and foundation required to resist scour.

The magnitude of local scour at an abutment is a function of depth and velocity of flow, the skew of the embankment to the floodplain, as well as the amount of flow from the overbank that passes through the bridge opening. Highly-skewed bridges are not recommended as shown in Figure 4.2 where a large skew angle to the flow direction has worsened the scour effects. It is also a function of where the abutment is located in relation to the main channel. It is recommended that an abutment not be located in or close to the main channel if at all possible.

The amount of contraction scour that occurs at a bridge crossing is a function of the degree that a bridge contracts floodplain flow. In general, bridges with higher degrees of contraction can be expected to have higher flow velocities and larger scour depths. If the depths of contraction scour are too large it may be necessary to increase the bridge length to reduce scour across the bridge opening.

Bridges should be designed to withstand scour from large floods and from stream instabilities expected over the life of a bridge. Refer to Section 5 for the details.

Figure 4.2: Scour at a highly skewed bridge

Source: Roads and Maritime (n.d.).

4.2.4 Specifications and Design Criteria

The following bridge design requirements for hydraulic effects are specified in AS 5100.1 and AS 5100.2.

General requirements for bridge structures

- AS 5100.1 requires that the bridge must sustain the SLS flood event (Section 2.1.4) without damage to the bridge, channel bed, banks, and road or railway embankments, including the local effects of piers and abutments.
- Under the effects of any flood event up to and including the ULS (Section 2.1.5), together with the effects of debris and scour, the bridge must not collapse. The hydraulic capacity of the system must allow for the passage of the 2000 year ARI (0.05% AEP) flood without catastrophic effects or failures.
- The design shall allow for the impact of any stream excavation, improvement works or any altered flood patterns caused by the bridge and the approach embankment of the road or rail system.
- Consideration shall be taken of the corresponding scour at the relevant floods. Any scour protection, if provided for the SLS, shall not be relied upon at the ULS. In a circumstance that the SLS flood is also at 100 year ARI (1% AEP), the scour protection at piers can only be considered effective up to 50 year ARI (2% AEP).
- Abutments shall be adequately protected to prevent scour for floods up to the SLS.

Bridge waterway requirements

- The bridge waterway requirements shall be specified by the relevant authorities. The span and vertical clearances required for watercraft during normal stream flow or at specific flood levels shall be specified by the waterway authority.
- The soffit level of the bridge shall be not less than the flood immunity level or as specified by the relevant authority.
- The afflux limit (and corresponding ARI) shall be as specified by the relevant authority.
- Estimation of flood discharges, levels and velocities shall be based on methods appropriate to the locality.
- Theoretical estimates shall be compared with local flood records and the performance of any existing structures. Where the catchment area is intersected by a length of road or rail track containing bridges, culverts, embankments and floodways, the behaviour of this system shall be considered as a whole in deriving design flood levels and velocities.
- As critical design conditions may occur at flood levels that just cause overtopping of the superstructure, an estimate of the return interval of such a flood shall be made and, if appropriate, this condition shall be considered in the design.

Collision from waterway traffic

- Piers for bridges over navigable waterways shall be located to minimize the possibility of impact by waterway traffic. When collision from shipping is possible, the type of vessel, weight of vessel and speed of impact on the bridge shall be recommended by the harbour master, port authority or other relevant authority. This includes the channel and adjacent pier locations. The upper bound loads shall consider all vessels currently operating in the waterway or likely to operate in the waterway for the next 100 years. The minimum velocity of impact shall be the larger of the maximum flood velocity or the maximum speed of the vessel under power. The proposed design vessel and speed shall be reviewed and approved by the relevant authority.
- Unless a more advanced method of analysis is adopted, or unless otherwise specified by the relevant authority, the ultimate equivalent static vessel impact force shall be determined in accordance with AASHTO *LRFD Bridge Design Specifications*. The resulting minimum equivalent static ship impact force applicable to piers in navigable waterways shall be approved by the relevant authority.
- Piers in the waterway shall be designed for an equivalent static vessel impact force in the direction of the channel centre-line. The piers shall be designed to resist a load of 50% of the equivalent static vessel impact force applied separately in a direction perpendicular to the channel centre-line. These forces shall be applied anywhere between 1.0 m above mean low water spring (MLWS) and 1.0 m above mean high water spring (MHWS).
- The superstructure shall be designed to resist a horizontal force equal to 20% of the equivalent static vessel impact force applied independently of impact loads to the piers.

Forces resulting from water flow

Clause 16 of AS 5100.2 requires that bridges that cross a river, stream or any other body of water shall be designed to resist the effects of water flow and wave action, as applicable. The design shall take into account the impacts of the following forces resulting from water flow:

- forces on piers due to water flow
- forces on superstructures due to water flow
- forces due to debris
- forces due to moving objects.

Hydraulic forces on substructures include drag force on piers, which is a function of the shape of the upstream pier nose, plan view pier shape, skew as well as the presence of any debris matting. A side force is also specified by Clause 16.4.2 of AS 5100.2 to be applied simultaneously with the drag force, perpendicular to the plane containing the pier. Hydrostatic force (due to the weight of water) is also present, but is typically opposed by equivalent hydrostatic pressure the opposite side of the bridge. Any imbalance will be due to variation the upstream and downstream water surface elevation.

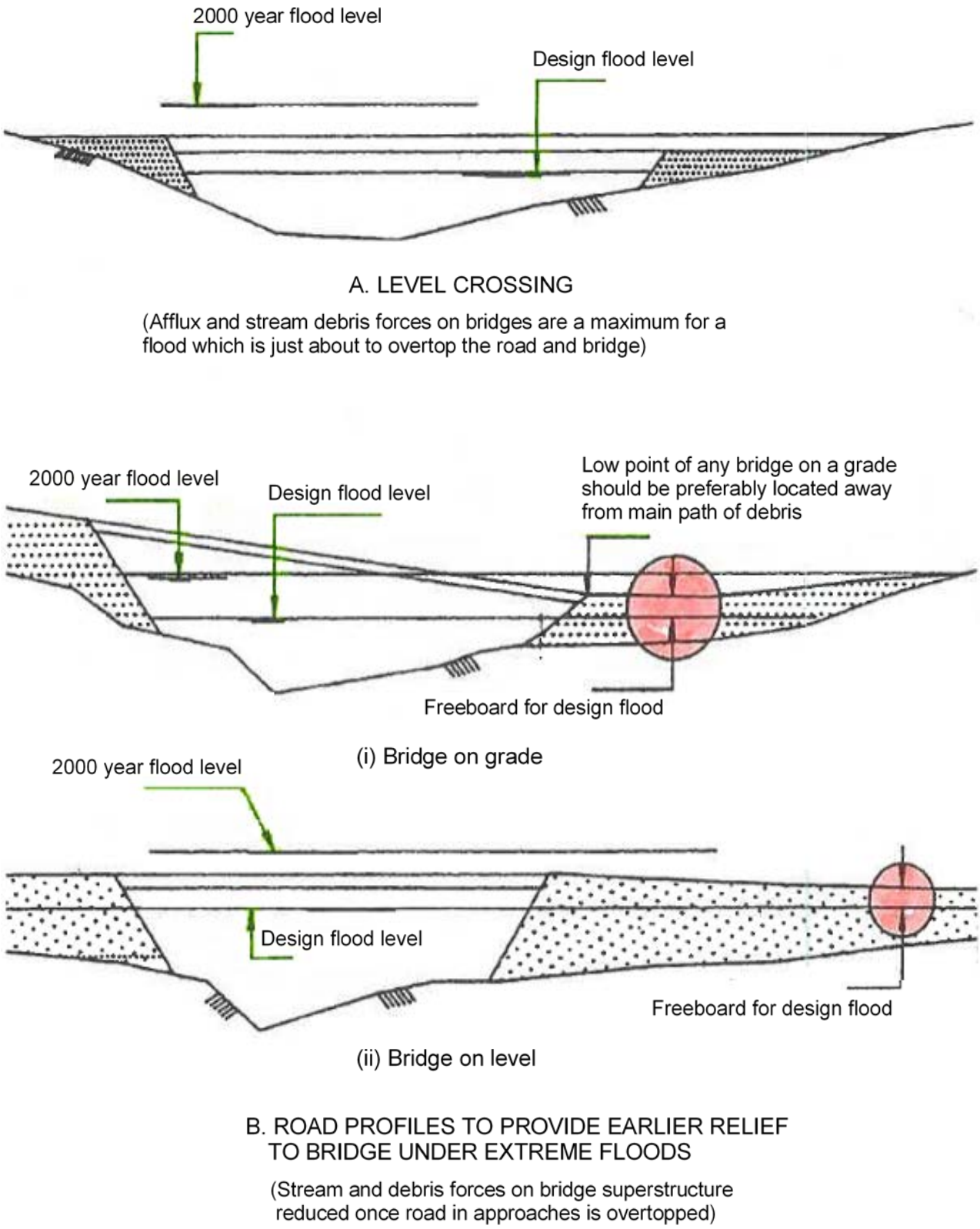
A bridge superstructure that is partially or fully submerged in a flood is subjected to the following forces resulting from the water flow (Clause 16.5 of AS 5100.2):

- a drag force normal to its longitudinal axis
- a vertical lift force (positive upwards)
- a moment about the girder soffit level (clockwise positive with the water flow from left to right).

Hydraulic forces on the bridge superstructure are dependent on the type of crossing as shown in Figure 4.3. Level crossing (A) would have maximum forces acting on the bridge superstructure when the flood is just about to overtop the bridge. Bridge or road approach B (i) and B (ii) on grade would have minimum hydraulic forces on the bridge superstructure. The crossing is provided with flood relief is usually provided on the road approach.

Refer to Clauses 16.6 and 16.7 of AS 5100.2, respectively for detailed provisions on the forces due to debris and forces due to moving objects.

Figure 4.3: Hydraulic forces in different types of crossing



Source: Roads and Maritime (n.d.).

4.3 Analysis Methods

There are a number of methods available for the hydraulic design of bridges. All hydraulic models, whether numerical or physical, rely on a set of assumptions and requirements in order to accurately simulate the flow condition at a structure. As no model will provide an exact representation of the complexity of the actual flow, it is important for engineers to understand these assumptions, as they form the limitations of that method. Ignoring or violating these assumptions and limitations or failing to critically analyse the model will produce inaccurate results. This is unacceptable given the high cost and potential consequences of failure of bridges.

The available modelling approaches are listed below (Zevenbergen et al. 2012):

- one-dimensional modelling: flow parameters change predominantly in one defined direction (x), along the channel
- two-dimensional modelling: flow parameters change in two directions (x and y)
  Horizontal velocity is computed as either components (V_x and V_y) or as a vector with constant magnitude and direction throughout the model.
- three-dimensional modelling (Computational Fluid Dynamics (CFD)): flow parameters change in three directions (x, y and z)
- physical modelling: geometrically scaled physical representations of bridges or bridge components.

Additionally, the decision should be made as to whether or not the flow condition should be modelled as steady or unsteady (i.e. changes with time). While almost all flow is unsteady to some extent, as the rate and depth of flow typically changes over long periods, the majority of bridge hydraulic analyses are conducted using steady flow.

There are several situations where unsteady flow conditions should be used, including:

- run-off from precipitation, or when depth and velocity of flow in a river change rapidly with time
- unsteady or transient flows released from reservoirs during operations for flood control, hydropower generation, recreation, and wildlife management
- where floodplain storage and/or the loss of floodplain storage are significant
- tidal applications
- dam break floods
- wind-generated storm surges or seiches
- landslide-generated waves
- earthquake-generated tsunami waves
- irrigation flows affected by gates, pumps, and diversions.

Unsteady flow analysis is discussed further in Section 4.6.

The appropriate analysis method for a structure should be selected primarily on the advantages and limitations of the method itself. However, the importance of the structures, project impacts, cost and schedule should also be considered. The most common approaches are one or two-dimensional modelling. While CFD and physical modelling can provide much higher levels of detail regarding flow patterns and hydrodynamic phenomena, their high cost and the lack of availability of computational and physical resources means they are typically used for localised situations, such as predicting local scour at piers, or modelling hydraulic forces on piers or the bridge superstructure. The decision to whether to use one or two-dimensional modelling should be based on the characteristics of the waterway being analysed. Table 4.1 provides a summary of the appropriateness of one or two-dimensional modelling to a wide variety of waterway and structural situations. Generally, one-dimensional modelling is appropriate for situations where lateral velocities are small, such as in-channel flow, minor floodplain flow, or where bridge constriction is minimal. The assumptions and limitations of one and two-dimensional modelling are further discussed in Section 4.4 and Section 4.5 respectively.

Table 4.1: Bridge hydraulic modelling selection

Bridge hydraulic condition	Hydraulic analysis method	
	One-dimensional	Two-dimensional
Small streams	●	◗
In-channel flows	●	◗
Narrow to moderate-width floodplains	●	◗
Wide flood plains	◗	●
Minor floodplain constriction	●	◗
Highly variable floodplain roughness	◗	●
Highly sinuous channels	◗	●
Multiple embankment openings	◗/○	●
Unmatched multiple openings in series	◗/○	●
Low skew roadway alignment (< 20º)	●	◗
Moderately skewed roadway alignment (>20º and < 30º)	◗	●
Highly skewed roadway alignment (> 30º)	○	●
Detailed analysis of bends, confluences and angle of attack	○	●
Multiple channels	◗	●
Small tidal streams and rivers	●	◗
Large tidal waterways and wind-influenced conditions	○	●
Detailed flow distribution at bridges	◗	●
Significant roadway overtopping	◗	●
Upstream controls	○	●
Countermeasure design	◗	●

Key:

● *well suited or primary use*

◗ *possible application or secondary use*

○ *unsuitable or rarely used*

◗/○ *possibly unsuitable depending on application*

Source: Zevenbergen et al. (2012).

4.4 One-dimensional Bridge Hydraulic Analysis

One-dimensional analysis is a term covering a wide range of analysis methods with differing levels of details, from a single waterway section to a more detailed water surface profile involving multiple cross-sections and stream reaches. They typically incorporate the assumption of uniform flow, in which flow parameters change predominantly in the x-direction, along the centre line of the channel. An approximate method for the calculation of backwater is the FHWA's HDS 1 method (Zevenbergen et al. 2012).

The HDS 1 model for calculating backwater is as follows (Equation 5):

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} + \alpha_1 [\left(\frac{A_{n2}}{A_4}\right) - (\frac{A_{n2}}{A_1})] \frac{V_{n2}^2}{2g} \qquad 5$$

where

h_1^*	=	total backwater, m
K^*	=	total backwater coefficient
α_1, α_2	=	kinetic energy distribution coefficients at cross-sections 1 and 2
A_{n2}	=	gross water area in constricted bridge waterway measured below normal stage at cross-section 2 (m^2)
V_{n2}	=	average velocity in constriction (total discharge divided by A_{n2}) (m/s)
A_1	=	total flow area at cross-section 1, including addition caused by backwater (m^2)
A_4	=	total flow area at cross-section 4, downstream of influence of bridge (m^2)
g	=	acceleration of gravity(m/s^2)

The locations of the cross-sections are shown in Figure 4.4.

The location of the cross-section is critical to ensuring accurate results. Any redistribution of flow caused by changing conveyance (area, depth or roughness) should be reasonably possible in the distance between cross-sections. Placing cross-sections too close together will create physically impossible results, while placing them too far apart will result in numerically unreliable answers due to the significant differences in energy slopes. Additionally, the cross-sections should be selected and orientated such that there is a consistent water surface for the simulated flow, otherwise the numerical solution to the model may differ significantly from reality.

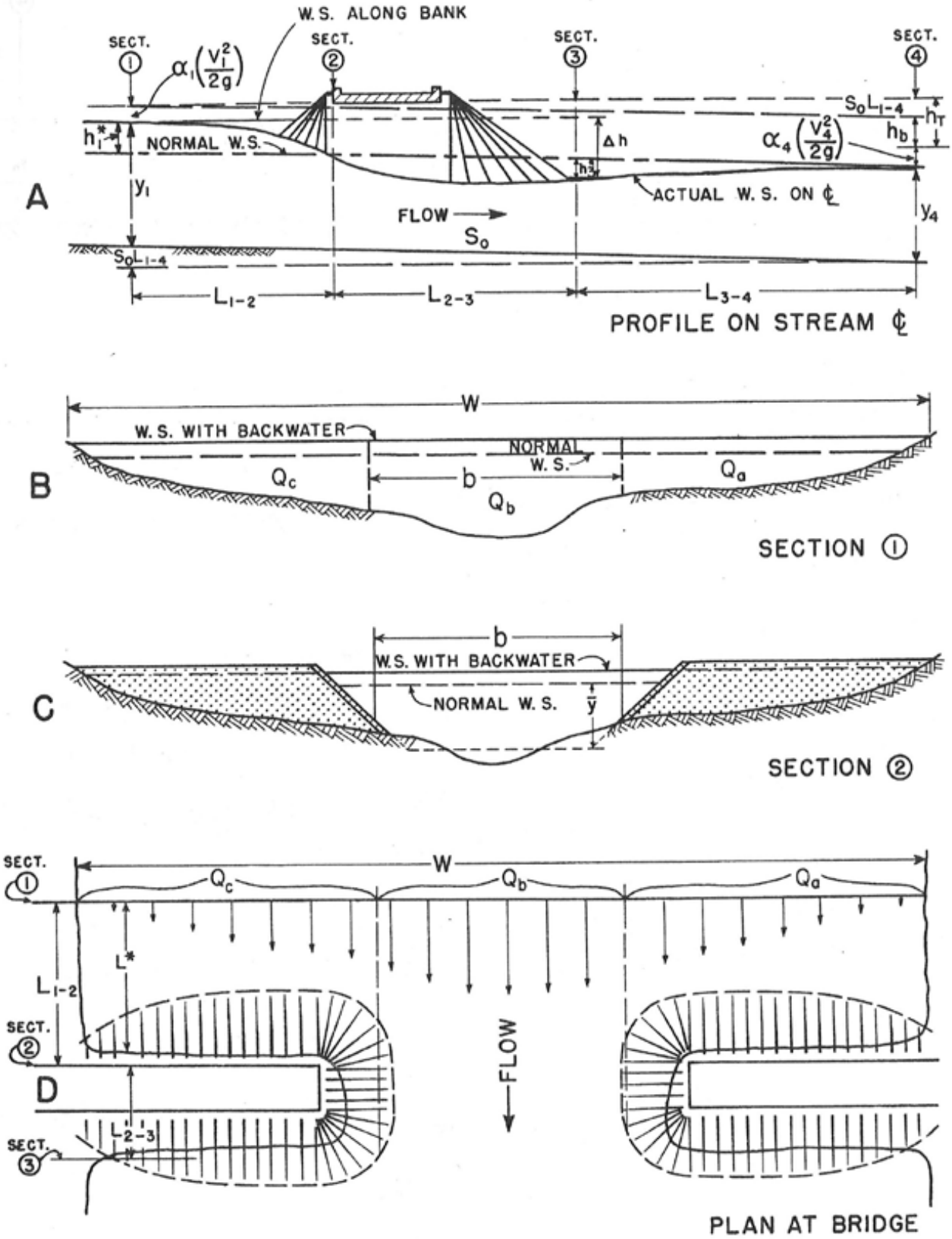
4.4.1 Limitations

As noted in Table 4.1, there are several situations where one-dimensional analysis is inappropriate. This is due to the assumption of steady flow in one direction, along the centre line of the channel. The conditions of this flow are as follows:

- a single water surface at each cross-section
- the flow is perpendicular to the cross-section along its entire length
- the energy slope for the cross-section applies to every point in the cross-section
- hydrostatic pressure exists throughout the cross-section
- channel slope is small
- energy slope is the same as for the corresponding normal depth
- the channel is prismatic with constant alignment and shape
- roughness is constant through the reach.

As flow in natural channels is inherently three-dimensional and unsteady, taking these assumptions as absolute would preclude one-dimensional analysis in most cases. However, if the engineer is aware of these limitations and applies the model appropriately, then the accuracy of the results should not be overly compromised.

Figure 4.4: Sketch illustrating positions of cross-sections 1 through 4 in HDS 1 backwater method



Source: Zevenbergen et al. (2012).

4.4.2 Special Cases

There are several situations where different approaches to one-dimensional modelling may be required in order to accurately model special conditions that may exist at a structure. These may include:

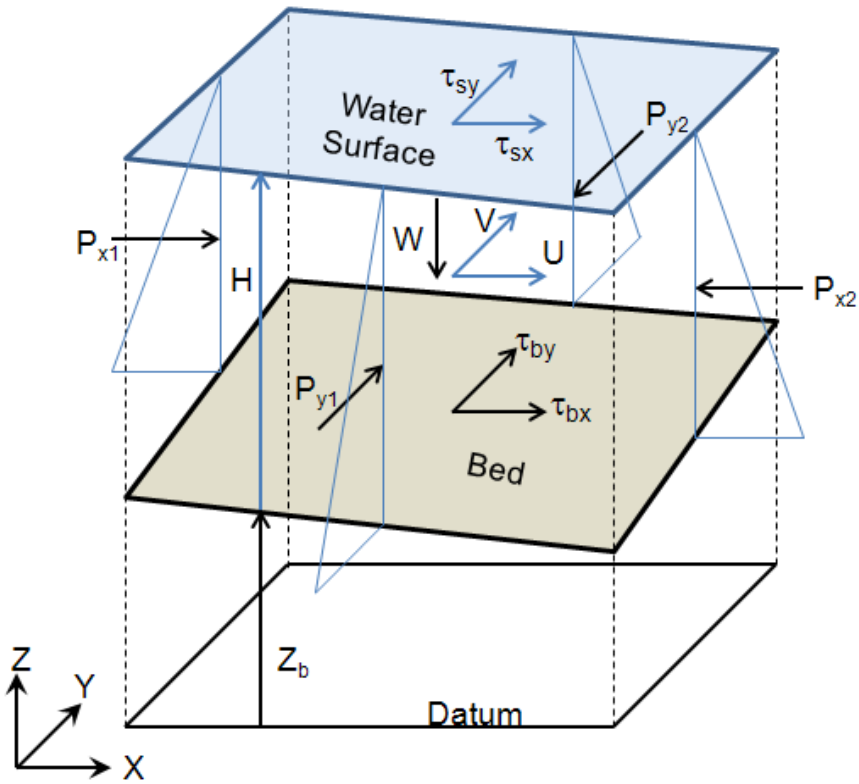
- skewed crossings
- crossings with parallel bridges
- split flow conditions
- crossings with multiple openings in the embankment
- lateral weirs.

Approaches for these situations are explained in further detail in Zevenbergen et al. (2012).

4.5 Two-dimensional Bridge Hydraulic Analysis

Two-dimensional hydraulic analysis is a more rigorous form of analysis that relies on less assumptions than one-dimensional analysis and can more accurately model complex situations. While one-dimensional modelling typically relies on the solution of the energy equation, in two-dimensional modelling, the momentum and continuity equations are applied to a control volume, as illustrated in Figure 4.5 (refer to Zevenbergen et al. 2012 for details). As opposed to one-dimensional modelling, velocities in both the x and y directions are considered; vertical velocities are considered negligible and hydrostatic pressure is assumed. Velocities may be expressed as a vector quantity (with magnitude and direction) as separate x and y components. Additionally, the elevation of the bed (Z_b) and water (H) depth vary over the area. The force variables are pressure (P) at the horizontal surfaces, water weight (W), bed and water surface shear stress components (τ_b and τ_s).

Figure 4.5: Control volume and two-dimensional hydraulic analysis variables



Source: Zevenbergen et al. (2012).

These equations are applied in two main model types, the finite element method and finite difference method. The finite element method uses an unstructured mesh or grid to solve the continuity and momentum equations through numerical integration techniques at each element. The finite difference method is a numerical solution technique for differential equations. Both these methods have the advantage of being able to model flow conditions more accurately than one-dimensional analysis through techniques such as decreasing mesh/grid size in order to provide greater detail on changes in terrain (land use, roughness, topography, or bathymetry) or velocity, or by removing/deactivating elements/grid cells to represent road embankments or piers.

4.5.1 Limitations

While two-dimensional hydraulic analysis is able to account for many of the limitations faced by one-dimensional analysis, there are several situations where more three-dimensional or CFD analysis may be required to model the complete flow field. This is this case when analysis pier, abutment or debris obstruction losses, and road overtopping, due to the fact that vertical velocities are not included in two-dimensional models, and hydrostatic pressure is assumed. Techniques for analysing these situations in two-dimensional models are noted in Zevenbergen et al. (2012). Additionally, while two-dimensional modelling can account for most processes associated with the pressure flow caused in submerged bridge decks, it is unable to account for flow separation at the leading edge of the deck.

4.5.2 Special Cases

Section 4.4 includes special cases of one-dimensional modelling that fall outside the typical model application. These include skewed crossings, parallel crossings, multiple openings and other less common applications. Most of these situations are not considered as special applications in two-dimensional models because the assumptions required by one-dimensional models are not required in two-dimensional models. Refer to Zevenbergen et al. (2012) for the guidance on the use of two-dimensional models for some of these cases and compares and contrasts the use of one- and two-dimensional models for them.

4.6 Unsteady Flow Analysis

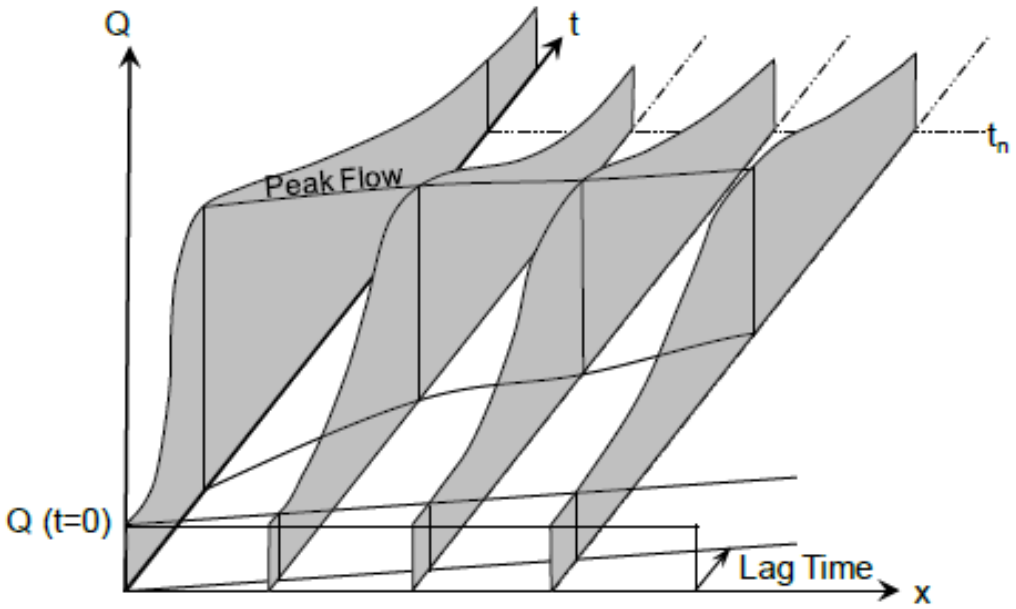
Unsteady flow is defined as flow that changes with time. While in actuality, almost all flow can be considered to be unsteady to some extent, for many applications in bridge hydraulics the flow can be assumed to be steady for a short reach of the channel. Several situations where unsteady analysis is important were presented in Section 4.3. There are several important differences between steady and unsteady flow analysis (Zevenbergen et al. 2012):

- Steady flow analysis assumes flow can vary with respect to space, but not time ($\partial Q/\partial x \neq 0$, $\partial Q/\partial t = 0$); unsteady flow assumes it can vary in time and space.
- For small bed slopes (i.e. slopes less than 0.0004) or highly transient flows, such as tidal influences or dam breach flood waves, the peak stages do not necessarily coincide with the peak discharges, and the rating curves of stage versus discharge are not single-valued.
- The total flow downstream from a junction of two tributaries is not necessarily the combination of the two flows.
- Tributary flows entering a main stream channel may experience a flow reversal caused by flow in the main stem backing up into the tributary or vice versa, e.g. when a large tributary flood enters the main channel during a period of low flow.
- If the inflow or stage at a boundary is changing rapidly, the acceleration terms in the momentum equation are important, and thus unsteady flow is a more robust solution.
- For full networks, where the flow divides and recombines, unsteady flow analysis should be used. This is due to the fact that the channel length, resistance and geometry will likely differ, causing the flow to travel at different speeds, affecting flow distribution.

Unsteady flow is typically analysed for one-dimensional and two-dimensional applications.

One-dimensional unsteady flow analysis is represented by the Saint-Venant equations outlined in Zevenbergen et al. (2012). These equations may be expressed in a three-dimensional space, with two axes corresponding to the distance along the channel (x) and time (t), along with the solution being sought such as discharge, depth, water surface elevation or velocity. This is illustrated in Figure 4.6, which also demonstrates features of unsteady analysis such as lag time and attenuation of flow.

Figure 4.6: Unsteady flow analysis solution of discharge versus distance and time



Source: Zevenbergen et al. (2012).

4.6.1 Limitations

The model limits of an unsteady model must be carefully chosen to include all potential storage upstream and downstream of the location of interest. If a bridge hydraulic model including only the minimum number of cross-sections were used as the unsteady model geometry, the simulation would be inaccurate because storage and routing effects would be significantly under represented. Therefore, unsteady models almost always require much longer upstream and downstream limits than steady models.

4.6.2 Special Cases

As two-dimensional steady flow analysis is used in more complex situations than one-dimensional analysis, the same is true for unsteady flow analysis. This due the ability for the equations to include additional terms such as wind stress that may occur during hurricane storm surges. Other situations where two-dimensional unsteady flow analysis is appropriate include:

- water levels and flow distributions around islands
- flow at bridges having one or more relief openings
- in extremely contracting and expanding reaches
- into and out of off-channel storage or flow situations such as overtopping of a levee
- flow at river junctions
- circulation and transport in water bodies with wetlands
- water surface elevations and flow patterns in large rivers, reservoirs and estuaries.

Further details of the applications of two-dimensional unsteady flow analysis can be found in Zevenbergen et al. (2012).

4.7 Design Procedures

The following procedure is recommended for hydraulic design of bridges in determining a bridge waterway (length and deck level):

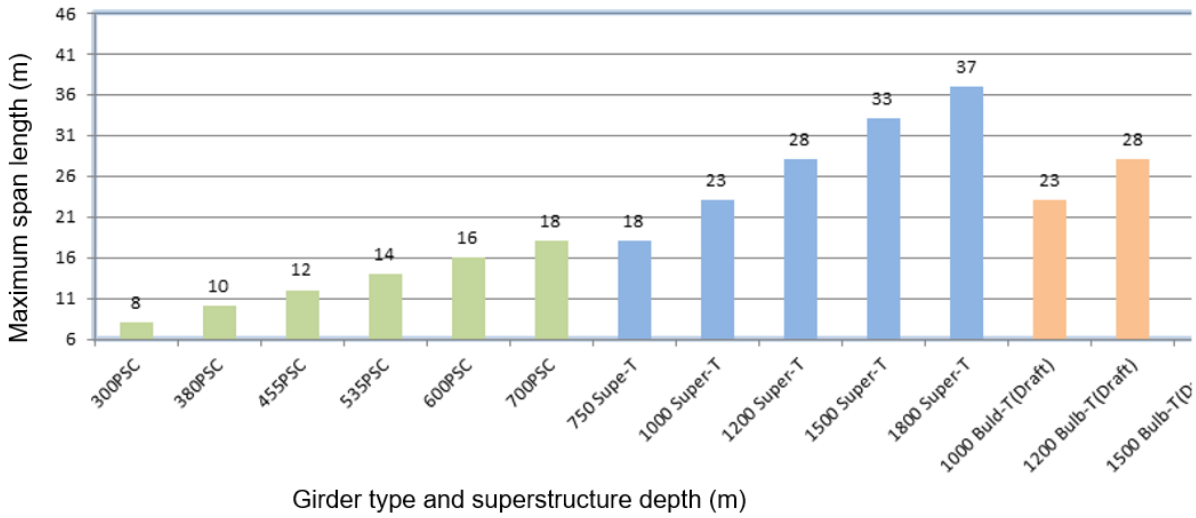
1. determine magnitude of flow at site for design recurrence interval
2. determine stage-discharge curve for the stream at the bridge site
3. determine the stage height at the bridge site for the design discharge from the stage-discharge curve (step 2)
4. select velocity of flow through bridge opening to limit scour or encourage scour as required
5. determine minimum length of bridge opening required to pass design discharge assuming water surface is at stage height
6. select a bridge deck level and trial length of bridge based upon minimum length of bridge opening and required length of spans
7. determine type of flow encountered
8. calculate backwater using the procedure relevant to the type of flow encountered
9. check the assumed deck level using the stage height and backwater for the design discharge based on the trial length of bridge.

If there is insufficient clearance beneath the bridge lift the bridge deck level and if necessary recalculate the backwater. If there is insufficient freeboard to the top of the embankment, lift the embankment or reduce the backwater by using a larger bridge.

It is worth noting that checking for afflux is required where flooding upstream is very sensitive to development, especially in urban catchments. In flood-sensitive areas, zero or negligible afflux is required and as a result, a longer bridge matching the flood width may be required.

The selection of the type of superstructure could affect the level of service and freeboard for the crossing. The depth of superstructure may vary from 0.25 m to more than 1.8 m, not taking into account the cross fall and the deck slab. The typical depth of superstructure for the span range 5–40 m is shown in Figure 4.7.

Figure 4.7: Typical depth of bridge superstructure



Source: Roads and Maritime (n.d.).

4.8 Special Considerations

There are several other considerations that should be taken into consideration alongside typical bridge hydraulic analysis, which are discussed in Section 4.8.1 to Section 4.8.2.

4.8.1 Backwater Effects of Bridge Piers

Hydraulic drag at bridge piers is a force that must be resisted by the structure. For the stream, it is a resistance to flow that must be overcome by an increase in energy driving flow through the bridge waterway. This typically takes the form of backwater. While the total backwater upstream of the bridge is generally dominated by the severe constriction caused by road embankments and bridge abutments, piers can be a significant factor in bridge design (Zevenbergen et al. 2012). As bridge piers are required for most structures in order to maintain economical span length and superstructure depths, the hydraulic analysis should inform the structural design in order to minimise the impact of piers on the waterway, in line with Clause 11.5 of AS 5100.1.

4.8.2 Coincident Flows at Confluences

Where a bridge over a waterway is located near a confluence with another stream, the potential influence of the other waterway on the hydraulics at the bridge should be considered. If the structure is upstream of the confluence it should be considered how the other waterway will affect the water surface profile through the bridge at various flood levels. If the bridge is located within or near the confluence zone, how the interaction with will affect the distribution and direction of flow through the confluence should also be considered (Zevenbergen et al. 2012). In order to consider the effects of the confluence, it is necessary to estimate the coincident flow probabilities of the waterways. That is, while it is unlikely that a 100 year flood level will occur in both simultaneously, the possible combinations of flows that have a 100 year recurrence level should be determined. Overestimating these probabilities will lead to a conservative design, while underestimating may have negative consequences for the structure.

4.9 Computation of Backwater

This section presents a method for computing the backwater caused by bridge constrictions based on the work of Bradley (1978).

The expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge, section 1, and a point downstream from the bridge at which normal stage has been re-established, section 4 (Figure 4.4). The expression is reasonably valid if the channel in the vicinity is essentially straight, the cross-sectional area of the stream is fairly uniform, the gradient of the bed is approximately constant between sections 1 and 4, the flow is free to contract and expand, there is no appreciable erosion of the bed in the constriction due to scour, and the flow is in the subcritical range.

The expression for computation of backwater upstream from a bridge constriction is in Equation 6:

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} + \alpha_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \qquad 6$$

where

h_1^* = total backwater (m)

K^* = total backwater coefficient

α_1 and α_2 = Kinetic energy distribution coefficients at cross-sections 1 and 2 (Figure 4.4)

A_{n2} = gross water area (m^2) in constriction measured below normal stage

V_{n2} = average velocity (m/s) in constriction or Q/A_{n2}

A_4 = water area (m^2) at section 4 (Figure 4.4) where normal stage is re-established

A_1 = total water area (m^2) at section 1 (Figure 4.4), including that produced by the backwater

g = acceleration of gravity, (m/s^2)

To compute backwater from Equation 6 it is first necessary to obtain the approximate value of h_1^* by using the first part of the expression (Equation 7):

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} \qquad 7$$

The value of A_1 in the second part of the expression which depends on h_1^* can then be determined and the second term of the expression evaluated (Equation 8):

$$\alpha_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \qquad 8$$

This part of the expression represents the difference in kinetic energy between sections 4 and 1 (Figure 4.4), expressed in terms of the velocity head $V_{n2}^2/2g$. The equation may appear cumbersome, but it has been set up as shown to permit omission of the second part when the difference in kinetic energy between sections 4 and 1 (Figure 4.4) is small enough to be insignificant in the final result. To permit the design engineer to readily recognise cases in which the kinetic energy term may be ignored, the following guides are provided:

- the bridge opening ratio (ratio of the bridge opening width to the total floodplain width), *M*, is greater than 0.7
- V_{n2} is less than 2 m/s, and
- $K^* V_{n2}^2/2g$ is less than 0.15 m.

The backwater obtained from the first term of Equation 6 can be considered sufficiently accurate, if values in the problem at hand meet all three conditions. Should one or more of the values not meet the above conditions, it is advisable to use Equation 6 in its entirety.

The following definitions are worth noting:

Conveyance – is a measure of the ability of a channel to transport flow. Using the Manning Equation (Equation 4) for open channel flow, the discharge, *q* in a subsection of a channel can be determined. Equation 4 can be re-arranged as (Equation 9):

$$\frac{q}{{S_o}^{1/2}} = \frac{ar^{2/3}}{n} = k \qquad 9$$

where

k = conveyance of subsection

Conveyance can, therefore, be expressed either in terms of flow factors or strictly geometric factors. In bridge waterway computations, conveyance as a means of approximating the distribution of flow in the natural river channel upstream from a bridge. Total conveyance K_1 is the summation of the conveyances of the subsections.

Bridge opening ratio (*M*) – defines the degree of stream constriction involved. It is defined as the ratio of the flow which can pass unimpeded through the bridge constriction to the total flow of the river (Equation 10):

$$M = \frac{Q_b}{Q_a + Q_b + Q_c} = \frac{Q_b}{Q} \qquad 10$$

or for the example shown on Figure 5.4,

$$M = 210/350 = 0.6$$

Because of the irregular cross-section common in natural streams and the variation in boundary roughness within any cross-section, the discharge is not uniform across a river but varies as might be indicated by the stream lines in Figure 5.4. The bridge opening ratio, M is most easily explained in terms of discharges, but it is usually determined from conveyance relationships. Since conveyance is proportional to discharge, assuming all subsections to have the same slope M can be expressed also as (Equation 11):

$$M = \frac{K_b}{K_a + K_b + K_c} = \frac{K_b}{K_1} \qquad 11$$

Kinetic energy coefficient – as the velocity distribution in a river varies from a maximum at the deeper portion of the channel to essentially zero along the banks, the average velocity head computed as $(Q/A_1)^2/2g$ for the stream at section 1 (Figure 5.4), does not give a true measure of the kinetic energy of the flow. A weighted average value of the kinetic energy is obtained by multiplying the average velocity head, above, by a kinetic energy coefficient, α_1, defined as (Equation 12):

$$\alpha_1 = \frac{\sum(qv^2)}{Q{V_1}^2} \qquad 12$$

where

v = average velocity (m/s) in a subsection

q = discharge (m^3/s) in same subsection

Q = total discharge (m^3/s) in river

V_1 = average velocity (m/s) in river at section 1 (Figure 5.4) or Q/A_1

A second coefficient α_2 is required to correct the velocity head for non-uniform velocity distribution under the bridge (Equation 13):

$$\alpha_2 = \frac{\sum(qv^2)}{Q{V_2}^2} \qquad 13$$

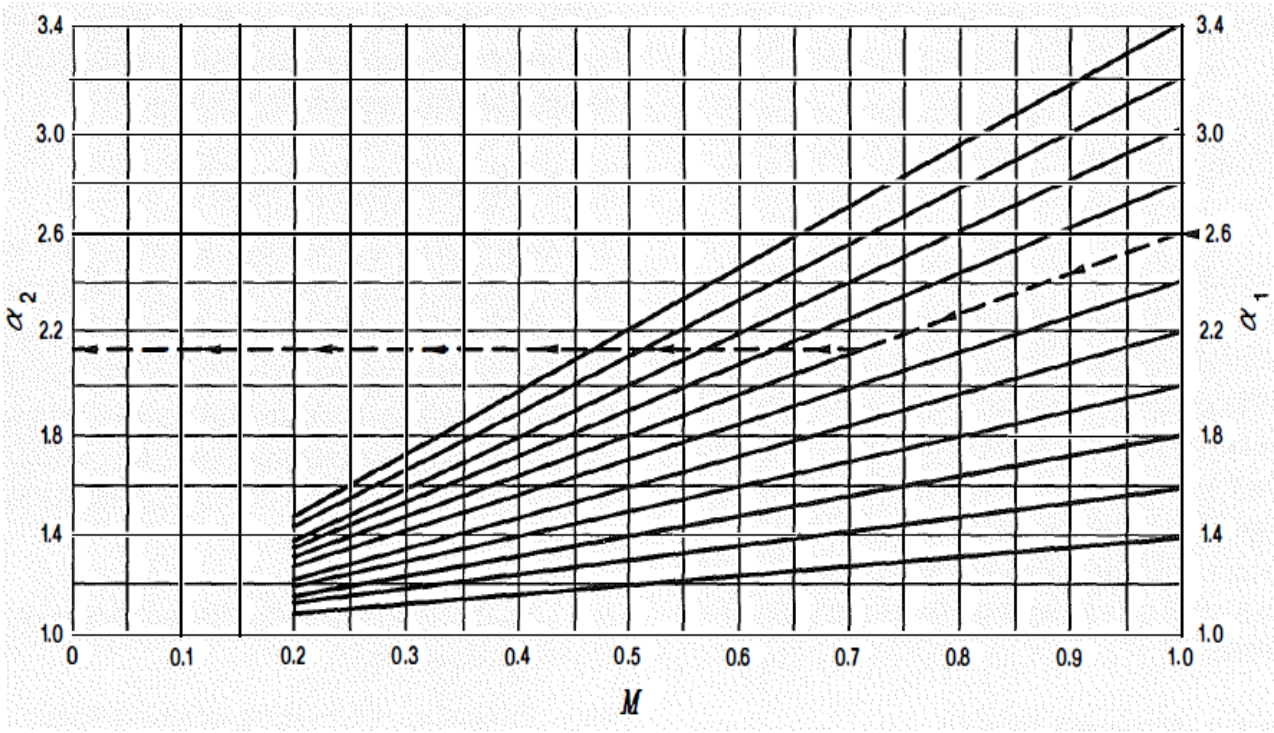
where

V_2 = average velocity in constriction, = Q/A_2

In this equation, v, q, and Q are defined as in Equation 12, but apply here to the constricted cross-section.

The value of α_1 can be computed, but α_2 is not readily assessed. The best that can be done in the case of the latter is to collect, tabulate and compare values of α_2 from existing bridges. Figure 4.8 relating α_2 to α_1 and the contraction ratio, M, is included for estimating purposes only. The value of α_2 is usually less than α_1 for a given crossing, but this is not always the case. Actually, there should be no definite relationship between the two, but there is a trend. Local factors at the bridge such as asymmetry of flow, variation in cross-section and extent of vegetation in the bridge opening will influence the value of α_2. It is suggested that values adopted for α_2 should err on the high side.

Figure 4.8: Aid for estimating α_2

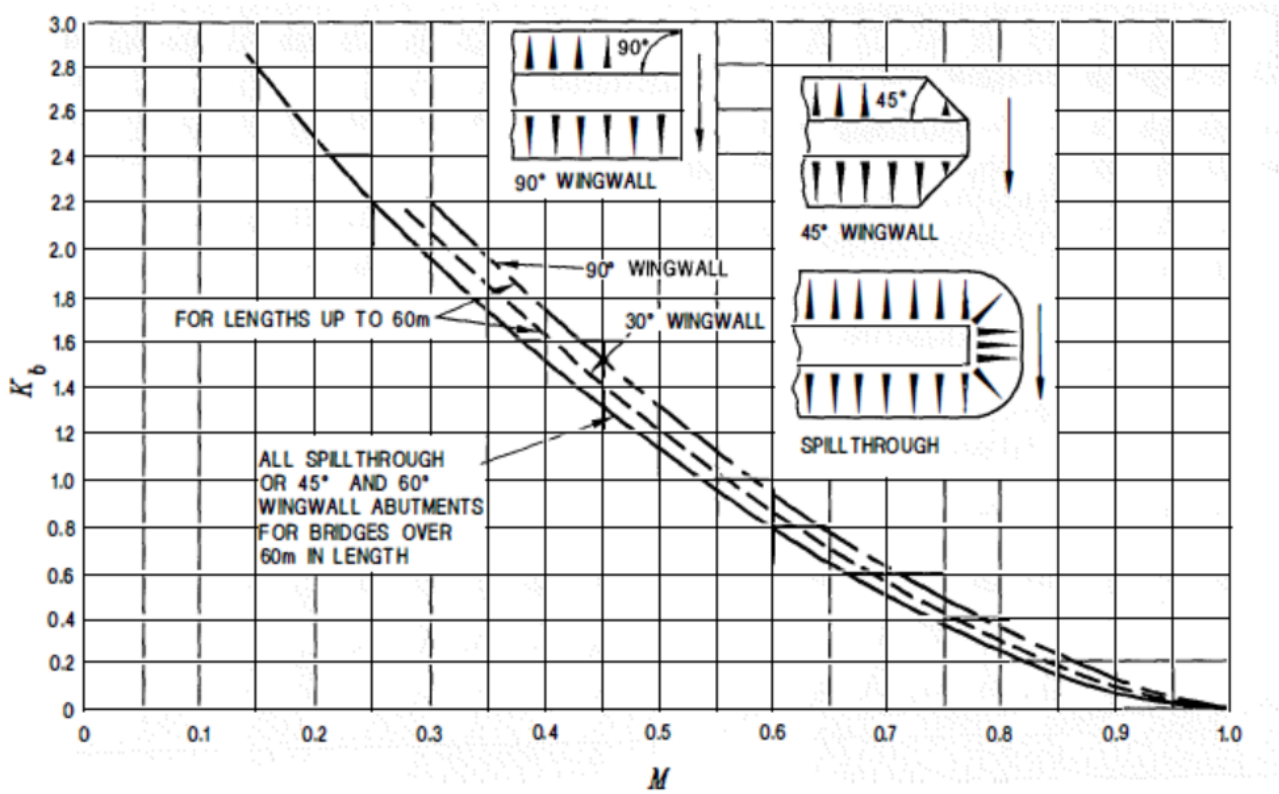


4.9.1 Backwater Coefficient

Two symbols are interchangeably used throughout this section and both are backwater coefficients. The symbol K_b is the backwater coefficient for a bridge in which only the bridge opening ratio, *M*, is considered. This is known as a base coefficient and the curves on Figure 4.9 are called base curves. The value of the overall backwater coefficient, K^*, is similarly dependent on the value of *M*, but also affected by:

- number, size, shape and orientation of piers in the constriction
- eccentricity, or asymmetric location of bridge with respect to floodplains
- skew (bridge crosses floodplain at other than 90° angle).

Figure 4.9: Backwater coefficient base curves (subcritical flow)



It will be demonstrated in the succeeding sections that the overall backwater coefficient K^* consists of a base curve coefficient K_b, to which are added incremental coefficients to account for the effect of piers, eccentricity and skew. The value of K^* is nevertheless primarily dependent on the degree of constriction of the flow at a bridge.

4.9.2 Effect of M and Abutment Shape (Base Curves)

Figure 4.9 shows the base curve backwater coefficient, K_b, plotted with respect to *M*, for wingwall and spillthrough abutments. Note how the coefficient, K_b, increases with channel wingwall constriction. The lower curve applies for 45° and 60° wingwall abutments and all spill-through types. Curves are also included for 30° wingwall abutments and for 90° vertical wall abutments for bridges up to about 60 m in length. For bridges over 60 m in length, regardless of abutment type, the lower curve is recommended because abutment geometry becomes less important to backwater as bridge length increases.

4.9.3 Effects of Piers (Normal Crossings)

Backwater caused by the introduction of piers in a bridge constriction is treated as an incremental backwater coefficient designated ΔK_p, which is added to the base curve coefficient. The value of the incremental backwater coefficient, ΔK_p, is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening, the type of piers (or piling in the case of pile bents), the value of the bridge opening ratio, *M*, and the skew of the piers to the direction of flood flow. The ratio of the water area occupied by piers A_p, to the gross water area of the constriction, A_{n2}, both based on the normal water surface, is assigned the letter J. In computing the gross water area, A_{n2} , the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from, Figure 4.10. The procedure is to enter chart A on Figure 4.10 with the proper value J and read ΔK, and then obtain the correction factor, σ, from chart B for opening ratios other than unity. The incremental backwater coefficient is then shown in Equation 14:

$$\Delta K_p = \sigma \Delta K \qquad 14$$

The incremental backwater coefficients for pile bents can, for all practical purposes, be considered independent of diameter, width, or spacing of piles, but should be increased if there are more than five piles in a bent. A bent with 10 piles should be given a value of K_p about 20 per cent higher than those shown for bents with five piles. If there is a possibility of debris collecting on the piers, it is advisable to use a larger value of J to compensate for the added obstruction.

For a normal crossing with piers, the total backwater coefficient becomes (Equation 15):

$$k^* = k_b(\text{Figure 4.9}) + \Delta K_p(\text{Figure 4.10}) \qquad 15$$

4.9.4 Effects of Piers (Skew Crossings)

In the case of skew crossings, the effect of piers is treated as explained for normal crossings (Section 4.9.3) except for the computation of J, A_{n2} and *M*. The pier area for a skew crossing, A_p, is the sum of the individual pier areas normal to the general direction of flow, as illustrated by the sketch on Figure 4.10. Note how the width of pier, W_p, is measured when the pier in not parallel to the general direction of flow. The area of the constriction, A_{n2}, for skew crossings, is based on the projected length of bridge, $b_s \cos \emptyset$ (Figure 4.10). Again, A_{n2} is a gross value and includes the area occupied by piers. The value of J is the pier area, A_p, divided by the projected gross area of the bridge constriction, both measured normal to the general direction of flow. The computation of *M* for skew crossings is also based on the projected length of bridge, which will be further explained in Section 4.9.6.

4.9.5 Effect of Eccentricity

Referring to the sketch on Figure 4.11, it can be seen that the symbols Q_a and Q_c at section 1 are used to represent the portion of the discharge obstructed by the approach embankments. If the cross-section is very asymmetrical, so that Q_a is less than 20 per cent of Q_c, or vice versa, the backwater coefficient will be somewhat larger than for comparable values of M shown on the base curves. The magnitude of the incremental backwater coefficient, ΔK_e accounting for the effect of eccentricity, is shown on Figure 4.11. Eccentricity, *e*, is defined as 1 minus the ratio of the lesser to the greater discharge outside the projected length of the bridge, or as shown in Equation 16:

$$e = (1 - Q_c/Q_a) \text{ with } Q_c < Q_a \qquad 16$$

or Equation 17:

$$e = (1 - Q_a/Q_c) \text{ with } Q_c > Q_a \qquad 17$$

Reference to the sketch on Figure 4.11 will aid in clarifying the terminology for instance, if Q_c/Q_a = 0.05, the eccentricity, *e* = (1 – 0.05) or 0.95 and the curve for 0.95 on Figure 4.11 would be used for obtaining ΔK_e. The largest influence on the backwater coefficient due to eccentricity will occur when a bridge is located adjacent to a bluff where a floodplain exists on only one side and the eccentricity is 1.0. The overall backwater coefficient for a very eccentric crossing with wingwall abutments and piers will be Equation 18:

$$K^* = K_b(\text{Figure 4.9}) + \Delta K_p(\text{Figure 4.10}) + \Delta K_e(\text{Figure 4.11}) \qquad 18$$

Figure 4.10: Incremental backwater coefficient for piers

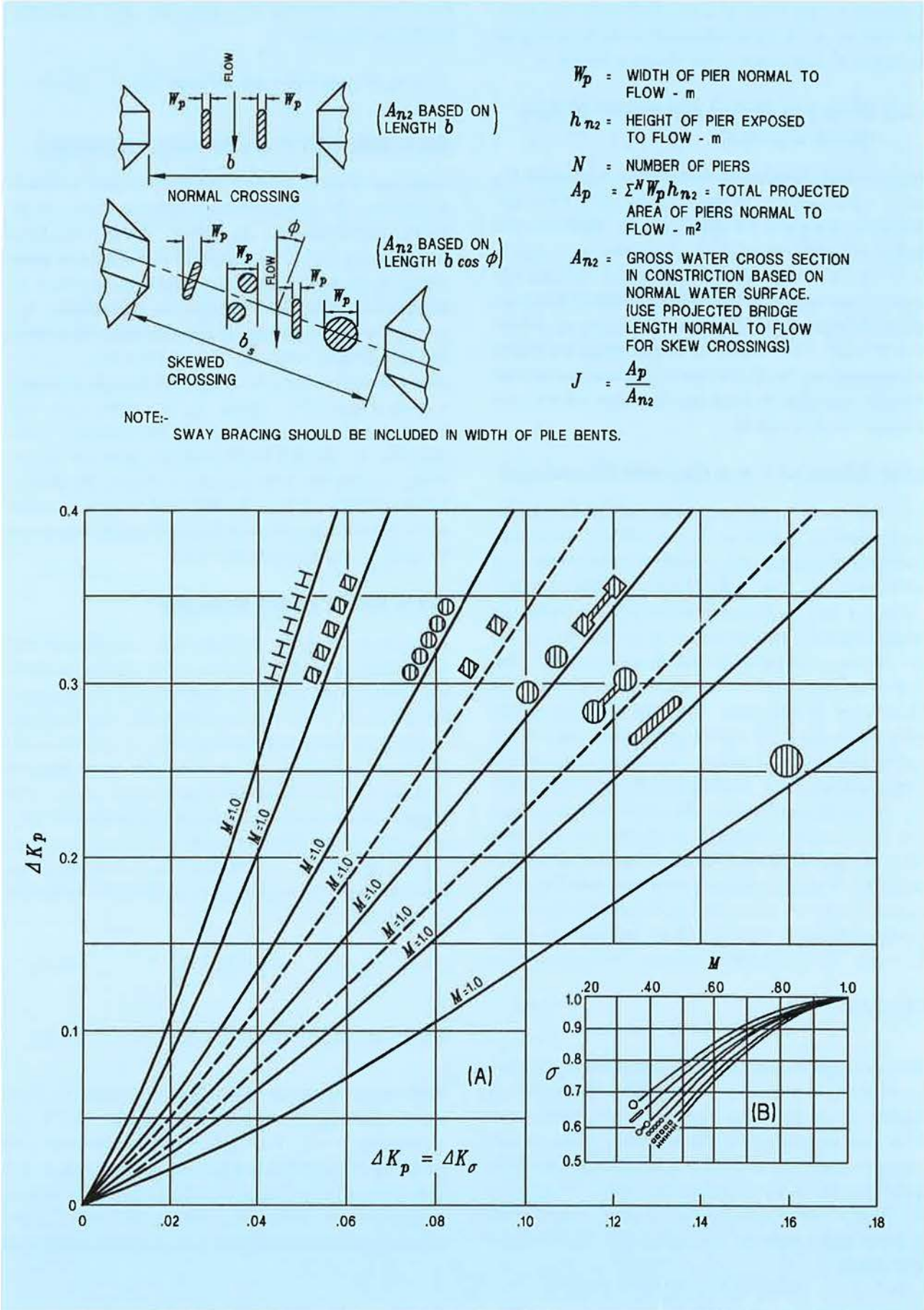
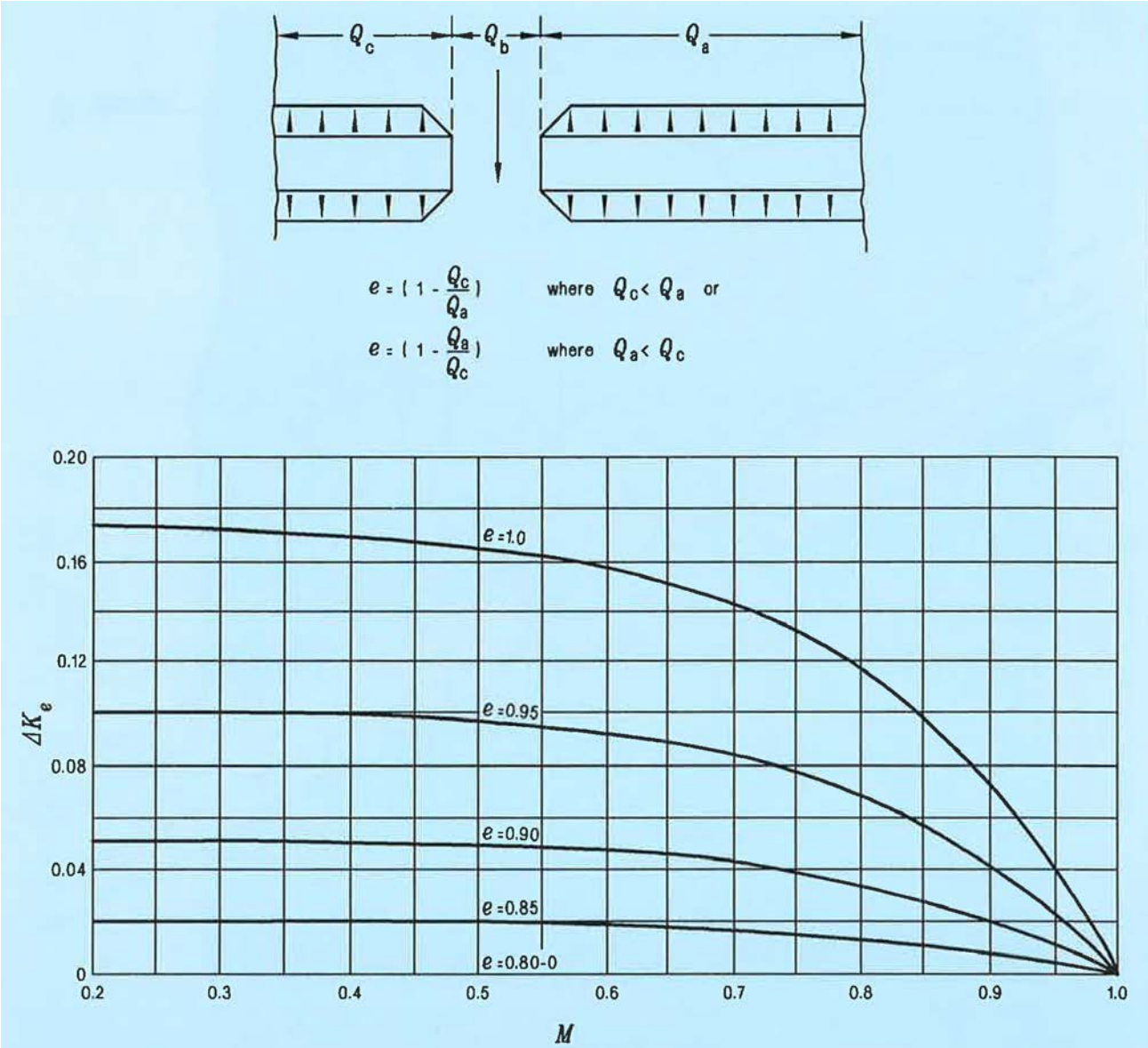


Figure 4.11: Incremental backwater coefficient for eccentricity



4.9.6 Effects of Skew

The method of computation for skew crossings differs from that of normal crossings in that the bridge opening ratio, *M*, is computed on the projected length of bridge rather than on the full length. The length is obtained by projecting the bridge opening upstream parallel to the general direction of flood flow as illustrated on Figure 4.12. The general direction of flow is the direction of flood flow as it existed prior to the placement of embankments in the stream. The length of the constricted opening is $b \cos \emptyset$ and the area, A_{n2}, is based on this length. The velocity head, $V_{n2}^2/2g$, to be substituted in the expression for computing backwater in Section 4.9.1, is based on the projected area A_{n2}.

Figure 4.13 shows the incremental backwater coefficient, ΔK_e, for the effect of skew, for wingwall and spill-through type abutments. The incremental coefficient varies with the opening ratio, *M*. the angle of skew of the bridge, $\emptyset$, with the general direction of flood flow, and the alignment of the abutment faces, as indicated by the sketch on Figure 4.13.

It should be noted that the incremental backwater coefficient, ΔK_e, can be negative as well as positive. The negative values result from the method of computation and do not indicate that the backwater will be reduced by employing a skew crossing. These incremental values are to be added algebraically to ΔK_b, obtained from the base curve. The total backwater coefficient for a skew crossing with abutment faces aligned with the flow and piers would be (Equation 19):

$$k^* = k_b(\text{Figure 4.9}) + \Delta K_p(\text{Figure 4.10}) + \Delta K_e(\text{Figure 4.13 (A)}) \qquad 19$$

From the model tests carried out it was found that crossings with skew up to an angle of 20° produced no particularly objectionable results for any of the four abutment shapes investigated. As the angle increased above 20°, however, the flow pattern deteriorated; flow concentrations at abutments produced large eddies, reducing the efficiency of the waterway and increasing the potential for scour.

The above statement does not apply to cases where a bridge spans practically an entire valley and there is little constriction of the flow.

Figure 4.14 was prepared from the same model data as Figure 4.13 (A). By entering Figure 4.14 with the angle of skew and the projected value of *M*, the ratio $b_s \cos\emptyset / b$ can be read from the ordinate. Knowing *b* and h_1^* for a comparable normal crossing, one can solve for b_s, the length of opening needed for a skewed bridge to produce the same amount of backwater for the design discharge.

Figure 4.12: Skewed crossing

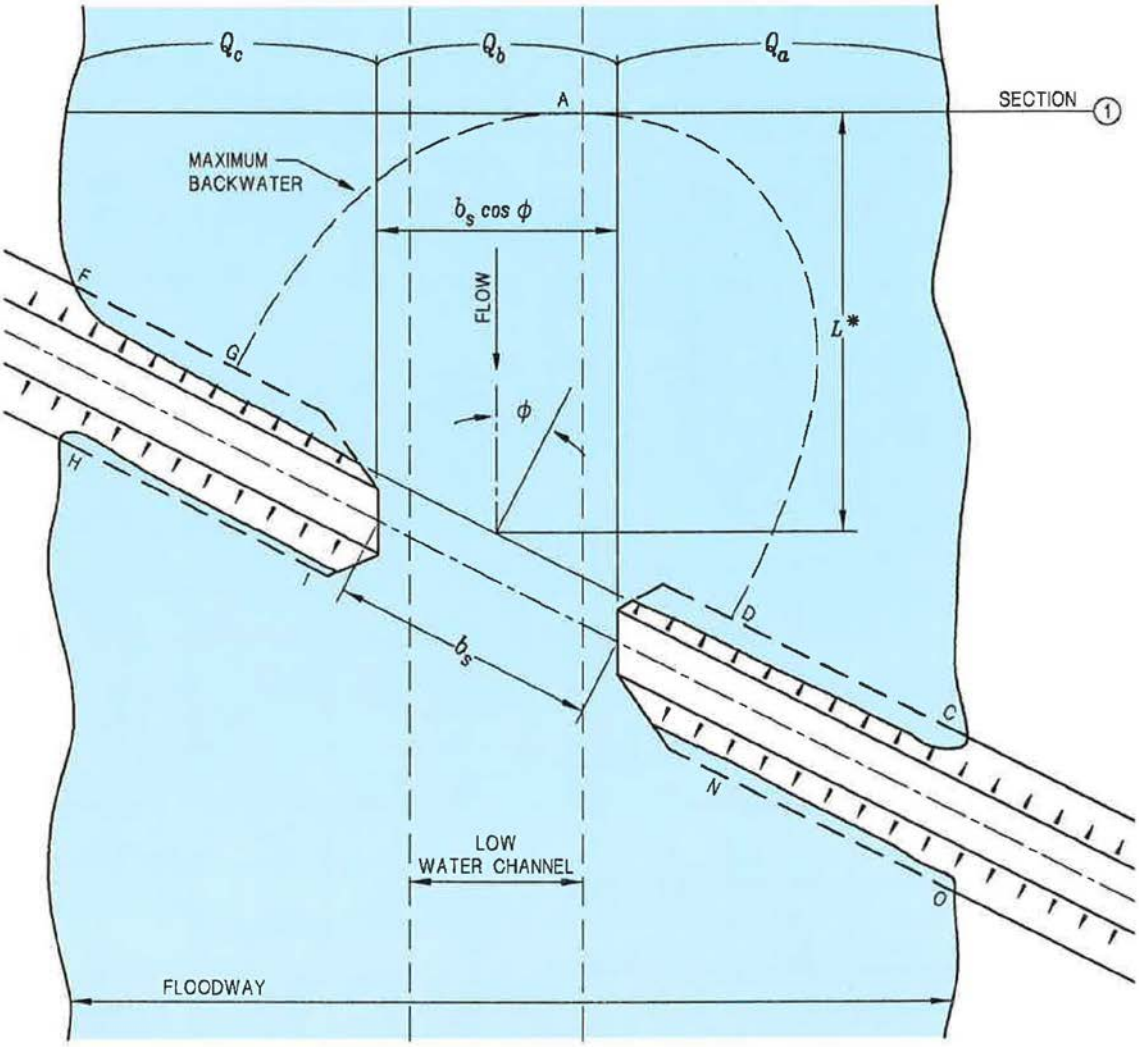


Figure 4.13: Incremental backwater coefficient for skew

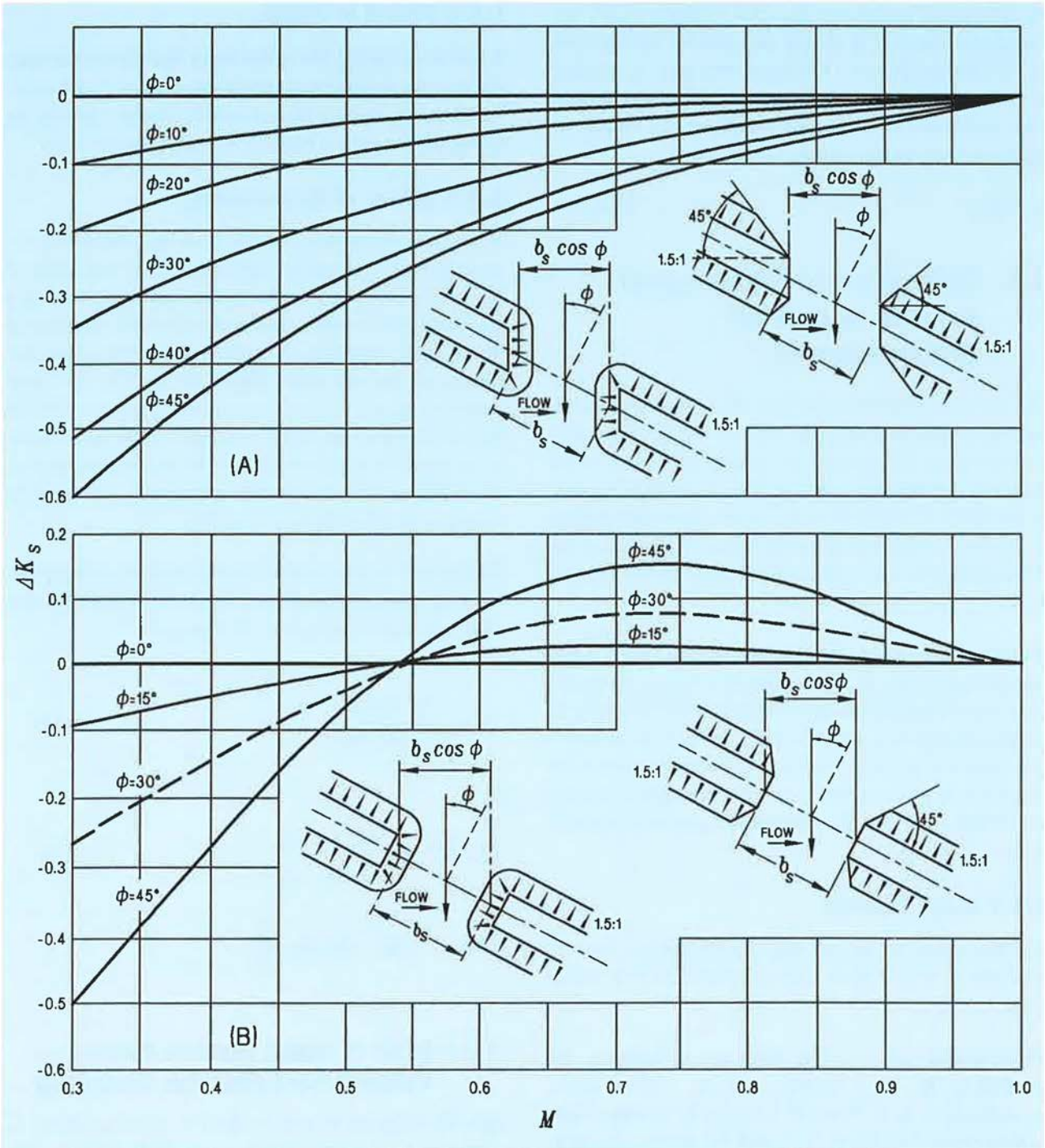
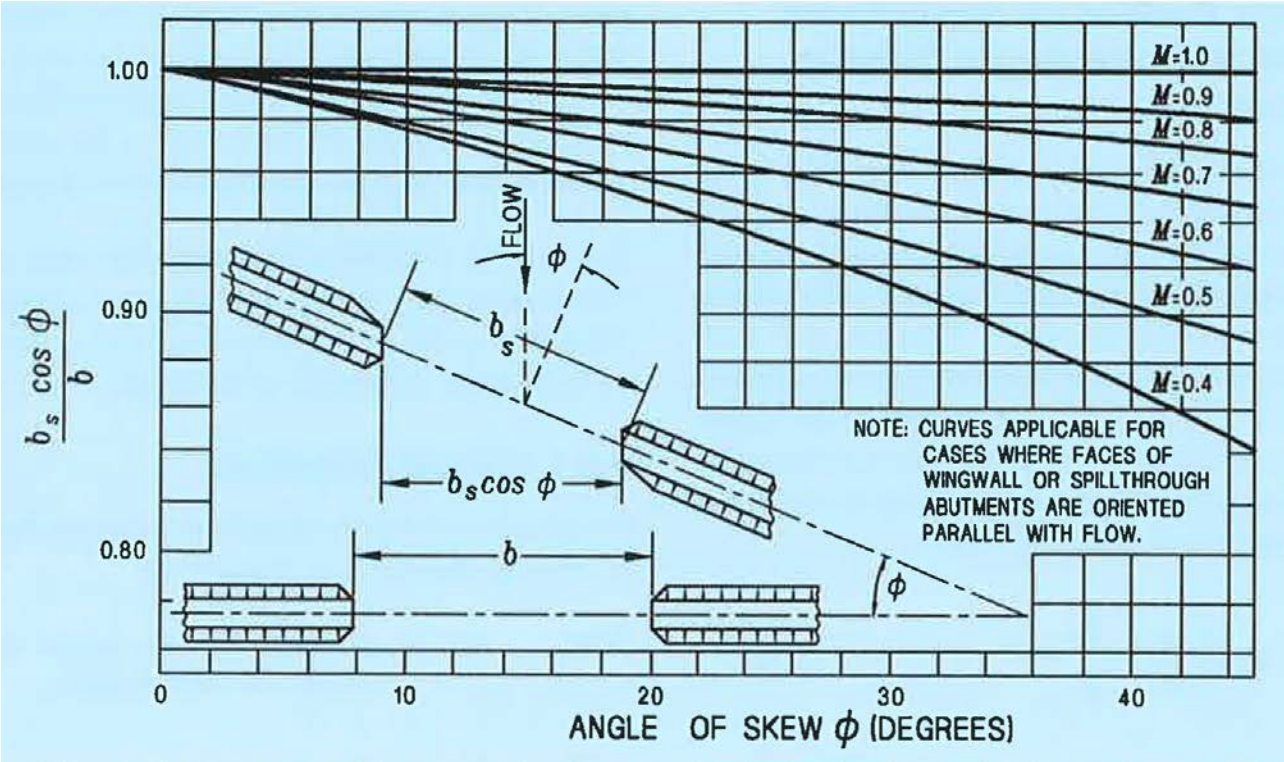


Figure 4.14: Ratio of projected to normal length of bridge for equivalent backwater (skewed crossing)



4.9.7 Effect of Dual Bridges

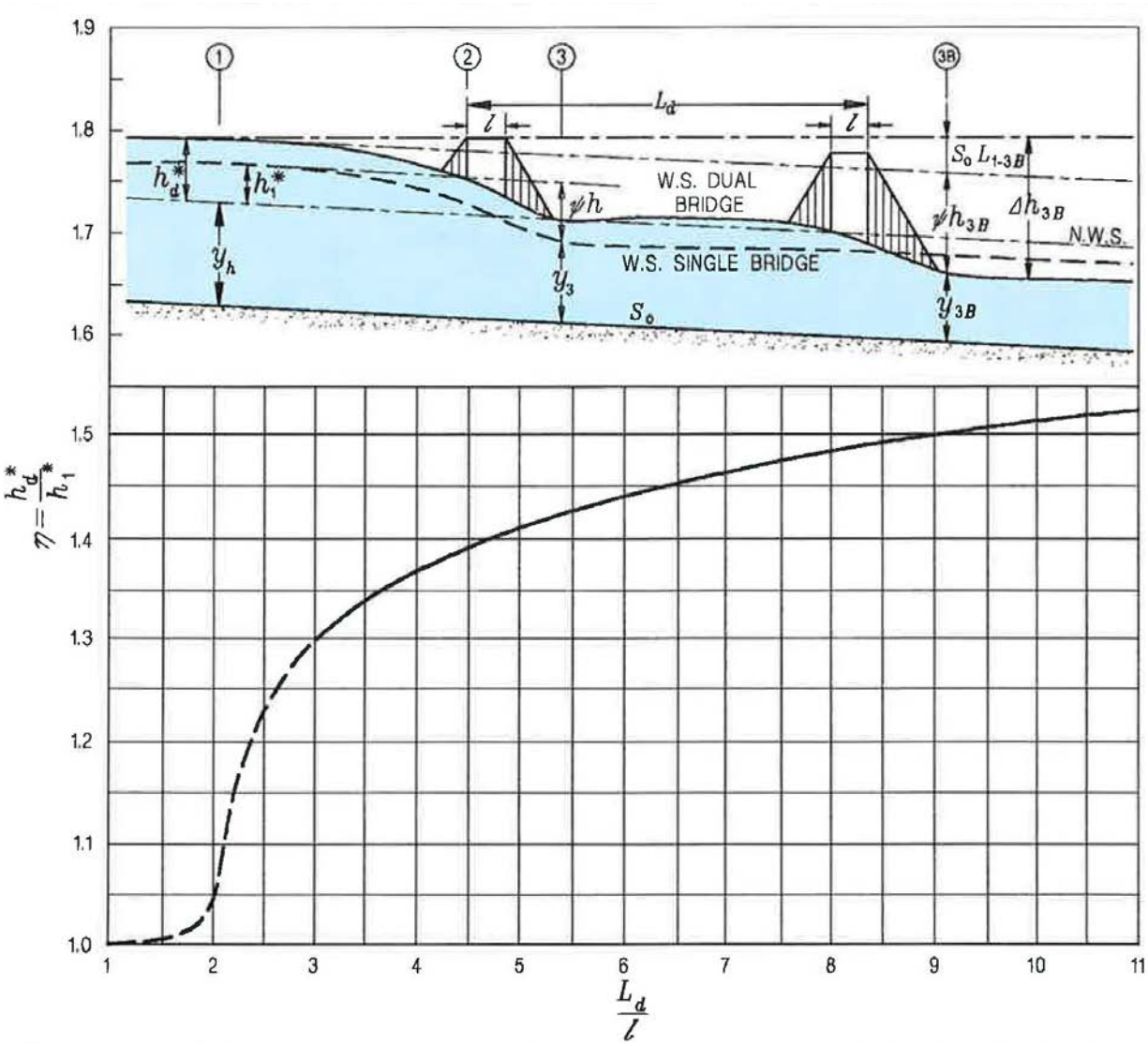
The backwater produced by dual bridges (such as two bridges, side by side, on a dual carriageway) is larger than that for a single bridge, yet less than the value which would result from considering the bridges separately.

When two bridges are in close proximity to one another, the flow pattern is elongated, but little different from that of a single bridge. As the bridges are spaced further apart, the embankment of the second bridge interferes with the expanding jet of the first, which must again contract and then re-expand downstream from the second bridge, producing additional turbulence and loss of energy. This is reflected in a larger backwater upstream of the first bridge.

To determine backwater for dual bridges which are essentially identical in design and parallel, and have an L_d/l, (see Figure 4.15) of not more than 11, it is first necessary to compute the backwater, h_1^* for a single bridge. The backwater for the dual combination, measured upstream from the first bridge is then (Equation 20):

$$h_d^* = h_1^* \eta \qquad 20$$

Figure 4.15: Backwater multiplication factor for dual bridges

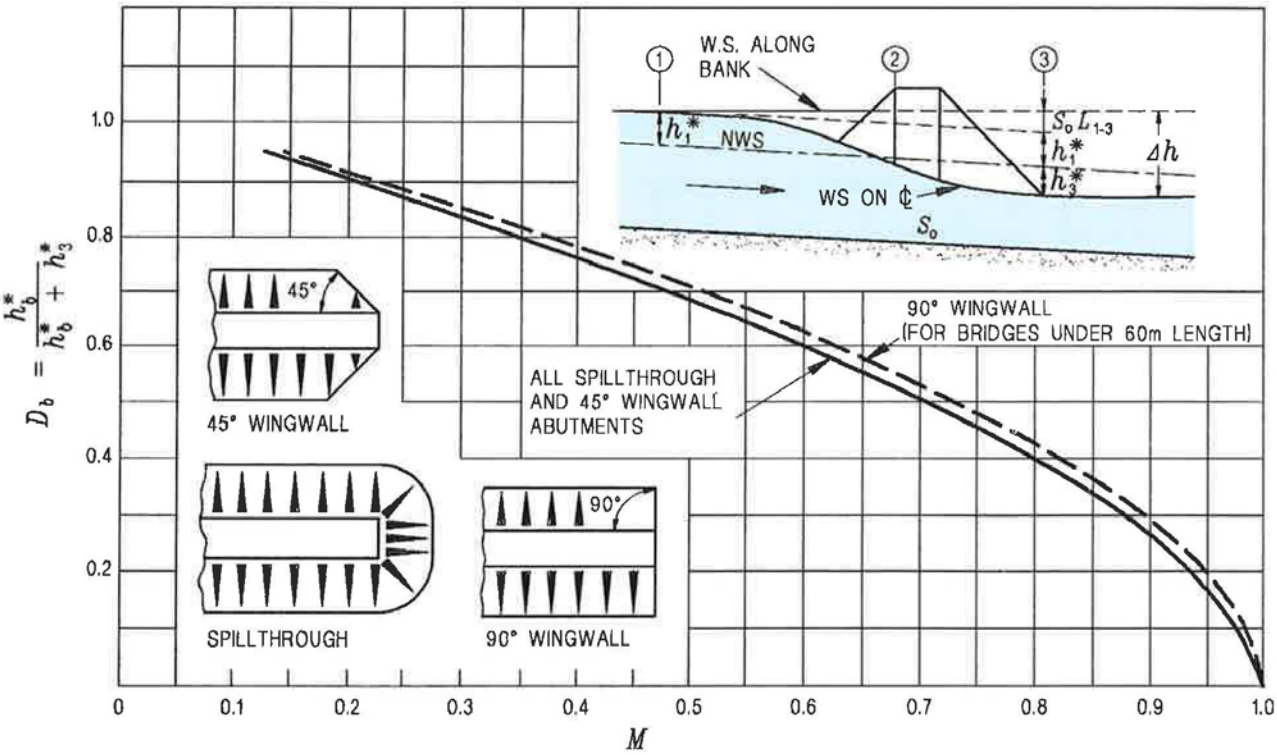


4.10 Difference in Water Level across Approach Embankments

The difference in water surface elevation between the upstream and downstream side of bridge approach embankments, Δh, has been interpreted erroneously as the backwater produced by a bridge. This is not the backwater as the sketch on Figure 4.16 will attest. The water surface at section 3 (Figure 4.4), measured along the downstream side of the embankment, is lower than normal stage by the amount h_3^*.

The difference in level across embankments is always larger than the backwater, h_1^*, by the sum $h_3^* + S_0 L_{1-3}$, where S_0 is the natural slope of the stream (Figure 4.16). The method of determining L_{1-3}, which is the distance from section 1 to section 3 (Figure 4.4) is given in Section 4.11. The differential level is significant in the determination of backwater at bridges in the field, since Δh is the most reliable head measurement that can be made.

Figure 4.16: Differential water level ratio base curves



4.10.1 Base Curves

The base curves for determining downstream levels given on Figure 4.16 have been constructed entirely from model data.

To determine Δh it is first necessary to compute the backwater, h_b^*, for a normal crossing, without piers, eccentricity or skew. The relevant curve for abutment type is then entered on Figure 4.16 with the contraction ratio, *M*, to obtain the differential level ratio (Equation 21):

$$D_b = \frac{h_b^*}{h_b^* + h_3^*} \qquad 21$$

or Equation 22:

$$h_3^* = h_b^* \left[\frac{1}{D_b} - 1\right] \qquad 22$$

The elevation on the downstream side of the embankment is simply normal stage at section 3 (Figure 4.4), less h_3^*.

4.10.2 Effect of Piers

It was found during the model study that the introduction of piers into the bridge constrictions increased backwater, whilst the h_3^* showed no measurable change. Hence, no correction for piers is required when determining Δh.

4.10.3 Effect of Eccentricity

In the case of severely eccentric crossings, the difference in level across the embankment applies only to the side of the river having the greater floodplain discharge. In plotting the experimental differential level ratios with respect to *M* for eccentric crossings, it was found that the points fell directly on the base curve (Figure 4.16). The individual values of h; and h; for eccentric crossings are different from those for symmetrical crossings, but the ratio of one to the other, for any given value of *M*, remains unchanged. Thus, Figure 4.16 can also be considered applicable to eccentric crossings if used correctly.

To obtain h_3^* for an eccentric crossing, with or without piers, enter the appropriate curve on Figure 4.16 with the value of *M* and read D_b as before. In this case Equation 23:

$$D_b = \frac{h_b^* + \Delta h_e^*}{h_b^* + \Delta h_e^* + h_3^*} \qquad 23$$

or Equation 24:

$$h_3^* = (h_b^* + \Delta h_e^*)\left[\frac{1}{D_b} - 1\right] \qquad 24$$

where

$$\Delta h_e^* = \Delta K_e \alpha_2 \frac{V_{n2}^2}{2g}$$

4.10.4 Drop in Water Surface across Embankment (Normal Crossing)

Having computed h_3^* and knowing the total backwater h_1^* (computed in accordance with Section 4.9), the difference in water surface elevation across the embankment is Equation 25:

$$\Delta h = h_3^* + h_1^* + S_0 L_{1-3} \qquad 25$$

where

h_1^* = total backwater (m), including the effect of piers and eccentricity.

$S_0 L_{1-3}$ = the normal fall in stream bed between sections 1 and 3 (Figure 4.4).

4.10.5 Water Surface on Downstream Side of Embankment (Skewed Crossing)

Individual values of h_1^* and h_3^* for skewed crossings again differ from those for symmetrical crossings, but the differential level ratio across the embankments at either end of the bridge can be considered the same as for normal crossings for any given value of *M*. The value of *M*, is of course, based on the projected length of bridge as explained in Section 4.9 for the effect of skew. Thus, it is again possible to use Figure 4.16 for skewed crossings. The differential level ratio, D_b, with or without piers, is obtained by entering the chart with the appropriate opening ratio, *M*. Then Equation 26:

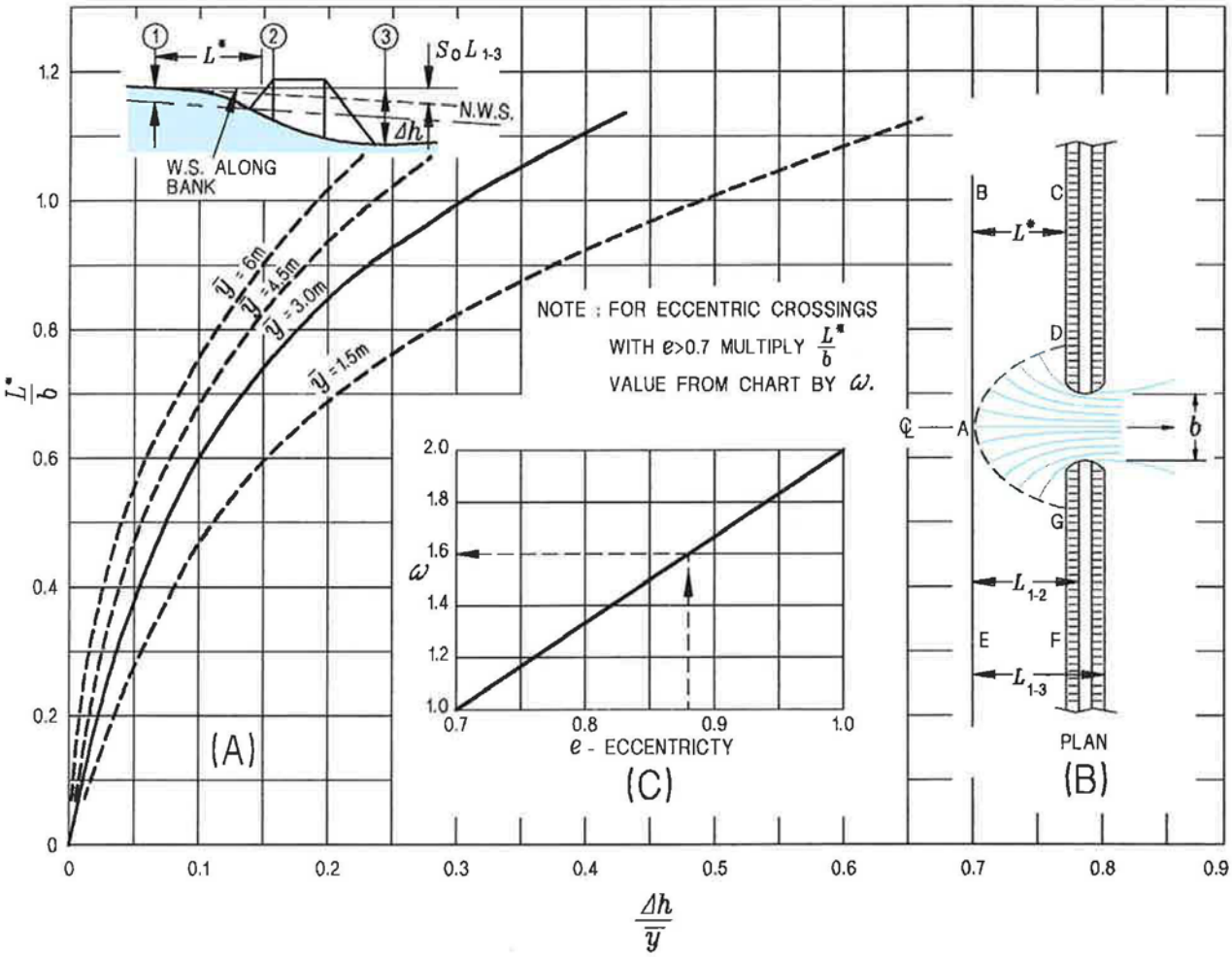
$$h_3^* = (h_b^* + \Delta h_s^*)\left[\frac{1}{D_b} - 1\right] \qquad 26$$

4.11 Location of Maximum Backwater

The maximum backwater occurs at point A upstream of the bridge centre line and at a distance L^* from the water line on the upstream side of the embankment (Figure 4.17). Where the floodplains on each side of the main channel are no wider than twice the bridge length and the hydraulic roughness is low then the water level in the areas ABCD and AEFG will be approximately the same as point A.

However, for wide and rough floodplains, there will be a flow gradient along the upstream side of the embankments. These flow gradients are likely to be more pronounced on the falling than rising stage of the flood.

Figure 4.17: Distance to maximum backwater



4.11.1 Normal Crossings

For normal crossings the distance to maximum backwater, L^*, may be obtained from Figure 4.17 with:

- Δh = difference in water surface elevation (m) across embankment
- $\overline{y} = A_{n2}/b$
- A_{n2} = cross-sectional area (m^2) under the bridge below normal water surface
- b = width (m) of waterway.

A trial solution is required for determining the differential level across the embankments, Δh, but from the result of the backwater computation it is possible to make a fair estimate of Δh. To obtain the distance to maximum backwater for a normal constriction, use Figure 4.17 with appropriate values of $\Delta h/\overline{y}$ and obtain the corresponding value of L^*/b. Solving for L^* and adding to this the additional distance to section 3, which is known, gives the distance L_{1-3}. Then the computed difference in level across the embankment is (Equation 27):

$$\Delta h = h_1^* + h_3^* + S_0 L_{1-3} \qquad 27$$

Should the computed value of Δh differ materially from the one chosen, the above procedure should be repeated until assumed and computed values agree.

4.11.2 Eccentric Crossings

Eccentric crossings with extreme asymmetry perform much like one half of a normal symmetrical crossing with a marked contraction of the jet on one side and very little contraction on the other.

For cases where the value of *e* (Section 4.9 for effect of eccentricity) is greater than 0.7, enter the abscissa on Figure 4.17(A) with $\Delta h/\overline{y}$ and $\overline{y}$ and read off the corresponding value of L^*/b as usual. Next multiply this value by L^*/b by a correction factor, ω, which is obtained from Figure 4.17(C).

4.11.3 Skewed Crossings

In the case of skewed crossings, the water surface elevations along opposite banks of a stream are usually different than at point A. One may be higher and the other lower depending on the angle of skew, the configuration of the approach channel and other factors. To obtain the approximate distance to maximum backwater, L^*, for skewed crossings (Figure 4.12), the same procedure is recommended as for normal crossings except the ordinate of Figure 4.17 is read as L^*/b_s, where is the full length of the skewed bridge.

4.12 Effect of Scour on Backwater

4.12.1 General

The estimation of backwater in Section 4.9 – Section 4.14 has been limited to the case where scour has not occurred. In actual practice where embankments have constricted the flow causing backwater and higher velocities through the bridge opening, scour will occur where the stream bed is composed of loose or soft material. If a flood persists for a sufficient period of time, equilibrium conditions will eventually result from the increase in waterway area, resultant reduction in backwater and velocity, and reduced capacity of the flow to cause further scour.

Figure 4.18 shows the effect of scour on bridge backwater.

In cases where the bridge foundations can be adequately protected and there is no adverse environmental impact, it may be permissible to encourage scour in the interest of utilising a shorter bridge. The same objective can be attained by enlarging the waterway area under a bridge by excavation during construction. In such cases it is desirable to be able to determine the amount of backwater to be expected with increase in the waterway area.

4.12.2 Backwater Determination

A design curve derived from model experiments is included as Figure 4.19. The correction factor for scour, ($C = h_{1x}^*/h_1^*$), is plotted with respect to A_s/A_{n2}, where A_s is the additional area due to scour at the constriction and the other terms bearing the subscript, *s* designate values with scour and those not bearing this subscript represent the same values computed without scour.

Supposing the backwater at a given bridge was 0.3 m with no scour; it would be reduced to 0.16 m if scour increased the waterway area by 50 per cent, and it would be reduced to 0.09 m should the waterway area be doubled.

The same reduction applies equally well to the ratios h_{3x}^*/h_3^* and $\psi h_s/\psi h$ (see Figure 4.18) so one curve suffices for all three. Thus, to obtain backwater and related information for bridge sites where scour is to be encouraged or cannot be avoided, or where the waterway is to be enlarged during construction, it is first necessary to compute the backwater and other quantities desired according to the method outlined in Section 4.9 for a rigid bed, using the original cross-section of the stream at the bridge site. These values are then multiplied by a common coefficient from Figure 4.19 as follows (Equation 28):

$$h_{1s} = Ch_1^* \qquad 28$$

$$h_{3s}^* = Ch_3^*$$

$$\psi h_s = C\psi h$$

Figure 4.18: Effect of scour on bridge backwater

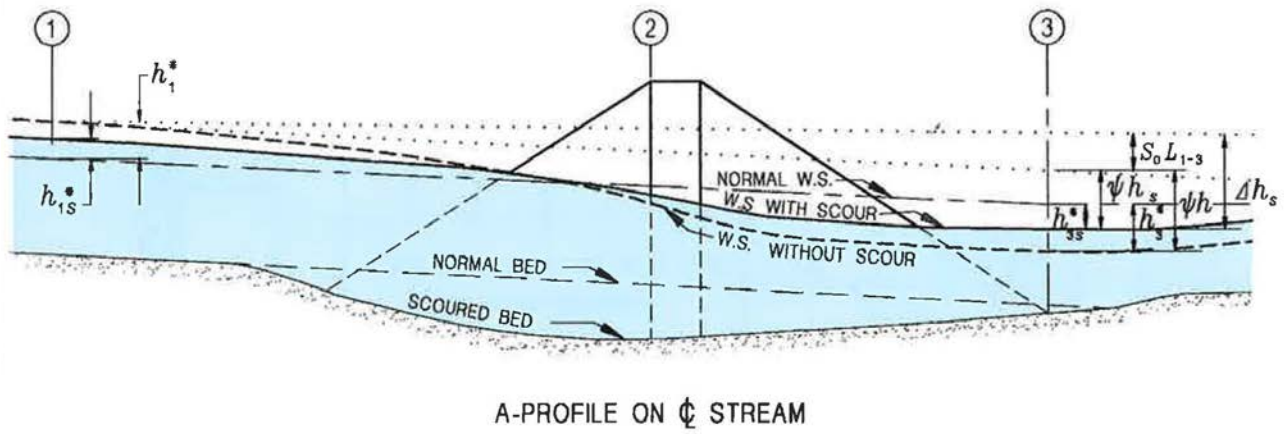
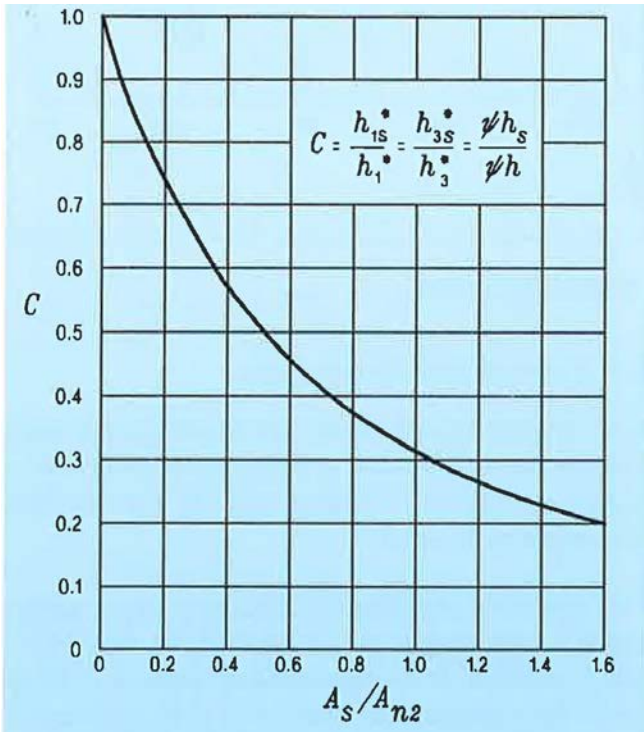


Figure 4.19: Correction factor for backwater with scour



4.12.3 Enlarged Waterways

The design engineer will probably be reluctant to depend on scour as a means of enlarging a waterway and thereby reducing backwater. If the waterway is enlarged by excavation, there is little to gain by excavating much beyond the limits (upstream and downstream) of the embankments, as the downstream channel acts as a control. If additional volume is removed upstream or downstream, the channel may simply refill by deposition. Any enlargement of the cross-section should be maintained to prevent reduction of area by the growth of vegetation.

Any proposal to reduce backwater by excavation to enlarge a bridge waterway area should be carefully studied. If there is reason to believe that the enlarged area cannot be maintained or that the stability of the stream will be disturbed, alternate solutions such as additional spans should be considered.

4.13 Superstructure Partially Inundated

Cases arise in which it is desirable to compute the backwater upstream from a bridge or the discharge under a bridge when flow is in contact with the girders. Once flow contacts the upstream girder of a bridge, orifice flow is established so the discharge then varies as the square root of the effective head. The result is a rather rapid increase in discharge for a moderate increase in backwater. The greater discharge, of course, increases the likelihood of scour under the bridge.

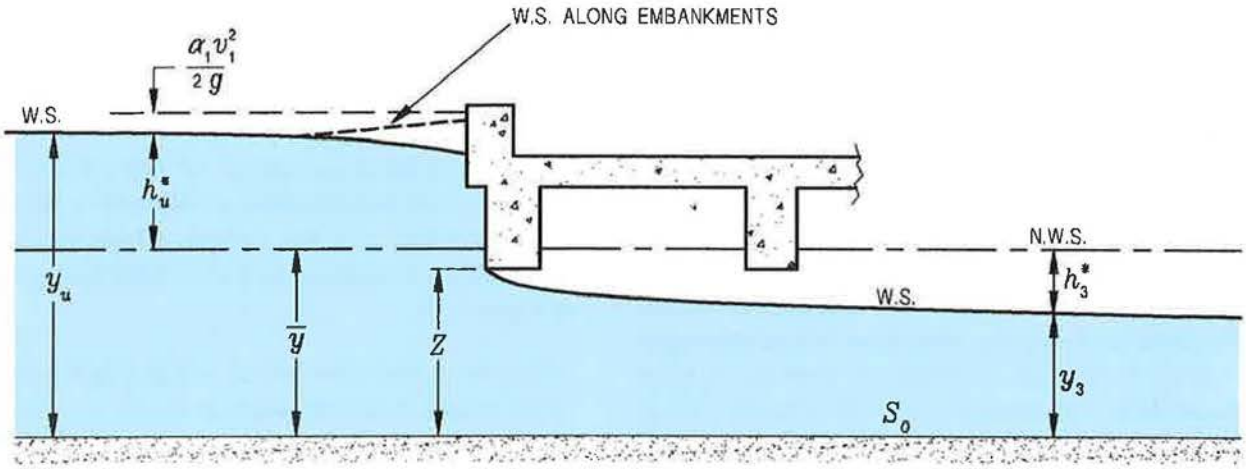
Two cases are considered below; the first where only the upstream girder is in the water as indicated by the sketch on Figure 4.20 and the second, where the bridge constriction is flowing full, all girders in the flow, as shown in Figure 4.21.

Where the normal water surface is higher than the soffits of the girders, backwater analyses should be carried out on the alternative assumptions that:

1. the upstream girder is in contact with the flow
2. all girders are in contact with the flow.

The higher of the two results should be adopted.

Figure 4.20: Discharge coefficients for upstream girder in flow (Case I)



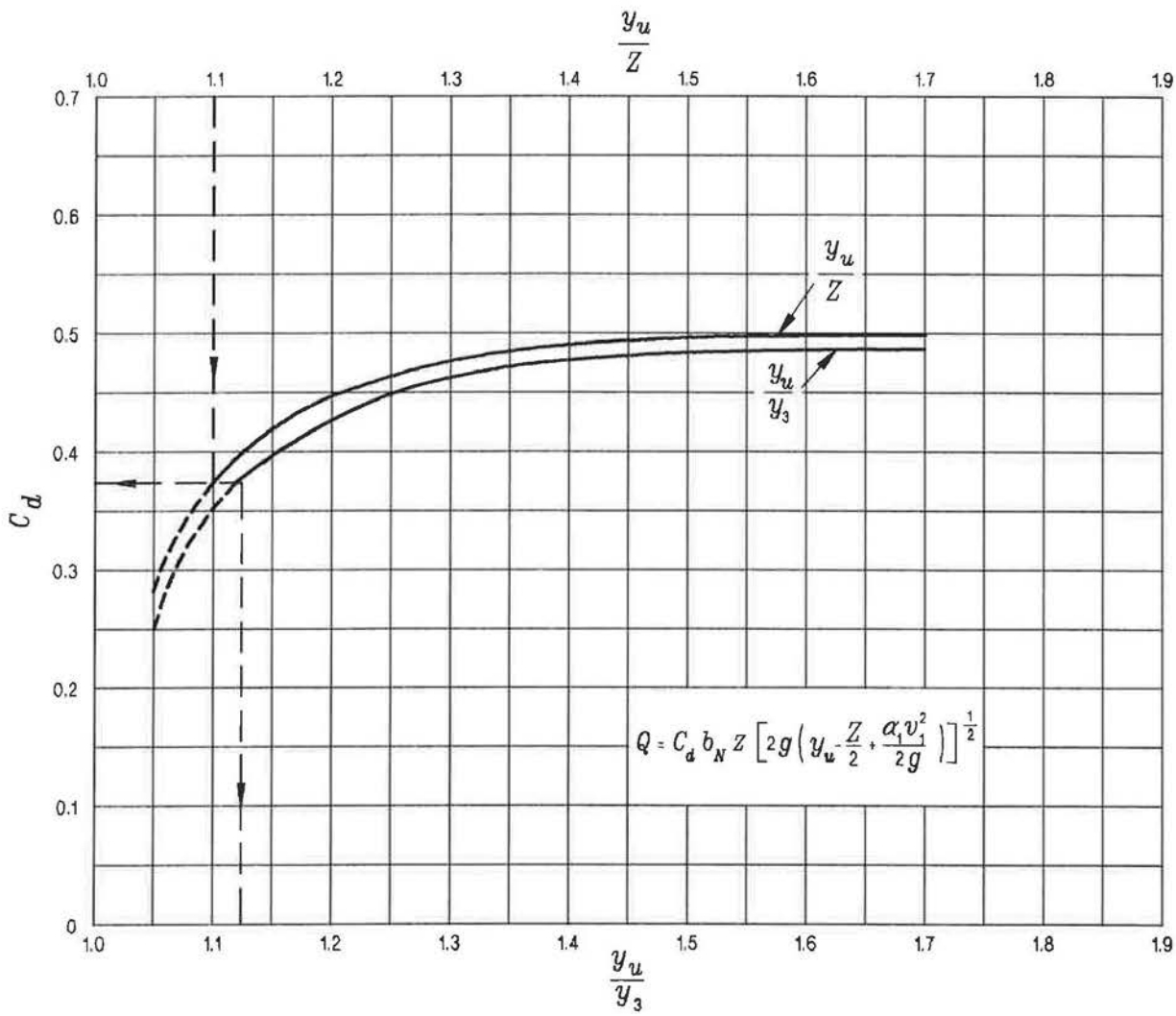
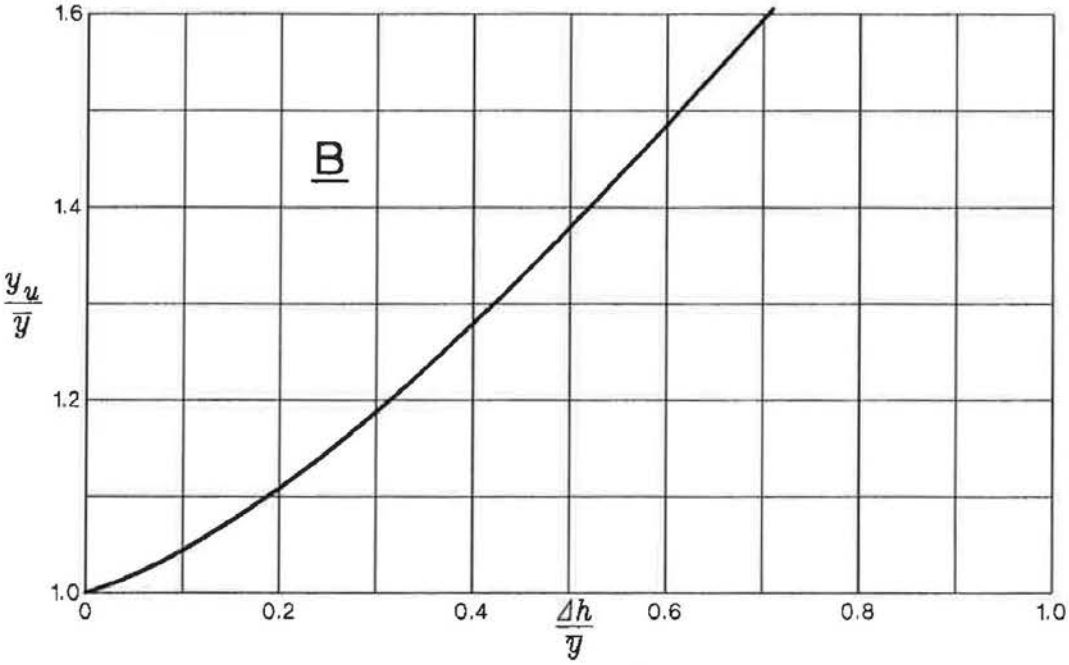
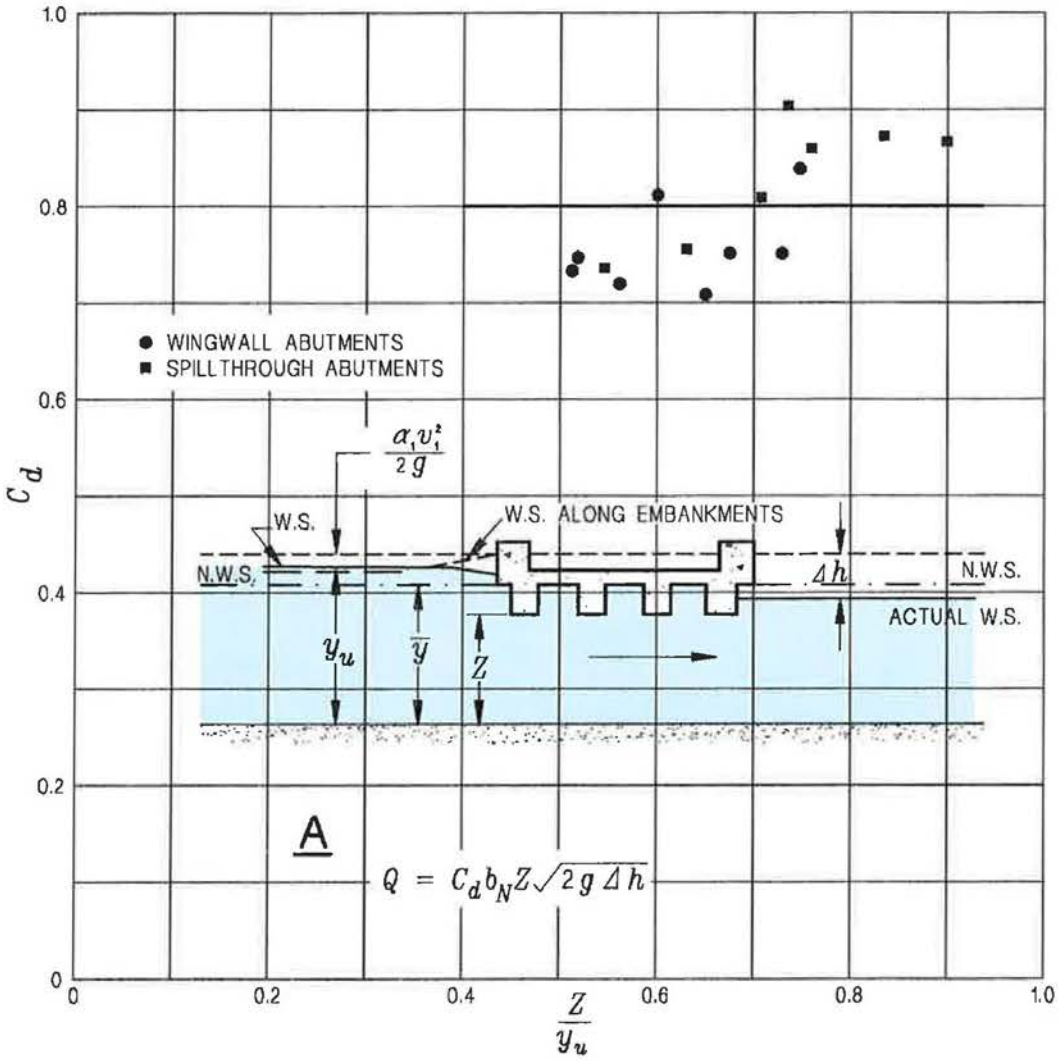


Figure 4.21: Discharge coefficients for all girders in flow (Case II)





4.13.1 Upstream Girder in Flow (Case I)

The most logical and simple method of analysis is to treat this flow condition as a sluice gate problem.

Using a common expression for sluice gate flow (Equation 29):

$$Q = C_d b_N Z \left[2g\left(y_u - \frac{Z}{2} + \frac{\alpha_1 V_1^2}{2g}\right)\right]^{1/2} \qquad 29$$

where

Q = total discharge (m³/s)

C_d = coefficient of discharge

b_N = net width (m) of waterway (excluding piers)

Z = vertical distance (m) under bridge from bottom of upstream girder to mean river bed level

y_u = vertical distance (m) of upstream water surface to mean river bed at bridge

For Case I, the coefficient of discharge, C_d is plotted with respect to the parameter y_u/Z on Figure 4.20. The upper curve applies to the coefficient of discharge where only the upstream girder is in contact with the flow. By substituting values in the above expression, it is possible to solve for either the water surface upstream or the discharge under the bridge, depending on the quantities known. It would appear that the coefficient curve (Figure 4.20) approaches zero as y_u/Z becomes unity. This is not the case since the limiting value of y_u/Z for which the expression applies is not much less than 1.1. There is a transition zone somewhere between $y_u/Z = 1.0$ and 1.1 where free surface flow changes to orifice flow or vice versa. The type of flow within this range is unpredictable. For $y_u/Z = 1.0$, the flow is dependent on the natural slope of the stream, while this factor is of little concern after orifice flow is established or $y_u/Z = 1.1$.

In computing a general river backwater curve across the bridge as shown on Figure 4.20, it is necessary to know the water surface elevation downstream as well as upstream from the bridge. The approximate depth of flow, y_3, can be obtained from Figure 4.20 by entering the top scale with the proper value of y_u/Z and reading down to the upper curve, then over horizontally to the lower curve, and finally down to the lower scale as shown by the arrows. The lower scale gives the ratio of y_u/y_3.

4.13.2 All Girders in Contact with Flow (Case II)

Where the entire area under the bridge is occupied by the flow, the computation is handled in a different manner. To compute the water surface upstream from the bridge, the water surface on the downstream side and the discharge must be known. Or if the discharge is desired, the drop in water surface across the roadway embankment Δh, and the net area under the bridge is required. The experimental points on Figure 4.21 (A), which are for both wingwall and spill through abutments, show the coefficient of is charge to be essentially constant at 0.80 for the range of conditions tested. The equation recommended for the average two to four lane concrete girder bridge for Case II is (Equation 30):

$$Q = 0.8 b_N Z (2g\Delta h)^{1/2} \qquad 30$$

Where the symbols are defined as in the expression for Case I. Here the net width of waterway (excluding width of piers) is used again. It is preferable to measure Δh across embankments rather than at the bridge proper. The partially inundated bridge behaves in a similar way to a submerged box culvert but on a larger scale. Submergence, of course, can increase the likelihood of scour under a bridge.

For working out general backwater curves for a river, it is desirable to know the drop in water level across existing bridges as well as the actual water surface elevation either upstream or downstream from the bridge. Once Δh is computed from the above expression the depth of flow upstream, y_u can be obtained from chart B, Figure 4.21 where $\bar{y}$ is the depth from normal stage to mean river bed at the bridge in metres.

4.14 Flow Passes through Critical Depth (Type II)

4.14.1 General

The computation of backwater for bridges on streams with fairly steep gradients, by the method outlined for Type I flow (refer Figure 3.4), may result in unrealistic values. When this occurs, it probably indicates that the flow encountered is Type II, and the backwater analysis for subcritical or Type I flow does not apply.

The water surface for Type IIA flow passes through critical stage under the bridge, but returns to normal or sub-critical flow some distance downstream. In the case of Type IIB flow, the water surface passes through critical stage under the bridge and then dips below critical stage downstream. The sole source of data for Type II flow is from model studies, which cover a limited range of contraction ratios.

4.14.2 Backwater Coefficients

The expression for the backwater coefficient for Type II flow is (Equation 31):

$$C_b = \frac{h_1^* + \bar{y} - y_{2c}}{\alpha_2 V_{2c}^2/2g} + \frac{\alpha_1}{\alpha_2}\left[\frac{V_1}{V_{2c}}\right]^2 - 1 \qquad 31$$

where

$\bar{y}$ = normal depth (m) in constriction = A_{n2}/b

y_{2c} critical depth (m) in constriction = A_{2c}/b

V_{2c} critical velocity (m/s) in constriction = Q/A_{2c}

A_{2c} area (m^2) in constriction below critical depth

α_2 = velocity head coefficient for the constriction

The backwater coefficient has been assigned the symbol C_b to distinguish it from the coefficient for subcritical flow.

The coefficient curve of Figure 4.22 accounts for the contraction ratio only, which is the major factor involved. The effect of piers, eccentricity, and skew have not been evaluated because of the tentative nature of the curve. The incremental coefficients on Figure 4.10, Figure 4.11 and Figure 4.13 for piers, eccentricity and skew, are not applicable to type II flow problems.

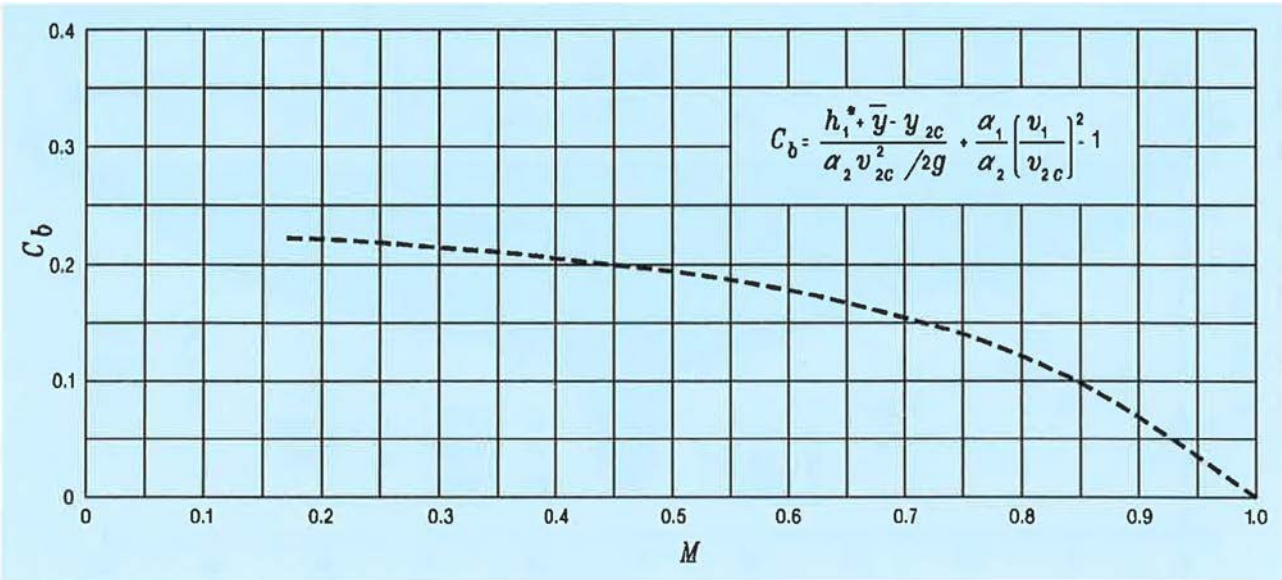
The expression for backwater for Type II flow, with no allowance for piers, eccentricity and skew, is then (Equation 32):

$$h_1^* = \alpha_2 \frac{V_{2c}^2}{2g}(C_b + 1) - \alpha_1 \frac{V_1^2}{2g} + y_{2c} - \bar{y} \qquad 32$$

4.14.3 Recognition of Flow Type

The prime difficulty here is to determine which type of flow will occur at a proposed bridge site in the field prior to starting backwater calculations. No definite answers can be given since most problems encountered of this nature will be borderline cases. It is suggested that the Type I approach is tried first and if the result appears unrealistic, repeat the backwater calculation using the Type II approach. It is more than likely that the difference in the two results will be great enough to readily spot the erratic one. If the computed backwater for the Type II flow is smaller than that computed for Type I flow, the flow will definitely be Type II.

Figure 4.22: Tentative backwater coefficient curve for Type II flow



4.15 Bridge Deck Drainage Design

This section is adopted from Section 4.10.3 of the Austroads *Guide to Road Design Part 5: Drainage: General and Hydrology Considerations* (Austroads 2013b).

The drainage of bridge decks is an important component of bridge performance. The surface water needs to be removed from the bridge deck effectively to minimise the safety hazard of water on the pavement surface, and prevent corrosion of the structure. The collected run-off needs to be discharged appropriately to meet the environmental requirements, prevent erosion of the surrounding ground and possibly undermining the foundations, and the width of flow on a bridge deck should not exceed that specified for its road approaches. Every effort should be made to ensure that longitudinal sags are not located on bridges.

4.15.1 Design Storm and Storm Duration

The following guidelines are recommended for the selection of design storm and storm duration for bridge deck drainage:

- for small and medium bridges: the design storm is 1 in 10 ARI (10% AEP) with either 6 or 10 minute duration
- for large bridge (e.g. Anzac Bridge and Sydney Harbour Bridge in NSW): 1 in 20 ARI (5% AEP) with either 10 to 20 minute duration.

4.15.2 Drainage of Carriageway

Transverse and longitudinal drainage of the carriageway should be undertaken by providing a suitable cross-fall and camber or gradient, respectively. Water flowing downgrade on bridge approaches should not be permitted to run onto the bridge unless permitted otherwise by the road agency. To reduce costs, short bridges should be detailed without formal superstructure drainage wherever possible, with the run-off from the bridge discharged into outfall drains at the end of the structure, as specified by the agency.

Longer bridges require drainage facilities; otherwise flow widths may exceed the allowable limits (which is typically 1.2 m if the shoulder width is greater than 1.2 m). Inlet structures, such as flush grates connect to the under-deck pipe work, which discharges away from the structure, waterway, or other thoroughfare beneath the structure. Drainage inlets should be of rigid, ultraviolet and corrosion-resistant material, not less than 100 mm in their least dimension, and should be provided with provision for cleanouts.

Deck drainage should be detailed to prevent the discharge of drainage water against any portion of the structure and to prevent erosion adjacent to the point of impact of the discharge from the outlet of the downpipe. The overhanging portions of a concrete deck should be provided with a drip bead or notch, which should be continuous where possible.

Drainage from bridges should not discharge directly into waterways, onto traffic lanes, railway corridors or any other thoroughfare below. As such, the use of scuppers for bridge deck drainage should be precluded.

4.15.3 Detailing for Drainage

Design details should ensure that water drains from all parts of the structure and should prevent the retention of dirt, leaves or other foreign matter. Where drainage pipes are provided in the closed cells of bridges, the pipes should be of durable material. Where pipes carrying liquids are located inside closed cells, drainage should be provided in case of leaking or bursting of the pipes.

If the bridge is located in a sag road section (Figure 4.23), there should be a maximum allowable ponding depth near the parapet. Drainage relief should be provided at the parapet by installing 100 mm diameter pipe at the parapet. This is to prevent excessive ponding of water on the bridge deck, hence increase the dead load to the bridge, and to limit the ponding depth to the shoulder of the bridge.

Figure 4.23: Flooding of Anzac Bridge due to blockage – no relief flow provided at bridge parapet

Source: Roads and Maritime (n.d.).

4.15.4 Drainage of Ballast Railway Bridges

Consideration should be given to the effective drainage of ballast-topped railway bridges, and waterproofing should be provided where necessary.

5. Bridge Scour

5.1 Introduction

This section provides an introduction into the different types of scour that can occur at a bridge site and the factors that affect scour. It also provides guidance on the following aspects of designing bridge foundations for scour:

- designing to minimise the effects of scour
- design of abutment protection works
- design of foundations for the ULS
- design of scour countermeasures at existing bridges.

A comprehensive review of scour at bridge sites has been presented by Melville (1988). This provides essential background material to give the professional engineer a clearer understanding of this phenomenon and what is known about it.

Adequate consideration should be given to the limitations and gaps in existing knowledge when using the methods of scour estimation recommended in this Guide. The design engineer needs to apply engineering judgement in comparing results obtained from scour computations with available hydrological, hydraulic and geotechnical data to achieve a reasonable and prudent design.

As little research has been carried out into scour in Australia, the following recommendations are generally based on the US FHWA practice (Arneson et al. 2012; Federal Highway Administration 1989).

5.2 Scour Characteristics

5.2.1 General

Scour is the result of the erosive action of water, excavating and carrying away material from the bed and banks of streams and from around the piers and abutments of bridges. Different materials scour at different rates. Loose granular soils are rapidly eroded by flowing water, while cohesive or cemented soils are more scour resistant. However, ultimate scour in cohesive or cemented soils can be as deep as scour in sand-bed streams. Under constant flow conditions, scour will reach maximum depth in sand- and gravel-bed material in hours; cohesive bed material in days; glacial till, sandstones, and shale in months; limestone in years, and dense granite in centuries. Under flow conditions typical of actual bridge crossings, several floods may be needed to attain maximum scour.

Determining the magnitude of scour is a complicated process due to its occasionally cyclic nature. Scour can be deepest near the peak of a flood, but is usually difficult to identify when floodwaters recede and scour holes refill with sediment.

Site-information subsurface information should be carefully studied by designers and inspectors when evaluating scour potential at bridges, giving particular attention to foundations on rock. Generally, massive rock formations with few discontinuities are highly resistant to scour during the lifetime of a typical bridge.

All equations for estimating contraction and local scour have been based on laboratory experiments with limited field verification. However, scour depths as deep as those computed by these equations have been observed in the field. The equations presented in this document are considered to be the most applicable for estimating scour depths.

5.2.2 Types of Scour

Total scour at a bridge crossing may comprise one or more of the following four inter-related components:

1. **Aggradation and degradation** – long-term stream bed elevation changes due to natural or man-induced causes within the reach of the river on which the bridge is located. Aggradation involves the deposition of material eroded from other sections of a stream reach, whereas degradation involves the lowering or scouring of the bed of a stream over relatively long reaches due to a deficit in sediment supply from upstream.
2. **Scour due to river morphology** – occurs naturally in the stream; it is a function of flow conditions and associated channel characteristics. It includes general bed movement and scour at channel contractions and bends.

   In addition, naturally-occurring lateral migration of a stream may also occur. This can erode abutments, the approach roadway or change the total scour by changing the flow angle of attack.
3. **Contraction scour** – the scour resulting from the contraction of flow by bridge approach embankments encroaching onto the floodplain and/or into the main channel (Figure 5.1). This scour causes a lowering of the streambed across the stream or waterway bed at the bridge.

   Scour due to a naturally-occurring channel contraction is similar to scour resulting from contraction of flow by bridge approach embankments and is treated as contraction scour.

   Contraction scour is different from long-term degradation in that contraction scour occurs in the vicinity of the constriction may be cyclic and/or related to the passing of a flood.
4. **Local scour** – involves removal of material from around piers, abutments, guide banks and embankments (Figure 5.2). It is caused by an acceleration of flow and resulting vortices induced by the flow obstructions.

The design scour depth at a site is assessed by combining the effects of these scour components.

Figure 5.1: Excessive scour due to significant constriction of waterway at a bridge site in NSW

Source: Roads and Maritime (n.d.).

Figure 5.2: Local scour at a bridge pier due to significant constriction of waterway

Source: Roads and Maritime (n.d.).

5.2.3 Factors Affecting Scour

Factors which can affect the extent of scour at a bridge include:

- slope and alignment of the natural stream
- bed material of stream and floodplains
- vegetation in the stream, and floodplains
- changes or potential changes in the prevailing conditions in the stream or the catchment, whether man-made or natural
- depth, velocity and alignment of flow through the constriction
- alignment and layout of the bridge and training works
- accumulation of debris (Figure 4.1)
- size, shape, orientation and arrangement of piers, footings and piles
- amount of bed material in transport.

5.2.4 Clear-Water and Live-Bed Scour

There are two conditions under which contraction and local scour may occur: clear-water scour and live-bed scour. Clear-water scour occurs when there is no movement of the bed material of the stream upstream of the crossing, but the acceleration of the flow and the vortices created by piers or abutments causes the material in the crossing to move. Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach into the bridge cross-section. With live-bed contraction scour the area of the contracted section increases until, in the limit, the transport of sediment out of the contracted section equals the sediment transported in. Live-bed scour is cyclic in nature, i.e. the scour hole that develops during the rising stage of a flood refills during the falling stage.

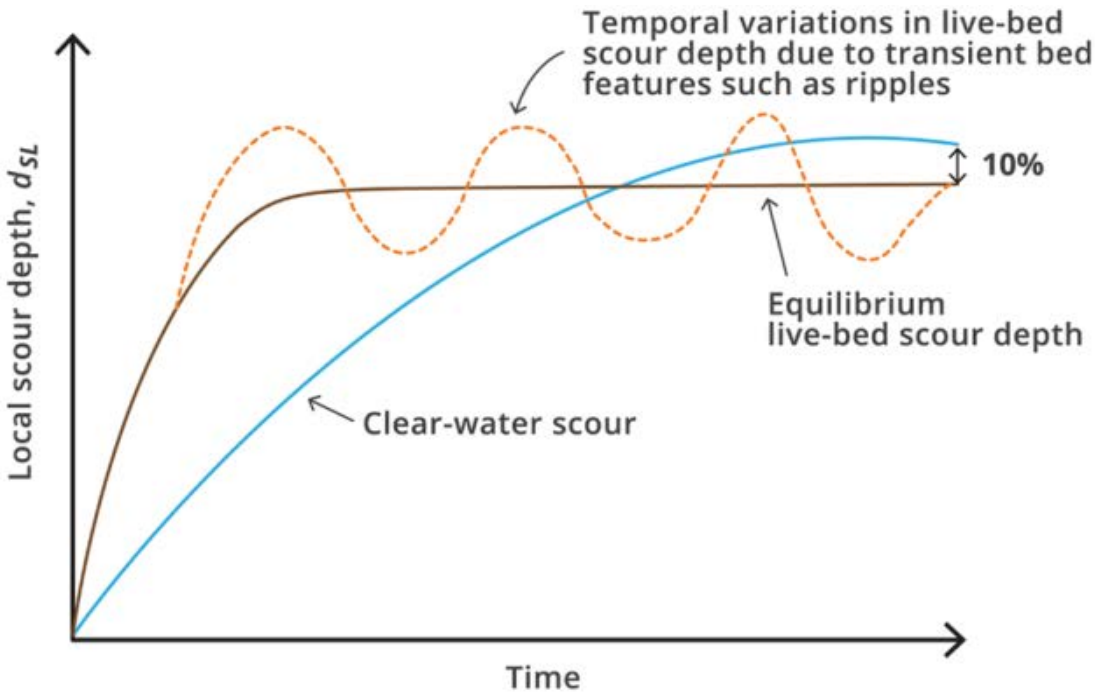
Typical clear-water scour situations include:

- coarse bed material streams
- flat gradient streams during low flow
- armoured stream beds
- vegetated channels.

During a flood event, bridges over streams with coarse bed material are often subjected to clear-water scour at low discharges, live-bed scour at the higher discharges and then clear-water scour on the falling stages.

Clear-water scour reaches its maximum over a longer period of time than live-bed scour (see Figure 5.3 for a comparison of live-bed and clear-water scour, as a function of time, at a pier in a sand-bed stream). This is because clear-water scour occurs mainly in coarse bed material streams. In fact, clear-water scour may not reach a maximum until after several floods. Maximum clear-water scour is about 10% greater than the equilibrium live-bed scour.

Figure 5.3: Illustrative pier scour depth in a sand-bed stream as a function of time (not to scale)



Source: Book 6, Ball et al. (2016).

5.2.5 Aggradation and Degradation

To determine what long-term bed elevation changes will occur in the life of a structure, the design engineer should carry out an assessment using the principles of river mechanics. Some of the factors that affect long-term stream bed elevation changes are:

- dams or reservoirs (upstream or downstream of a bridge)
- changes in watershed land use (urbanisation, deforestation, etc.)
- channelisation
- cut-offs of meander bends (natural or man-made)
- changes in the downstream channel control
- sand or gravel mining from the stream bed
- diversion of water into or out of the stream
- natural lowering of the total system.

5.2.6 Scour Due to River Morphology

With the rise in stage accompanying flood passage through an alluvial river reach, there is an increase in velocity and shear stress on the bed. As a result, the channel bed tends to scour during high flow. Because sediment is being contributed from upstream, as the shear decreases with the fall of stage the sediment tends to be deposited on the bed. This occurs with both perennial and ephemeral alluvial streams.

A greater depth of scour will occur at channel contractions, because of the decrease in waterway area and resultant increase in velocity. This scour may occur over the full width of the stream bed. At bends, the non-uniform velocity distribution may cause scour of the bed and bank at the outside of the bend and deposition on the inside of the bend. High velocities at the outside of the bend or downstream of the bend can substantially contribute to local scour at abutments and piers.

5.2.7 Contraction Scour

Contraction scour occurs when the flow area is constricted by a bridge (Figure 5.4). The decrease in flow area increases the average velocity and bed shear stress through the constriction. This results in an increase in erosive forces and more bed material being removed from the constricted reach than is transported into it. The resultant scour lowers the natural bed elevation in the constriction. As the bed elevation is lowered, the flow area increases and the velocity and shear stress decrease until relative equilibrium is reached. At this point, the quantity of bed material transported into the reach is equal to that removed from the reach.

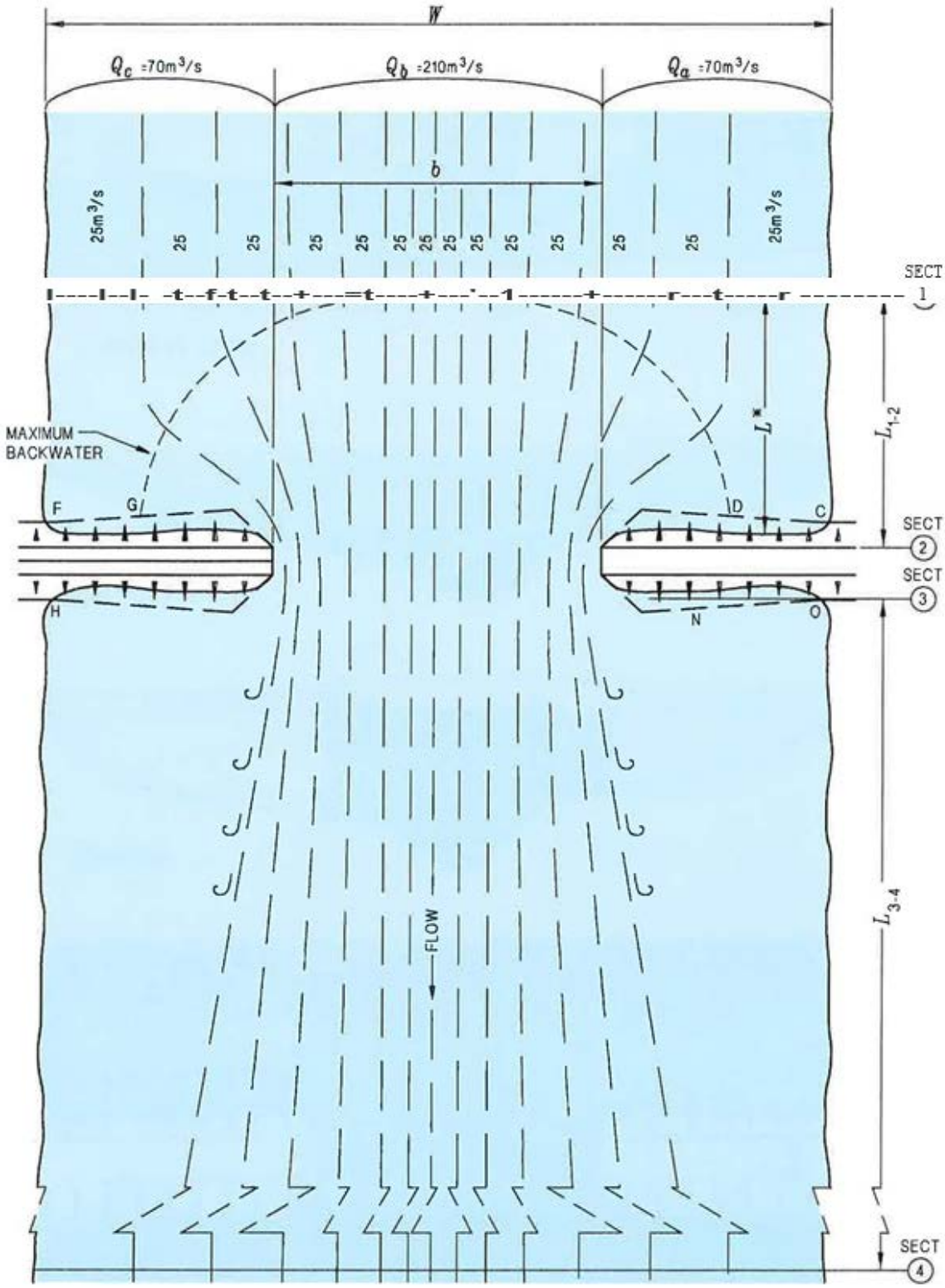
Contraction scour is similar to the scour that would occur naturally in a stream contraction and is typically cyclic. That is, the bed scours during the rising stage of a runoff event, and fills on the falling stage.

The constriction of flow due to a bridge can be caused by a decrease in flow area of the stream channel by the abutments projecting into the channel or by the approaches to a bridge cutting off the floodplain flow. Clear-water scour is more likely to occur at the bridge section, because the floodplain flow normally does not transport significant concentrations of bed material sediments. This clear water picks up additional sediment from the bed upon reaching the bridge opening.

When bridge superstructures obstruct flow or become completely submerged, pressure flow occurs. Under these circumstances contraction scour and local scour can be expected to increase substantially. The depth of scour will depend on the velocity of the approach flow and the clearance between the superstructure and the stream bed. For the same approach velocity, constriction and local scour can be expected to increase with decreasing clearance between the superstructure and the stream bed.

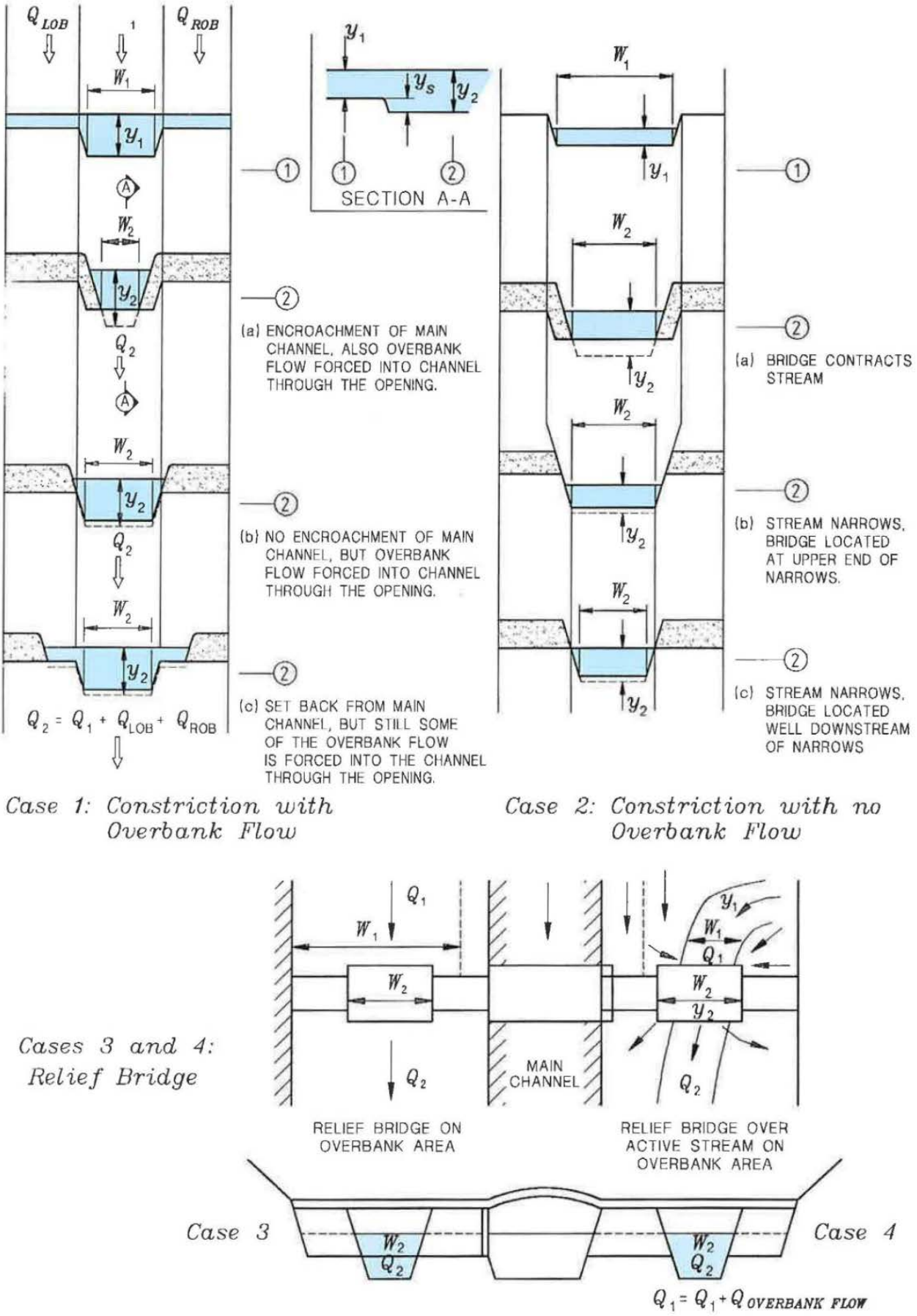
A guide bank at an abutment decreases the risk from scour at the abutment by realignment of the stream lines of the floodplain flow, so that they enter the bridge opening clear of the bridge abutment. However, scour will occur at the upstream end of the guide bank and this may extend downstream under the bridge. Another method of decreasing abutment scour is to install relief bridges. They decrease the scour problem by decreasing the quantity of clear-water returning to the main channel.

Figure 5.4: Flow lines for typical normal crossing



Contraction scour can be caused by different bridge site conditions. There are four conditions as illustrated on Figure 5.5 and now described (Arneson et al. 2012).

Figure 5.5: The four main cases of contraction scour



Case 1 – Overbank flow is forced back to the main channel by the bridge approaches. There are three different situations in which this can occur:

a. The river channel width becomes narrower, either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river (Figure 5.6).

b. No constriction of the main channel is involved, but the overbank flow area is completely obstructed by the embankment (Figure 5.7).

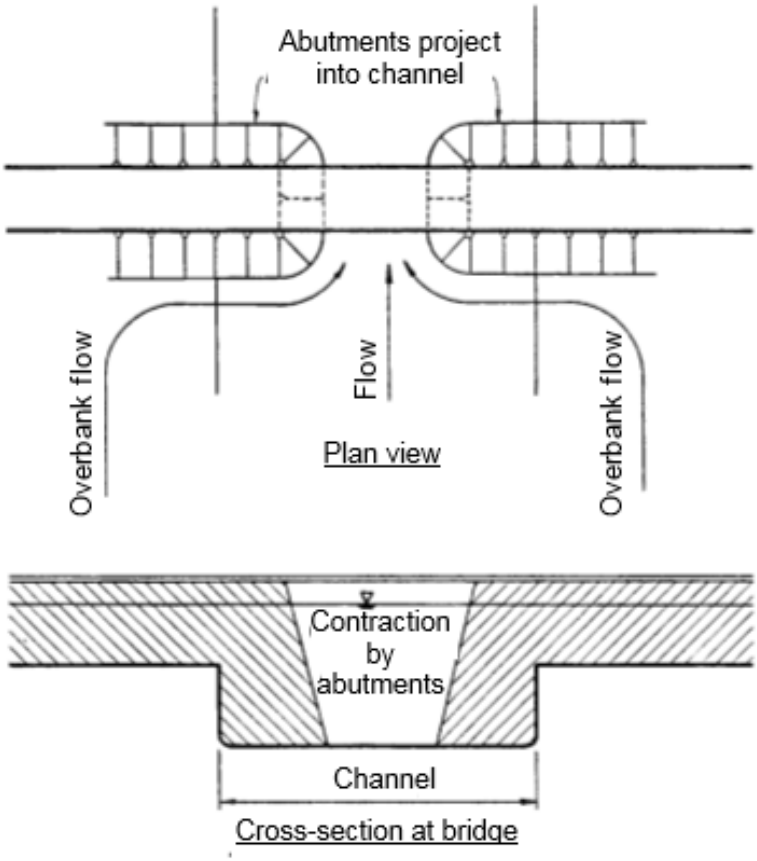
c. Abutments set back from the stream channel (Figure 5.8).

Case 2 – Flow is confined to the main channel with no overbank flow. The normal river channel width becomes narrower due to the bridge itself or the bridge site being located at a narrower reach of the river (Figure 5.9 and Figure 5.10).

Case 3 – A relief bridge in the overbank area with little or no bed material transport in the overbank area; i.e. clear-water scour (Figure 5.11).

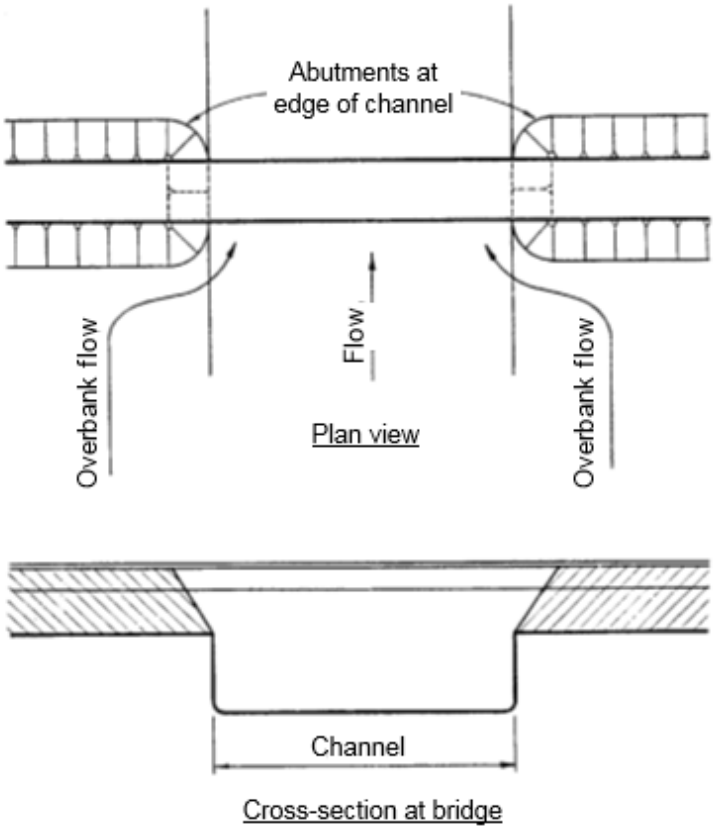
Case 4 – A relief bridge over a secondary stream in the overbank area (similar to Case 1) (Figure 5.12).

Figure 5.6: Case 1a: abutments project into channel



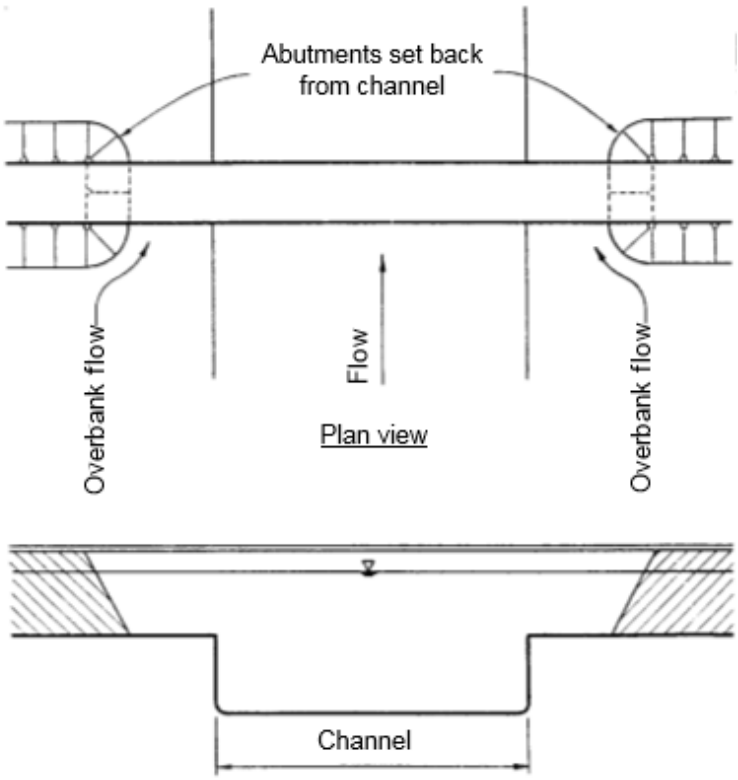
Source: Arneson et al. (2012).

Figure 5.7: Case 1b: abutments at edge of channel



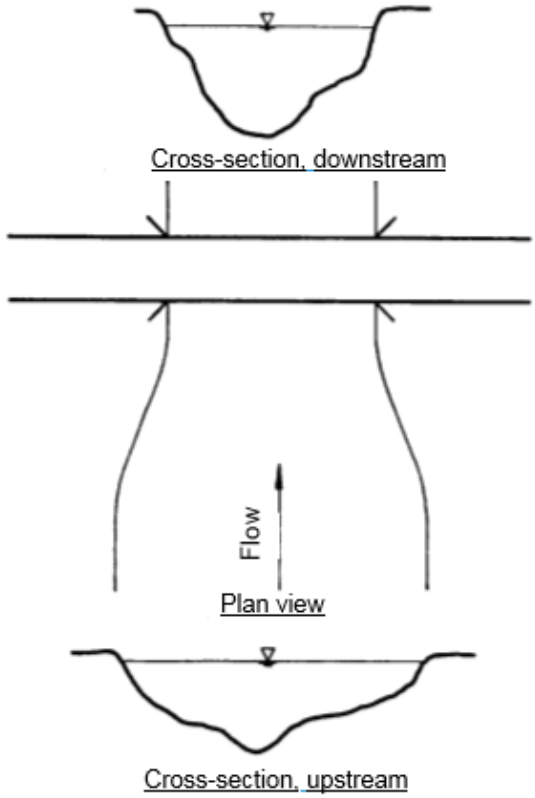
Source: Arneson et al. (2012).

Figure 5.8: Case 1c: abutments set back from channel



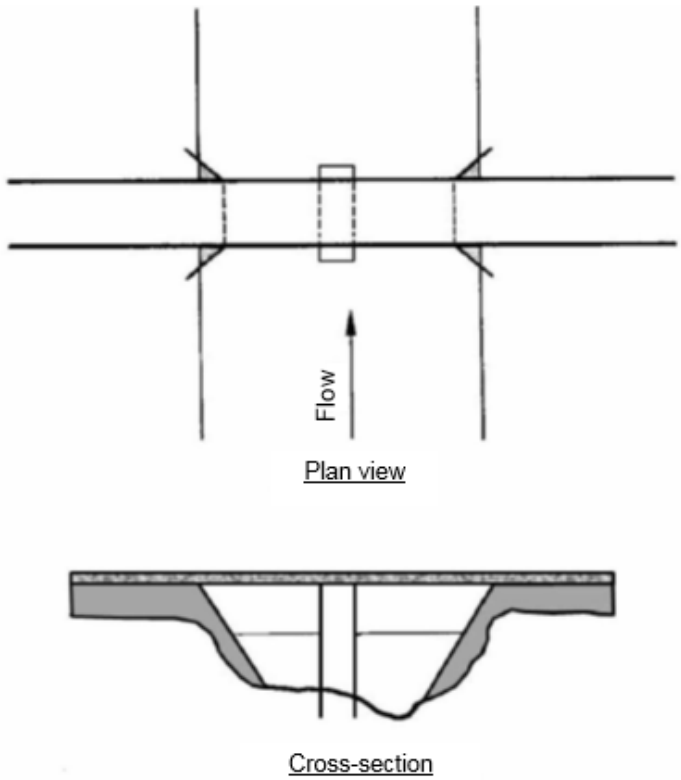
Source: Arneson et al. (2012).

Figure 5.9: Case 2a: river narrows



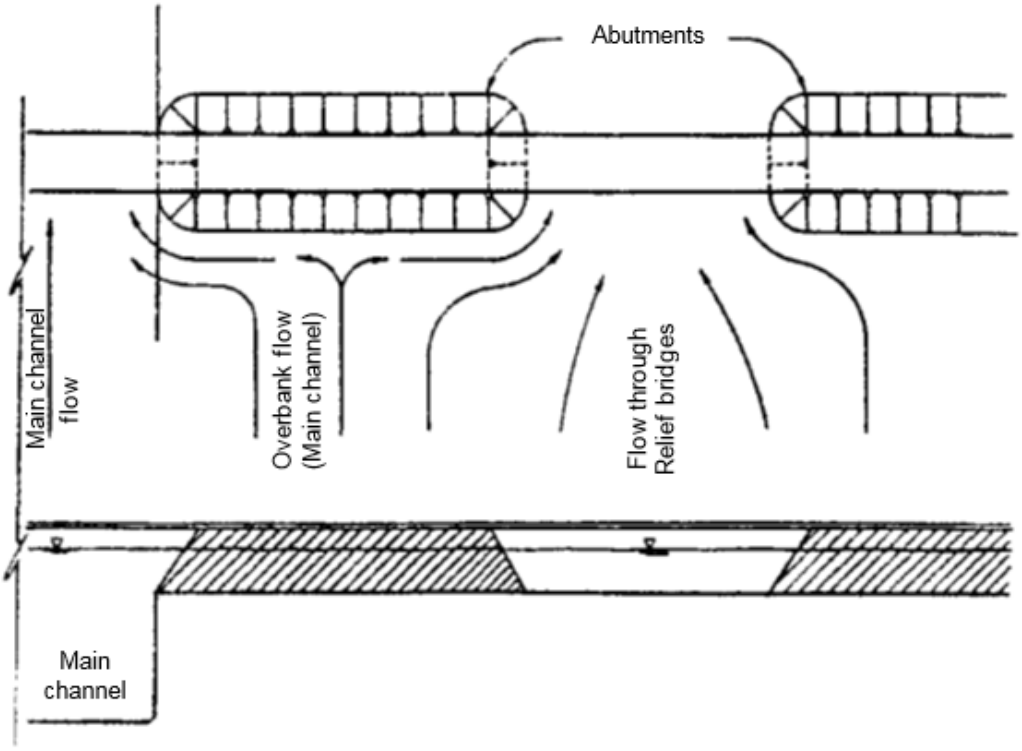
Source: Arneson et al. (2012).

Figure 5.10: Case 2b: bridge abutments and/or piers constrict flow



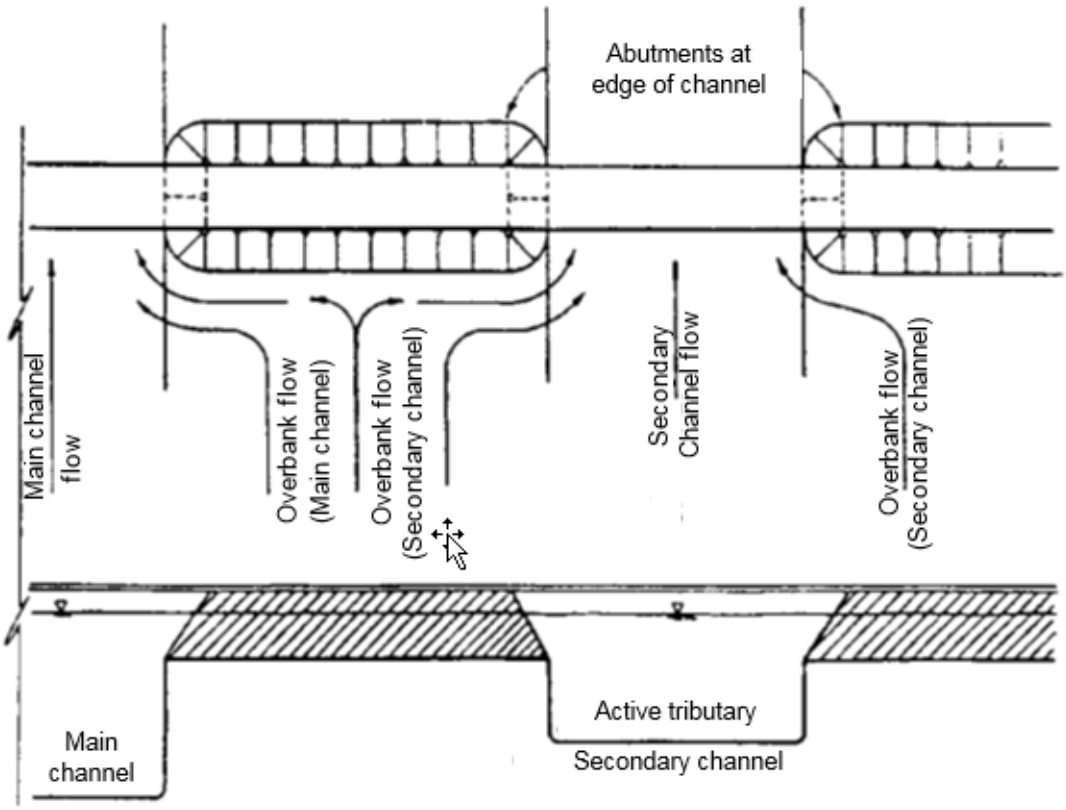
Source: Arneson et al. (2012).

Figure 5.11: Case 3: relief bridge over floodplain



Source: Arneson et al. (2012).

Figure 5.12: Case 4: relief bridge over secondary stream



Source: Arneson et al. (2012).

Notes:

- Cases 1, 2, and 4 may either be live-bed or clear-water scour depending on whether there is bed material transport from the upstream reach into the bridge reach during flood flows.
- For case 1c, the depth of contraction scour depends on factors such as
  - how far back from the bank line the abutment is set
  - the condition of the overbank (is it easily eroded, are there trees on the bank, is it a high bank, etc.)
  - whether the stream is narrower or wider at the bridge than at the upstream section
  - the magnitude of the overbank flow that is returned to the bridge opening
  - the distribution of the flow in the bridge section, and other factors.

    In this case, the main channel under the bridge may be live-bed scour; whereas, the set-back overbank area may be clear-water scour.
- Case 3 may be clear-water scour even though the floodplain bed material is composed of sediments with a critical velocity that is less than the flow velocity in the overbank area. The reasons for this are
  - there may be vegetation growing part of the year
  - if the bed material is fine sediments, the bed material discharge may go into suspension (wash load) at the bridge and not influence contraction scour.
- Case 4 is similar to Case 3, but there is sediment transport into the relief bridge opening (live-bed scour). This case can occur when a relief bridge is over a secondary channel on the floodplain.

5.2.8 Local Scour

The basic mechanism causing local scour at a pier (Figure 5.13) or abutment (Figure 5.14) is the formation of vortices at their base, resulting from acceleration of the flow around the nose of the pier or embankment. The vortex removes bed material from the base of the obstruction and a scour hole develops. As the depth of scour increases, the strength of the vortices is reduced, thereby reducing the transport rate from the base region, and eventually equilibrium is re-established and scouring ceases.

Figure 5.13: Example of local scour at piers

Source: Roads and Maritime (n.d.).

Figure 5.14: Example of local scour at an abutment

Source: Roads and Maritime (n.d.).

In addition to a horseshoe vortex around the base of a pier, there is a vertical vortex downstream of the pier called the wake vortex. Both vortices remove material from the pier base region. However, the intensity of these forces diminishes rapidly as the distance downstream of the pier increases. Therefore, immediately downstream of a long pier there is often deposition of material.

Figure 5.15 and Figure 5.16 show the usual form of scour at an abutment and abutment and adjacent pier, respectively, and Figure 5.17 shows the usual form of local scour holes at piers.

Figure 5.15: Scour at an abutment

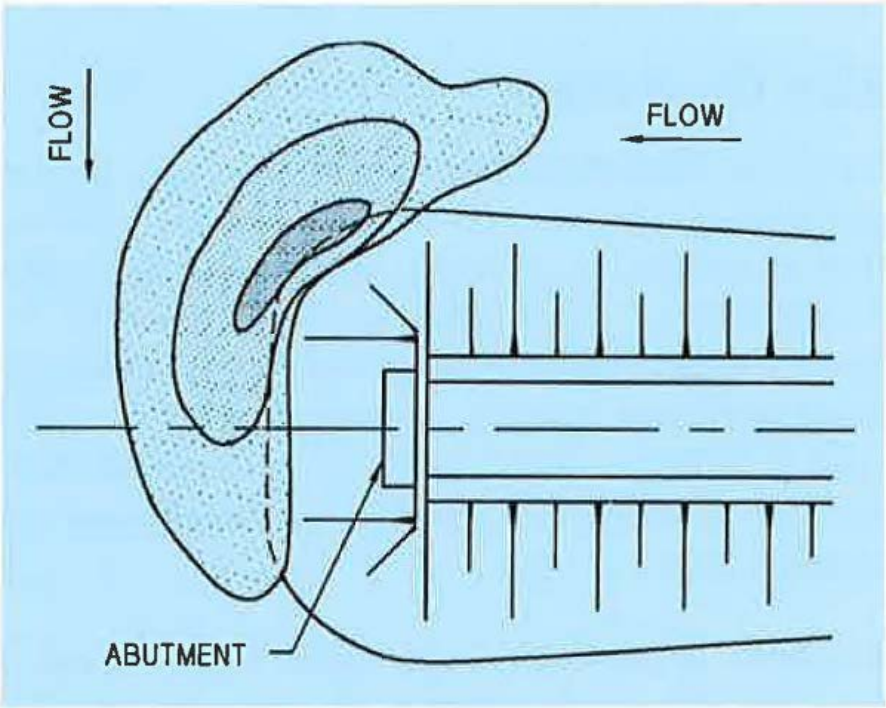


Figure 5.16: Scour at an abutment and adjacent pier

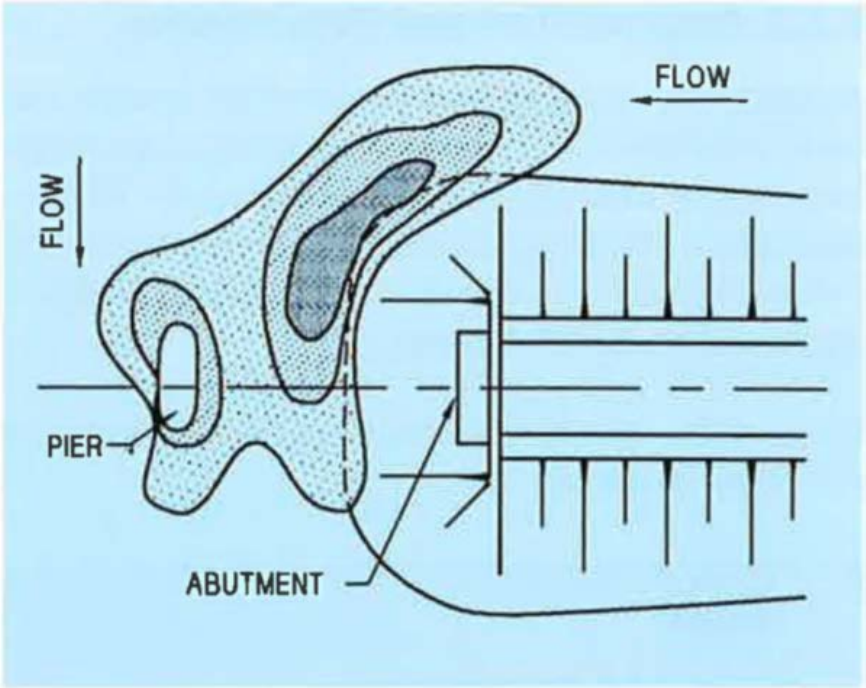
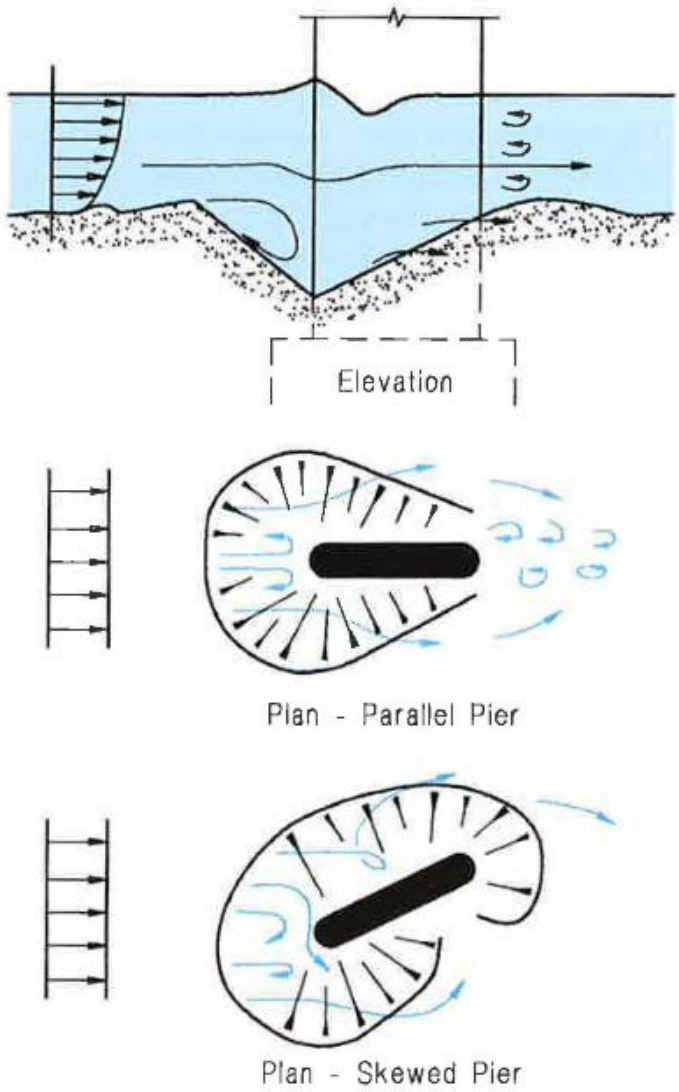


Figure 5.17: Usual form of local scour holes at piers



5.3 Bridge Scour Design and Evaluation

5.3.1 General

Damage to bridge approaches from rare flood events can usually be repaired quickly. However, a bridge that has sustained major damage or failed from scour can create safety hazards to motorists, as well as large social impacts and economic losses over a long period of time. A greater assurance that scour will not endanger the foundations of a bridge is required, therefore, than is warranted for the design of its approaches.

AS 5100.1 requires that account shall be taken of the corresponding scour at the relevant floods. Any scour protection, if provided at the bridge piers, shall not be relied upon. However, abutments shall be adequately protected to prevent scour that could affect the stability of the bridge for floods up to the SLS flood.

The hydraulic capacity of a bridge varies widely, as it is highly dependent on both the design of the structure and the waterway itself. However, design of scour protection should consider the flood event that produces the highest velocity and greatest bed shear. This will generally be the flood event that overtops the bridge, and is usually during the rising limb of the event. This suggests that the greatest velocity does not always correspond with the peak flood level (TMR 2013).

Preventative design is favoured by the economics of repairing post scour damage. The final bridge design should balance all of the competing interests, given that if the 100 year ARI (1% AEP) design flood is used for a collection of bridges, then over a 50 year lifespan the design flood will be exceeded for 4 out of every 10 bridges.

The aim of bridge design should identify the flood event that produces the highest velocities and worst case. The scour design event should be considered as the design flood event that produces an overtopping event plus an additional 300 mm in water surface. This additional overtopping amount and increased blockage factors can be tailored to the site characteristics for factors such as debris loading. Specifying a particular design event will ignore the subtle differences between designs and is being deliberately avoided. Furthermore, an extreme flood event (2000 year ARI or 0.05% AEP) may not be the design flood that produces the greatest turbulence or highest velocity in a reach. The structural design of the bridge still requires the design to be tested against the extreme flood event.

The floods that should be used for scour estimation are as follows:

- for the evaluation of bridge foundations – the 2000 year ARI (0.05% AEP) flood or the overtopping flood, if it produces more severe scour conditions (see Section 2.1.5)
- for the design of protection works to the fill around bridge abutments and to bridge approaches – the total waterway design flood (see Section 2.1.2).

Waterway investigations of bridge sites should address both the sizing of the bridge waterway and the designing of the foundations to resist scour. The scope and depth of the investigation should be commensurate with the importance of the road and the consequences of failure.

The size of a bridge will have an impact on scour. The waterway area is determined by the length and height of the deck, as well as the channel geometry. The waterway area of the bridge and flow conditions of the channel will determine the velocities through the bridge, with high velocities resulting in scour.

As flood levels are increased, the flow begins to be built up behind the bridge deck. Up to the point of overtopping, the flow through the bridge becomes pressure flow as flow is driven through the bridge opening by the additional hydraulic head (thickness of the superstructure). This often also includes debris trapped on guardrail which will increase hydraulic head by the additional blockage. Therefore, in terms of scour reduction it is preferable to increase the waterway area of the bridge by extending the bridge rather than increasing the deck level and associated bridge approach embankments. As there will be less constriction (horizontal and vertical) in the channel, the flow velocities through longer bridges will be lower, minimising pressure flow and scour of the bed material. However, increasing bridge length is typically more costly than increasing the height. Figure 5.18 shows an example of flood damage on a bridge site over Hunter River at Bowmans Crossing (NSW). This bridge was designed with a 1 in 20 flood immunity. During a flood in 2007 which was close to a 1 in 20 year flood, high stream velocity had occurred through the bridge.

Figure 5.18: High stream velocity through the bridge during flooding

Source: Roads and Maritime (n.d.).

Abutment scour usually occurs within several zones of sediment and soil, leading to different rates of erosion. The bed of the main channel is more erodible than the floodplain, because the bed is formed of loose sediment while the floodplain is formed from more cohesive soil often protected by a cover of vegetation. Abutments are essentially short, erodible (in the direction of flow) contractions. As the flow width narrows, the stream velocity increases as does the associated turbulence. Higher flow velocities and large-scale turbulence around an abutment may erode the abutment. Field observations indicate that, two prime scour regions develop as follows:

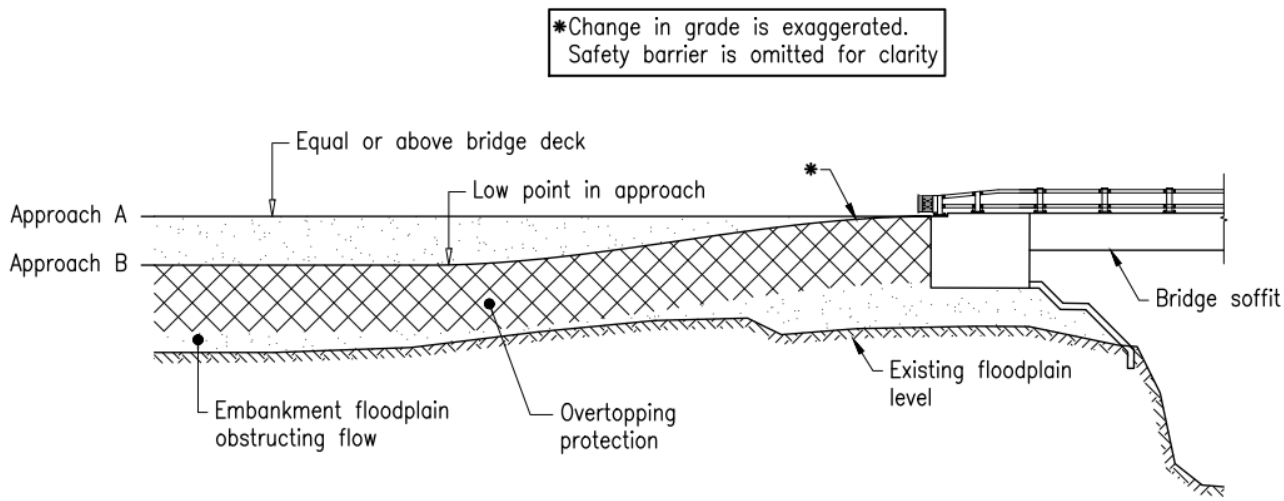
- where the channel or overbank bed is least resistant to hydraulic erosion; this could be the main bed if flow velocities are sufficiently large
- where the flow velocities and turbulence are greatest; this usually is near the abutment.

Optimum bridge design should consider the approach levels where high flood immunity is not feasible.

Building the bridge approach embankment across the floodplain at, or above, the level of the bridge deck will exacerbate contraction scour. In some situations, typically when crossing floodplains on inland rivers, it may be preferable to deliberately reduce the level of the approach embankment to below the deck soffit. Figure 5.19 illustrates the difference between a sag and crest vertical curves in a bridge design. A crest vertical curve (Approach B) will lower the approach to below the bridge deck. As water levels rise, the bridge approach acts as a weir and flow overtops the bridge approach, protecting the bridge. In Approach A, severe vertical contraction scour results from flow being dammed behind the road embankment and being forced through, and over, the bridge. Shallow flow over a long bridge approach will generally exceed the flow through the bridge's waterway opening. This lowers velocity and pressure flow and reduces the hydraulic forces acting at the bridge and scour of the bed. Allowing the bridge approaches to be overtopped will reduce scour and time of submergence, allowing the flood to travel unhindered downriver. This is particularly appropriate in locations where bridge flood immunity is very low or where afflux is a concern due to nearby houses.

Velocities through bridge approaches should be kept below 2.5 m/s or lower. This requires finding a careful balance between time of submergence and bridge scour concerns.

Figure 5.19: Crest vertical curve (Approach B) will minimise vertical contraction scour (Approach A blocks more floodplain flow)



Source: TMR (2013).

5.3.2 New Bridges

The following general design procedure for scour is recommended for determining bridge type, size and location of substructures:

1. Determine relevant flood event(s). If there is an overtopping event that causes greater hydraulic stresses to the bridge than the hydraulic design event then that flood should be used for computing scour and designing the foundations.
2. Develop hydraulic parameters necessary to estimate scour for the flood flows in Step 1 by applying a one- or two-dimensional hydraulic model. The full range of hydraulic conditions that could impact the flow conditions at and near the bridge being designed.
3. Estimate total scour for the hydraulic conditions identified from Steps 1 and 2 above. The resulting scour computed from the selected flood event should be considered in the design of a foundation.
4. Plot the total scour depths obtained in Step 3 on a cross-section of the stream channel and floodplain at the bridge site.
5. Evaluate the results obtained in Steps 3 and 4 to determine if they are reasonable. This should be based on the judgment of a multi-disciplinary team comprised of hydraulic, geotechnical, and structural engineers. There are many factors that could affect the magnitude of the overall scour estimate, including but not limited to storm duration, erodibility of channel materials, flow conditions or debris.
6. Evaluate the proposed bridge size, configuration, and foundation elements on the basis of the scour analysis performed in Steps 3 through 5. Modify the design as necessary taking into account various measures to minimise scour such as increasing bridge length, adjusting the location of the bridge, changing the configurations of substructure elements and providing guide banks.
7. Perform the bridge foundation analysis on the basis that all streambed material in the scour prism above the total scour line (Step 4) has been removed and is not available for bearing or lateral support.

5.3.3 Existing Bridges

Scour countermeasures (Section 5.5) can be designed and installed for scour critical bridges. Normally the critical flood or, otherwise, the 50 or 100 ARI (2% or 1% AEP) floods are to be used for scour protection design.

5.3.4 Design Procedures for Abutment Protection

The common approach is to design the abutment protection to accommodate the waterway design flood (or SLS) without damage, and assume the abutment is fully scoured under the ULS flood event.

The available methods for estimating the magnitude of abutment scour were developed in a laboratory environment and lack field verification (Arneson et al. 2012). In the field, conditions can vary significantly from the ideal conditions found in a laboratory. As a result, these methods may over-estimate the magnitude of actual scour. Abutments should be protected against scour by the use of one of several approaches in order to assure that they or the fill material placed around them does not fail. These approaches include:

- The use of a designed scour countermeasure (such as rock riprap and/or guide banks) to keep scour from developing at the base of the abutment or adjacent embankments. This method is a reasonable and cost effective approach for determining abutment foundation depth, but relies on a properly designed and inspected scour countermeasure.
- Assuming that all embankment fill material has washed away and that the abutment essentially behaves as a pier. This method is advantageous in that the failed embankment can be more easily repaired than a failed abutment, but provides a disadvantage due to the adverse flow conditions in the floodplain and channel near the abutment. Therefore, if scour estimated by treating the abutment as a pier could lead to deep foundation depths.
- Using procedures specifically developed for estimating abutment scour. As these are empirical methods, the hydraulic variables must be accurately and realistically determined.

The following procedure can be used for the design of the protection measure:

1. Select either the total waterway design flood or a lesser flood, if it is expected to produce the most severe scour conditions. When a bridge is designed to be overtopped with a flood with an ARI less than the total waterway flood, it is quite likely that this flood will produce the worst conditions.
2. Develop water surface profiles for the flood flows in Step 1.
3. Using the procedures in Section 5.4, estimate the contraction scour and local pier scour for the worse condition from Step 1.
4. Plot the total scour depths obtained in Step 3 on a cross-section of the stream at the bridge site. The cross-section should include the soil profiles obtained from the geotechnical investigation at the site.
5. Considering the limitations in current scour estimation procedures, are the scour estimates reasonable? Based on engineering judgement the adopted scour depth(s) may differ from the calculated values.
6. Determine the rock riprap protection required at abutments and guide banks for the depth of contraction scour adopted in Step 5. If the local pier scour impacts on the abutment, then provide sufficient rock protection to protect both the abutment and pier.

The following should be considered in determining the form and extent of protection to be provided:

- the degree of uncertainty in the scour prediction method
- the potential for and consequences of failure
- the added cost of making the bridge abutments and approaches less vulnerable to scour
- the overall flood flow pattern at the bridge site.

5.3.5 Foundation Design to Resist Scour

Foundations in the floodplain should generally be placed at the same level as those in the stream channel, to allow for possible lateral shifting of the stream. Abutment foundations should be placed below the elevation of the thalweg in the bridge opening.

Spread footings on soil – the top of the footing should be placed below the design scour line. When there is any risk of scour undermining spread footings, deep foundations in the form of piles should be used.

Spread footings on rock highly resistant to scour – the bottom of the footing should be placed directly on the cleaned rock surface. Small embedments should be avoided since blasting to achieve keying frequently damages the rock structure and makes it more susceptible to scour. If lateral restraint is required, it should be provided with steel dowels drilled and grouted into the rock.

Spread footings on erodible rock – careful assessment is required of rock that is potentially erodible. The decision on whether rock will be susceptible to scour should be based on an analysis of intact rock cores, including rock quality designations and local geology, as well as hydraulic data and anticipated structure life.

Excavation into erodible rock should be made with care. All loose rock should be removed from the excavation and any overbreak beneath the footing should be made up with lean concrete. The footing should be poured against the sides of the excavation for the full depth of the footing. The excavation above the top of the footing should be backfilled with rock riprap sized to withstand flood flow velocities.

Piled foundations with pile caps – the top of the pile cap should be placed at a depth equal to the contraction scour depth. This will minimise obstruction to flood flows and resulting local scour.

Factors that will assist in minimising scour are:

- the alignment of the piers with the direction of flood flows
- the use of round piers, especially where there are complex flow patterns during flood events
- the use of piers streamlined to decrease turbulence and minimise the potential for the build-up of debris
- the use of spill-through abutments, rather than vertical wall abutments, which produce twice as much scour.

5.3.6 Evaluation of Foundation Design for ULS Scour

The following procedure can be used:

1. Determine whether the bridge will or will not be overtopped with a flood with an ARI less than 2000 years (an AEP greater than 0.05%), as indicated in Section 2.1.5. If it will be overtopped, use the overtopping flood to evaluate the foundation design. If not, then the 2000 year ARI (0.05% AEP) flood should be used to evaluate the foundation design.
2. Develop water surface profiles for the flood flows in Step 1.
3. Using the procedures in Section 5.4, estimate the total depths of scour for the worse condition from Step 1 above.
4. Plot the total scour depths obtained in Step 3 on a cross-section of the stream at the bridge site. The cross-section should include the soil profiles obtained from the geotechnical investigation at the site.
5. Considering the limitations in current scour prediction procedures, are the answers obtained in Steps 3 and 4 reasonable? Based on engineering judgement the adopted scour depth(s) may differ from the calculated values.
6. Evaluate the proposed substructure type, size and location and modify them, if necessary. The overall flood flow pattern at the bridge site should be visualised to assist in identifying those elements of the bridge at risk from scour.
7. Substructure design should be carried out on the basis that all stream bed material above the total scour line (Step 4) has been removed and is not available for bearing or lateral support.

5.3.7 Scour Related to Construction

Removal of vegetation and development of borrow areas upstream of a bridge and its approaches will often result in changes in flow patterns, which may affect the depth and extent of scour along embankments, abutments and piers.

Clearing of vegetation and removal of borrow material should be controlled in the vicinity of bridges and their approach embankments.

5.4 Methods of Estimating Scour

5.4.1 General

There are many methods for estimating scour at bridges. They have all been developed from a limited range of laboratory data, with very little verification in the field. Therefore, all theoretical estimates of scour should be treated with caution and engineering judgement used in applying them. Refer to Arneson et al. (2012) for a full list of primary technical resources and research that has been conducted in recent years in the USA for estimating scour methods.

5.4.2 Design Approach

The steps involved in estimating scour at bridges are as follows:

1. Obtain the fixed-bed channel hydraulics for determining scour analysis variables, including the magnitude of the discharges for the design floods, possible future factors that will produce a combination of high discharge and/or low tailwater control and the water-surface profiles for the discharges.
2. Determine long-term profile degradation or aggradation, taking into account historic records, observational data, or using other empirical methods.
3. Compute the magnitude of the natural contraction scour.
4. Determine the magnitude of local scour at piers.
5. Determine the foundation elevation for abutments.

6. Plot and evaluate the total scour depths, including estimated long-term bed elevation change, contraction scour, and local scour at the piers and abutments. These data are plotted on the cross-section of the stream channel or other general floodplain at the bridge crossing.
7. Re-evaluate the bridge design.
8. Determine the protection required at the abutments.

The design approach is based on the assumption that the scour components develop independently. Thus, the potential local scour is added to the contraction scour without considering the effects of the contraction scour on the channel and bridge hydraulics. However, if the contraction scour is significant, the channel and/or bridge hydraulics should be adjusted for the effects of the contraction scour before estimating the local scour.

5.4.3 Live-bed Contraction Scour

The modified version of Laursen's 1960 equation (Arneson et al. 2012) for live-bed scour at a long contraction can be used to estimate the depth of scour in a contracted section (Equation 33 and Equation 34). These equations assume that bed material is being transported from the upstream section:

$$\frac{y_2}{y_1} = \left[\frac{Q_2}{Q_1}\right]^{0.86} \left[\frac{W_1}{W_2}\right]^{k1} \quad (33)$$

$$y_s = y_2 - y_0 \quad (34)$$

where

y_1 = average depth in the upstream main channel, m
y_2 = average depth in the contracted section, m
y_s = average scour depth, m
y_0 = existing depth in the contracted section before scour (m)
Q_1 = flow in the upstream channel transporting sediment (m^3/s)
Q_2 = flow in the contracted channel (m^3/s)
W_1 = bottom width of the upstream main channel that is transporting bed material (m)
W_2 = bottom width of main channel in contracted section less pier width(s) (m)
k_1 = exponent determined based on the mode of bed material transport (Table 5.1)

Table 5.1: Estimation of exponent k1

V/T*	k_1	mode of bed material transport
< 0.50	0.59	mostly contact bed material discharge
0.50 to 2.0	0.64	some suspended bed material discharge
> 2.0	0.69	mostly suspended bed material discharge

where
*V** = $(\vartheta_0/\Delta)^{1/2} = (g y_1 S_1)^{1/2}$, shear velocity in the upstream section, m/s
T = fall velocity of bed material based on the D50 (m/s) (Figure 5.20)
g = acceleration of gravity (9.81 m/s^2)
S_1 = slope of energy grade line of main channel, m/m
ϑ_0 = shear stress on the bed, N/m^2
Δ = density of water (1000 kg/m^3)

Notes:

- *Q_2 may be the total flow going through the bridge opening as in cases 1a and 1b, but is not the total flow for case 1c (see Section 5.2.7). For case 1c contraction scour must be computed separately for the main channel and the left and/or right overbank areas.*
- *Q_1 is the flow in the main channel upstream of the bridge, not including overbank flows.*

- *The Manning 'n' ratio is eliminated in the Laursen live-bed equation to obtain Equation 34. This was done for the following reasons. The ratio can be significant for a condition of dune bed in the upstream channel and a corresponding plane bed, washed out dunes or anti-dunes in the contracted channel. However, Laursen's equation does not correctly account for the increase in transport that will occur as the result of the bed planning out (which decreases resistance to flow, increases the velocity and the transport of bed material at the bridge). In this cause, Laursen's equation will indicate a decrease in scour; in reality, there would be an increase in scour depth. In addition, at flood flows, a plane bedform will usually exist upstream and through the bridge waterway, and the values of Manning 'n' will be equal. Consequently, the 'n' value ratio is not recommended or presented in Equation 34.*
- *W_1 and W_2 are not always easy to define. It can be acceptable in some cases to use the topwidth of the main channel to define these widths. It is important to be consistent so that W_1 and W_2 refer to either bottom widths or top widths.*
- *The average width of the bridge opening (W_2) is normally taken as the bottom width, with the width of the piers subtracted.*
- *Laursen's equation will overestimate the depth of scour if the bridge is located upstream of a natural contraction or if the contraction is caused by the bridge abutments and piers. However it is currently the best available equation.*
- *The y_0 depth may be approximated by y_1 in sand channel streams where the contraction scour hole is filled in on the falling stage. The depth may be approximated by Sketches or surveys through the bridge can help in determining the existing bed elevation.*
- *Scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material armouring the bed. Scour depths should be calculated for live-bed scour conditions using the clear-water scour equation in addition to the live-bed equation where coarse sediments are present, and that the smaller calculated scour depth be used.*

Figure 5.20: Fall velocity of sand-sized particles

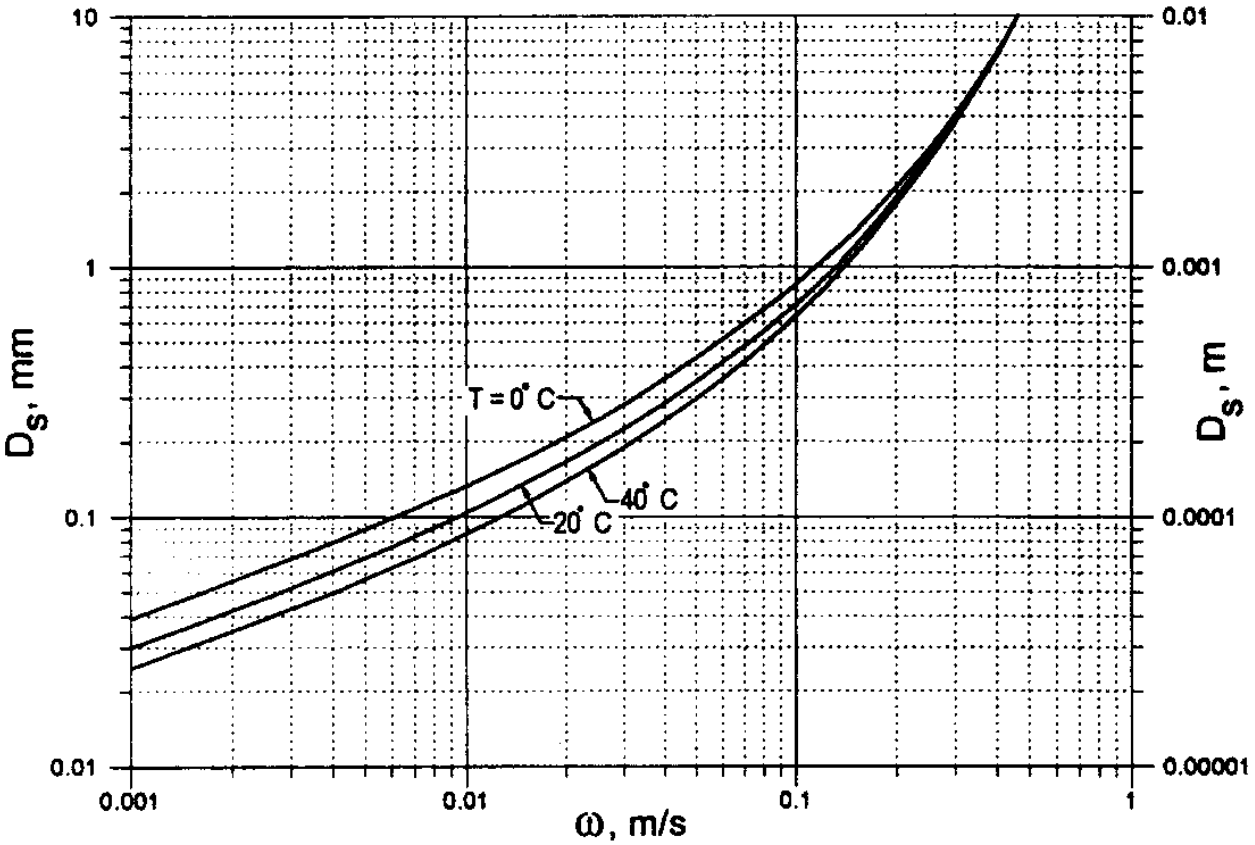


Figure 5.21 and Figure 5.22 defines the terms used for case 1a, and case 1c, respectively.

Figure 5.21: Definition sketch for scour depths for case 1a

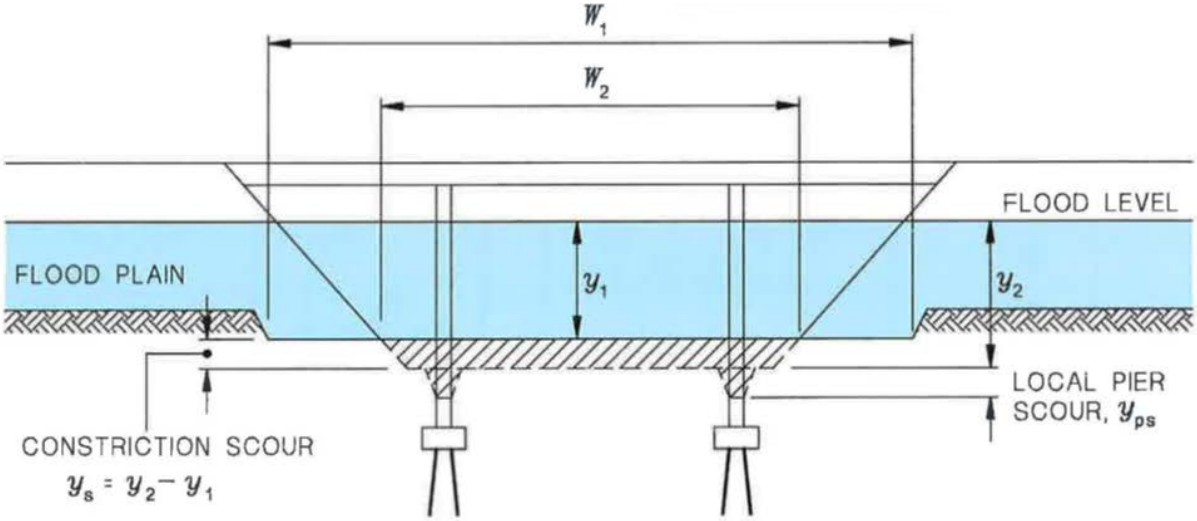
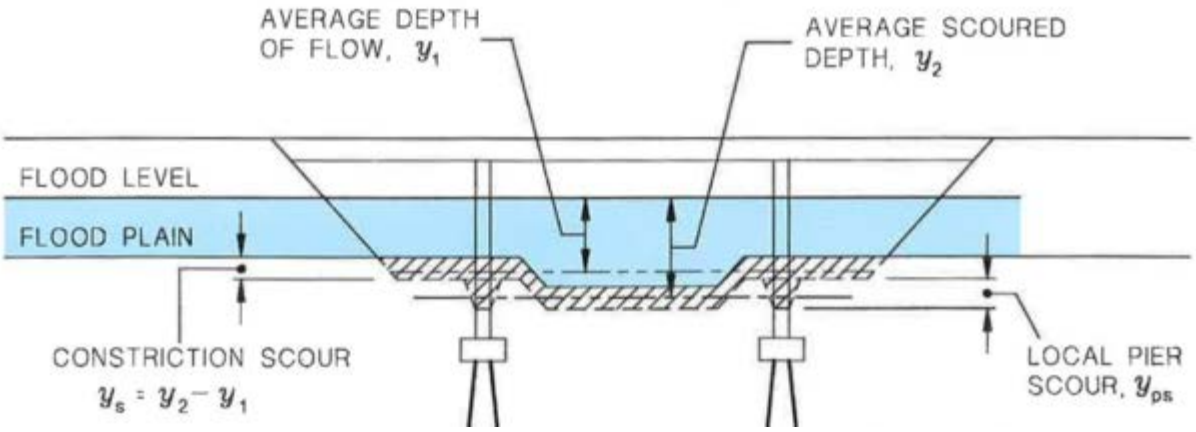


Figure 5.22: Definition sketch for scour depths for case 1c



5.4.4 Clear-water Contraction Scour

The recommended clear-water contraction scour equation is based on a development suggested by Laursen (Arneson et al. 2012). The equation is (Equation 35):

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2}\right]^{3/7} \qquad 35$$

where

y_2	=	average equilibrium depth in the contracted section after contraction scour (m)
Q	=	discharge through the bridge or on the set-back overbank area at the bridge associated with the width W (m^3/s)
D_m	=	diameter of the smallest non-transportable particle in the bed material (1.25 D_{50}) in the contracted section (m)
D_{50}	=	median diameter of bed material (m)
W	=	bottom width of the contracted section less pier widths (m)
W_2	=	bottom width of main channel in contracted section less pier width(s) (m)
K_u	=	0.0077

The average contraction scour depth, y_s, can be determined using Equation 34.

Equation 35 is a rearranged version of Equation 33. D_{50} equal to 0.2 mm is a reasonable lower limit that can be applied to this equation. Using a size smaller than 0.2 mm will over-estimate clear-water contraction scour.

The scoured section can be considered slightly armoured due to the fact that D_{50} is not the largest particle in the bed material. Therefore, the D_m is assumed to be 1.25 D_{50}. The depth of scour for stratified bed material can be determined by using the clear-water scour equation sequentially with successive D_m of the bed material layers.

5.4.5 Contraction Scour with Backwater

The live-bed contraction scour equation is derived assuming a uniform reach upstream and a long contraction into a uniform reach downstream of the bridge. The equation computes a depth after the contraction where the sediment transport in the downstream reach is equal in and out. The clear-water contraction scour equations are derived assuming that the depth at the bridge increases until the shear-stress and velocity are decreased so that there is no longer any sediment transport. With the clear-water equations it is assumed that the flow is uniform. A level water surface is used in both equations to calculate contraction scour depth (Equation 34). A more consistent computation would be to write an energy balance before and after the scour. For live-bed the energy balance would be between the approach section and the contracted section. Whereas, for clear-water scour it would be the energy at the same section before and after the contraction scour.

Backwater can decrease the velocity, shear stress and the sediment transport in the upstream section in extreme cases, increasing the scour at the contracted section. The backwater can, by storing sediment in the upstream section, change live-bed scour to clear-water scour.

5.4.6 Contraction Scour in Cohesive Materials

The live-bed and clear-water contraction scour equations presented in Section 5.4.3 and Section 5.4.4 are developed for cohesionless sediments and provide estimates of scour for a hydraulic conditions sufficient to produce ultimate scour. For silts and clays, the critical shear stress increases due to cohesion (Arneson et al. 2012). The only reliable way of determining critical shear for silt and clay particles is to perform materials testing (refer also to Briaud et al. 2011). Briaud et al. (2011) outlines an equation to compute ultimate scour for cohesive materials, based on laboratory data (Equation 36). This computes the centreline scour downstream of the bridge entrance (scour in the vicinity of the entrance is 35% greater) and assumes that upstream flow depth is equal to the flow depth at the constriction (Equation 37):

$$y_{s-ult} = 0.94y_1\left(\frac{1.83V_2}{\sqrt{gy_1}} - \frac{K_u\sqrt{\frac{\tau_c}{\rho_w}}}{gny_1^{\frac{1}{3}}}\right) \quad (36)$$

$$\tau = \gamma\left(\frac{V_2 n}{K_u}\right)^2 y_o^{-1/3} \quad (37)$$

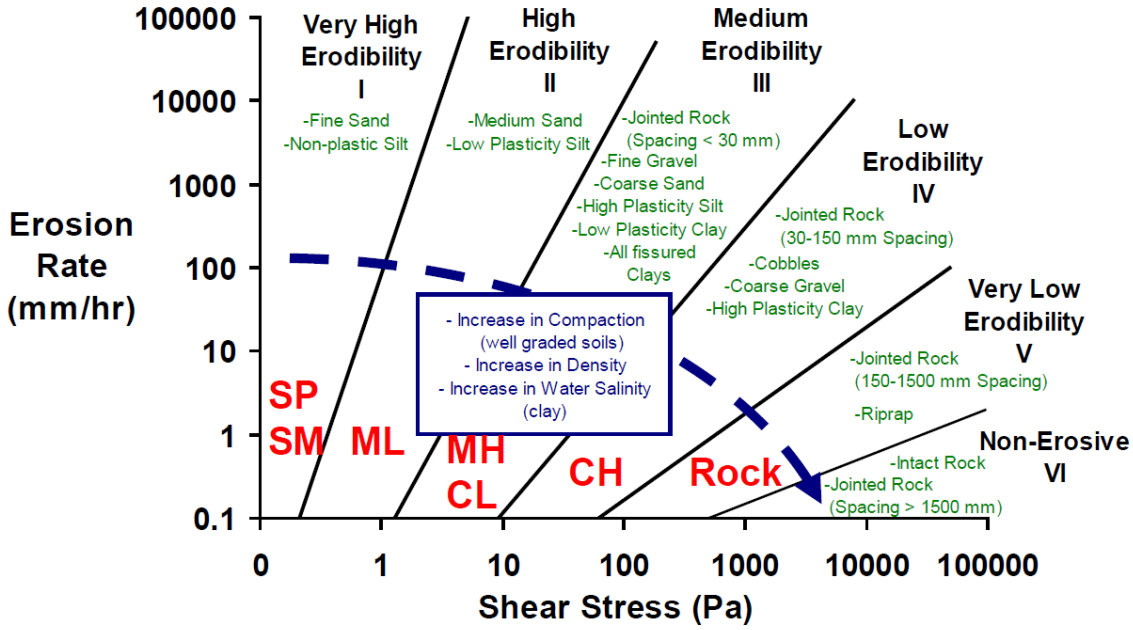
where

y_1	=	upstream average flow depth (m)
V_2	=	average flow velocity in the contracted section (m/s)
τ_c	=	critical shear stress (N/m^2)
ρ_w	=	density of water, (kg/m^3)
n	=	Manning n
K_u	=	1.0
γ	=	specific weight of water (N/m^3)
y_0	=	existing depth in the contracted section before scour (m)

Including cohesion will typically reduce ultimate scour in comparison to fine-sand in the clear-water contraction scour equation.

Ultimate scour may not be reached in the lifetime of a bridge for cohesive materials if there is not sufficient flooding. Further information is required to estimate scour over the life of a bridge, including the erosion rate versus excess shear curve, as well as flow magnitudes and durations. Initial shear stress can be calculated using Equation 37. If the shear stress does not exceed the critical value for the material then no scour will occur during that flow period. If it is exceeded, then the ultimate scour can be calculated using Figure 5.23 from Briaud et al. (2011) which illustrates the generalised relationships between critical shear and erosion.

Figure 5.23: Generalised relationships for scour in cohesive materials



Source: Briaud et al. (2011).

Time rate of scour is also an important consideration for contraction scour in cohesive soils. The actual scour that occurs during the first flood event during the life of the bridge depends on the initial scour rate, ultimate scour for the flow and its duration (see Equation 38):

$$y_s(t) = \frac{t}{\frac{1}{\dot{z}_1} + \frac{t}{y_{s-ult}}} \qquad 38$$

where

$\dot{z}_i$ = initial rate of scour (m/hr)

t = duration of flow (h)

For subsequent flood events, scour will only occur when the ultimate scour of the event exceeds previous scour. This will always occur when the shear exceeds previously occurring shear, but may also occur when it does not. During the life of a bridge, scour in cohesive material is cumulative and can increase even during smaller events that occur after large flood events. Equation 38 can be used to compute scour for subsequent events, provided that the time is adjusted using Equations 39 and 40:

$$t = t_{event} + t_e \qquad 39$$

$$t_e = \frac{y_{s-ult}\, y_{s-prior}}{\dot{z}_1 (y_{s-ult} - y_{s-prior})} \qquad 40$$

where

t_e = equivalent time scour event would have required to reach prior scour (h)

$y_{s-prior}$ = cumulative scour that has been reached in prior flood events (m)

These steps must be completed for all scour events over the life of a bridge. This is due to the fact that the scour a bridge will experience during one flood depends not only on the magnitude and duration of that flood, but also the amount of scour that has occurred in previous floods. Therefore, the sequence of floods will also affect the total scour observed over the life of the bridge. The greatest cumulative scour will occur when floods increase in magnitude over the life of the bridge, and vice versa for the smallest cumulative scour. Even where cumulative scour is not calculated, ultimate scour should be computed for the design flood. The hydraulic engineer should work closely with a geotechnical engineer to fully account for scour in cohesive soils.

5.4.7 Contraction Scour in Erodible Rock

Contraction scour can also occur in erodible rock. Some rock types are vulnerable to weathering and abrasion. In addition to hydraulic forces, channels in rock materials may degrade due to wetting and drying, freeze-thaw, abrasion, and chemical reactions. Some rock, such as weakly-cemented sandstone and other friable rock, may be as erodible as sand while other rock may be extremely erosion resistant. The concepts from the previous section can also be applied to erodible rock; however, it is not only necessary to determine the critical shear and erosion rate information, but also account for other potential factors. In order to fully account for scour in rock, the hydraulic engineer must work closely with a geotechnical engineer and geologist.

5.4.8 Mean Velocity Method

This is an alternative method which uses the concept of cross-sectional velocity as a criterion of contraction scour. It can be used as a check on the contraction scour estimates obtained from Equation 33 for live-bed and clear-water scour for Cases 1, 2 and 4 (Section 5.2.7). The method is as follows:

1. Calculate mean velocity of flow in the unrestricted main channel for the discharge at which contraction scour will just commence.
2. Determine average contraction scour level that will make the mean velocity through the bridge opening equal to the estimated mean velocity calculated in (i) above.

It is assumed that the discharge at which contraction scour will just commence is that of the total waterway design flood. This flood has been chosen as experience has shown that it appears to give realistic estimates of contraction scour. The total waterway design flood can be either the 50 or 100 year ARI (0.02 or 0.01 AEP) flood depending upon the standard adopted by different road agencies (see Section 2.1.2).

It is not anticipated that estimates of contraction scour will vary greatly with the use of either flood, as velocities in the main channel of a stream should be of similar magnitude for both floods.

5.4.9 Scour at Abutments

Where the abutments and roadway embankments obstruct river flow, scour will occur at bridge abutments. Several causes of abutment failures observed during post-flood field inspections of bridge sites include:

- overtopping of abutments or approach embankments
- lateral channel migration or stream widening processes
- contraction scour
- local scour at one or both abutments.

Abutment damage is often caused by a combination of these factors. The abutments most vulnerable to damage are generally those located at or near the channel banks. Large scour holes with depths as much as four times that of the approach floodplain flow depth have been observed where abutments are set back from channel banks, especially on wide floodplains.

Abutment scour depends on the interaction of the flow obstructed by the abutment and roadway approach and the flow in the main channel at the abutment. The discharge returned to the main channel at the abutment is not simply a function of the abutment and roadway length. Abutment scour depth depends on abutment shape, discharge in the main channel at the abutment, discharge intercepted by the abutment and returned to the main channel at the abutment, sediment characteristics, cross-sectional shape of the main channel at the abutment (especially the depth of flow in the main channel and depth of the overbank flow at the abutment), and alignment. In addition, there may be tree-lined or vegetated banks, low velocities, and shallow depths upstream of the abutment.

Various methods have been used in estimating abutment scour in determining the potential depth of scour to aid in the design of the foundation and placement of rock riprap and/or guide banks. Some examples include Froehlich's live-bed scour equation, the HIRE equation in FHWA's HDS 6 (Arneson et al. (2012) and the approach developed under NCHRP Project 24-20 (Ettema, Nakato, & Muste 2010). Froehlich's live-bed scour equation is reproduced below for illustration. Refer to Arneson et al. (2012) for full details.

Froehlich's live-bed scour equation (Equation 41) was established based on the analyses of 170 live-bed scour measurements in laboratory flumes by regression analysis:

$$\frac{y_s}{y_a} = 2.27\, K_1\, K_2\, (\frac{L'}{y_a})^{0.43}\, Fr^{0.61} + 1 \qquad 41$$

where

K_1	=	coefficient for abutment shape
K_2	=	coefficient for angle of embankment to flow: $(\theta/90)^{0.13}$ $\theta < 90^{o}$ if embankment points downstream $\theta > 90^{o}$ if embankment points downstream
L'	=	length of active flow obstructed by the embankment (m)
A_e	=	flow area of the approach cross-section obstructed by the embankment (m^2)
Fr	=	Froude Number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$
V_e	=	velocity at approach cross-section obstructed by the embankment Q_e/A_e (m/s)
Q_e	=	flow obstructed by the abutment and approach embankment (m^3/s)
Y_a	=	average depth of flow on the floodplain (A_e/L) (m)
L	=	length of embankment projected normal to the flow (m)
y_s	=	scour depth (m)

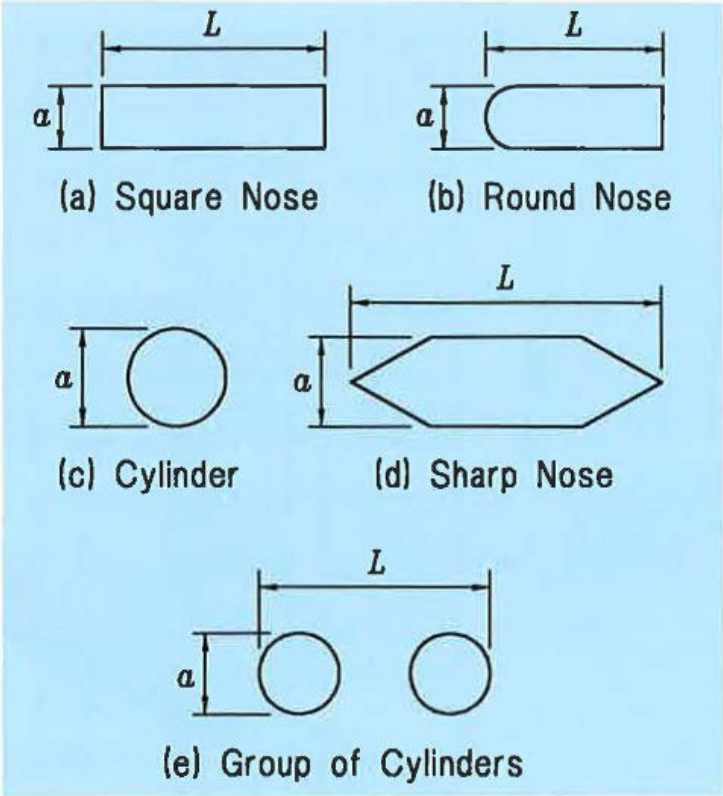
5.4.10 Local Scour at Piers

Local scour at piers is a function of bed material characteristics, bed configuration, flow characteristics, fluid properties, and the geometry of the pier and footing. The bed material characteristics are either granular or non-granular, cohesive or non-cohesive, erodible or non-erodible rock.

In order to determine local pier scour, flow characteristics such as velocity, depth upstream of the pier, angle of attack and whether or not the flow is free surface or pressure need to be determined. The viscosity of the flow is also required (surface tension can be ignored).

The key pier geometry characteristics are its type, dimensions, and shape. Types of piers can include single column, multiple columns, or rectangular; with or without friction or tip bearing piles; with or without a footing or pile cap; footing or pile cap in the bed, on the surface of the bed, in the flow or under the deck out of the flow. Important dimensions are the diameter for circular piers or columns, spacing for multiple columns, and width and length for solid piers. Shapes can include round, square or sharp nose, circular cylinder, group of cylinders, or rectangular (Figure 5.24). In addition, piers may be simple or complex. A simple pier is generally considered to be a single shaft, column or multiple columns exposed to the flow, whereas, a complex pier may have the pier, footing or pile cap, and piles exposed to the flow.

Figure 5.24: Common pier shapes



The HEC-18 pier scour equations (based on the Colorado State University (CSU) equation) are recommended for both live-bed and clear-water pier scour (Equation 42 and Equation 43). Refer to Figure 5.25 for the definition sketch of pier scour.

$$\frac{y_s}{y_a} = 2.0\ K_1\ K_2\ K_3\ (\frac{a}{y_1})^{0.65}\ Fr_1^{0.43} \qquad 42$$

$$\frac{y_s}{a} = 2.0\ K_1\ K_2\ K_3\ (\frac{y_1}{a})^{0.35}\ Fr_1^{0.43} \qquad 43$$

where as a rule of thumb, the maximum scour depth for round nose piers aligned with the flow is:

$y_s \leq 2.4$ times the pier width (a) for $Fr \leq 0.8$

$y_s \leq 3.0$ times the pier width (a) for $Fr > 0.8$

y_s = scour depth (m)

y_1 = flow depth directly upstream of pier

K_1 = correction factor for pier nose shape

K_2 = correction factor for angle of attack of flow

K_3 = correction factor for bed condition

A = pier width (m)

L = length of pier (m)

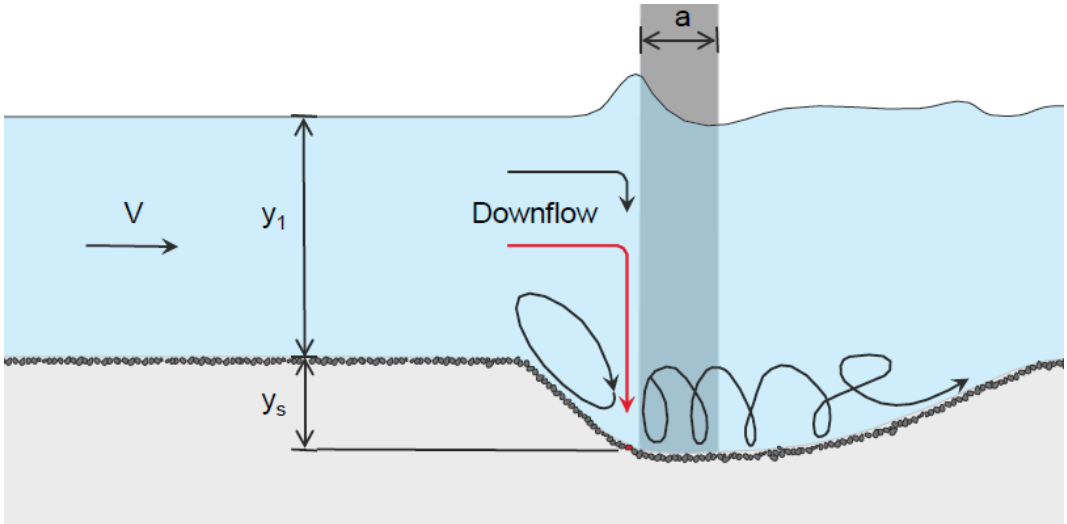
Fr_1 = Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$

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

g = acceleration of gravity, 9.81 (m/s^2)

This equation can be applied to wide pier applications, more complex (3-element) substructure configurations, multiple columns skewed to the flow, estimating scour from debris on piers, and scour in tidal waterways.

Figure 5.25: Definition sketch for pier scour



Source: Arneson et al. (2012).

Note: The correction factor K_1 should be determined using Table 5.2 for flow angle of attack up to 5 degrees. For greater angles, K_2 (Table 5.3) dominates and K_1 should be considered as 1.0. If L/a is larger than 12, use the values of L/a = 12 as a maximum.

Table 5.2: Correction factor, K1 for pier nose shape

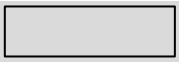
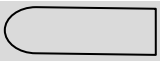
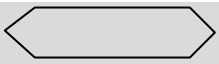
Shape of nose		K_1
(a) Square nose		1.1
(b) Round nose		1.0
(c) Circular cylinder		1.0
(d) Sharp nose		0.9
(e) Group of cylinders		1.0

Table 5.3: Correction factor, K2 for angle of attack of flow

Angle	L/a = 4	L/a = 8	L/a = 12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.5	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Angle = skew angle of flow
L = length (m) of pier
a = width (m) of pier

Top width of scour hole – for practical purposes the top width of a scour hole in cohesionless bed material from one side of a pier or footing can be taken as 2.8 y_s.

Footings and pile caps – where the top of the footing (or pile cap) is at or below the stream bed (after taking into account contraction scour and long-term degradation) it is recommended that the pier width be used for the value of 'a' in the pier scour equation.

Where the footing or pile cap extends above the stream bed, a second computation should be made using the width of the footing (or pile cap) for the value of a and the depth and average velocity in the flow zone obstructed by the footing for the y_1 and V_1 respectively in the scour equation. The larger of the two scour computations should be used. The average velocity of flow at the exposed footing (V_f) should be determined using Equation 44 (Jones 1989):

$$\frac{V_f}{V_1} = \frac{\ln(10.93\,\frac{y_f}{k_s} + 1)}{\ln(10.93\,\frac{y_1}{k_s} + 1)} \qquad 44$$

where

V_f	=	average velocity in the flow zone below the top of the footing (m/s)
V_1	=	mean velocity of approach flow upstream of the pier (m/s)
y_f	=	distance from the bed to the top of the footing
k_s	=	the grain roughness of the bed = D_{84} (m) of the bed material
y_1	=	depth of flow upstream of the pier (m)

Exposed pile groups – pile groups that project above the stream bed (as a result of contraction scour or long-term degradation) can be analysed conservatively by representing them as a single pier width equal to the projected area of the piles, ignoring the clear space between them. Good judgement needs to be used in accounting for debris which may collect on the piles and cause the pile group to act as a much larger obstruction to flow.

If a pile group is exposed to the flow as the result of local scour then it is unnecessary to consider the piles in calculating pier scour.

The correction factor K_1 in Equation 42 for the multiple piles would be 1.0 regardless of the layout of the piles. If the pile group is a square then K_1 would be 1.0. However, if the pile group is a rectangle then the dimensions used would be based on the assumption that they are a single pier and the appropriate *L*/*a* value used for determining K_2.

Multiple columns – for multiple columns skewed to the flow, the scour depth depends on the spacing between the columns. The correction factor K_2 for angle of attack would be smaller than for a solid pier. The pier width, a in Equation 42 would be the total projected width of all the columns in a single bent, normal to the flow angle of attack. The correction factor K_1, for the multiple column would be 1.0 regardless of column shape. If debris is likely to pile up against the multiple columns, it would be logical to consider the multiple columns as a solid elongated pier. In this instance the appropriate *L*/*a* value and flow angle of attack would then be used to determine K_2 from Table 5.3.

Pier scour in cohesive material

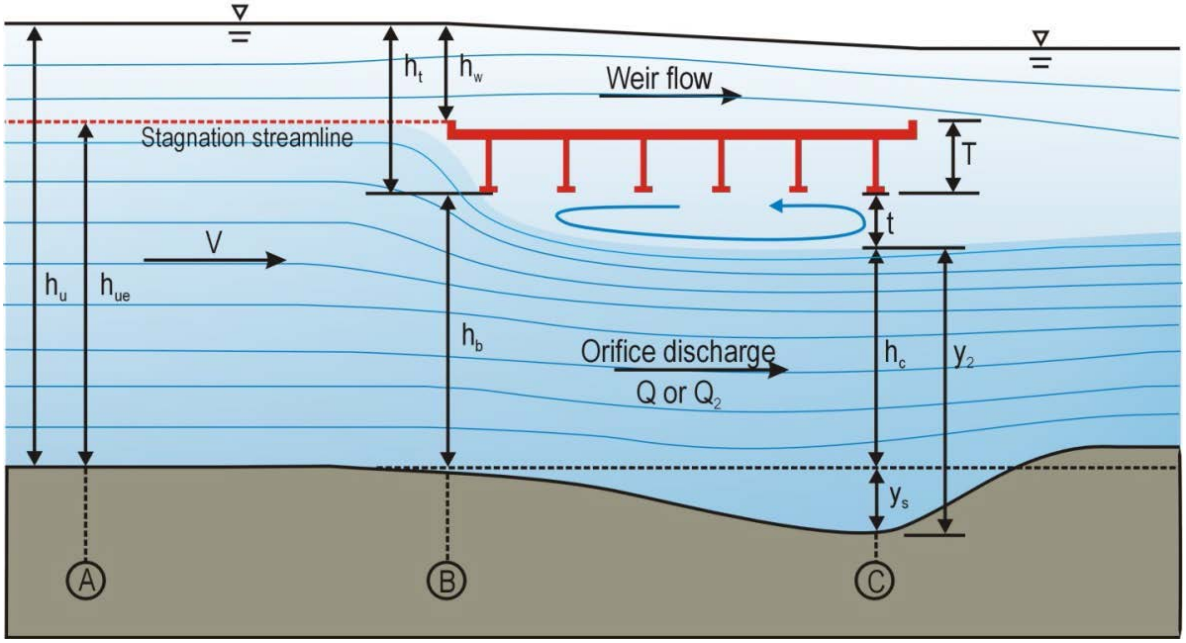
Pier scour in cohesive materials generally progresses more slowly and is more dependent on soil properties than for non-cohesive sediments. These properties include critical velocity, critical shear stress, and the erosion rate for hydraulic conditions that exceed the critical value.

It should be noted that the HEC-RAS software (see Table 3.3) can be used to compute scour estimates for contraction and local scour at piers and abutments (but only for cohesionless soils).

5.4.11 Pressure Flow Scour

Scour in pressurised flow conditions can be estimated as follows. Refer to Figure 5.26 for the definition and geometric parameters.

Figure 5.26: Pressure flow scour at a fully submerged bridge site



Source: Arneson et al. (2012).

The bridge 'superstructure' mentioned in this section refers to a continuous cross-section of the structural and non-structural elements that span the waterway and that can produce significant blockage when it is partially or fully inundated. Discharge under the superstructure can be conservatively assumed to be all approach flow below the top of the superstructure at height $h_b + T$, where h_b is the vertical size of the bridge opening prior to scour and T is the height of the obstruction including girders, deck, and parapet. For floods that do not create overtopping, all discharge upstream goes into the bridge opening.

The depth at the location of maximum scour is comprised of three components:

- h_c – the vertically contracted flow height from the streamline bounding the separation zone under the superstructure at the maximum scour depth
- y_s – the scour depth
- t – the maximum thickness of the flow separation zone (this zone does not convey any net mass from the upstream opening of the bridge to the downstream exit).

The pressure scour depth y_s is determined by using the horizontal contraction scour equations to calculate the height, $y_s + h_c$, required to convey flow through the bridge opening at the critical velocity. This height is equivalent to y_2 (the average depth in the contracted section) in the clear-water contraction scour (Equation 35) and the live-bed contraction scour (Equation 33). Combining this relation with the definitions of t and h_b (Equation 45):

$$y_s = y_2 + t - h_b \qquad 45$$

It should be noted that h_b in pressure flow scour is analogous to y_0 (existing depth in the contracted section before scour) in contraction scour (see Equation 33 and Equation 34). These equations show that the scour depth of pressure flow can be significantly greater than that of non-pressure flow because depth available to convey flow through the opening under the bridge is reduced by the flow separation thickness, *t*. This thickness can be calculated using Equation 46:

$$\frac{t}{h_b} = 0.5\left(\frac{h_b h_t}{h_u^2}\right)^{0.2}\left(1 - \frac{h_w}{h_t}\right)^{-0.1} \qquad 46$$

where

h_b	=	vertical size of the bridge opening prior to scour (m)
h_u	=	upstream channel flow depth (m)
h_t	=	distance from the water surface to the lower face of the bridge girders, equals $h_u - h_b$ (m)
h_w	=	weir flow height, $h_w = h_t - T$ for $h_t > T$, $h_w = 0$ for $h_t \leq T$

5.4.12 Worked Examples

The following worked examples, which are for bridges in rural areas where the backwater resulting from the total waterway design flood is not a governing factor, are included to demonstrate the methods of scour estimation. They are not intended to indicate the backwater that is acceptable in all situations. In more densely populated areas, backwater is commonly limited to 150 mm or less.

1) Bridge over Irwin River on Brand Highway

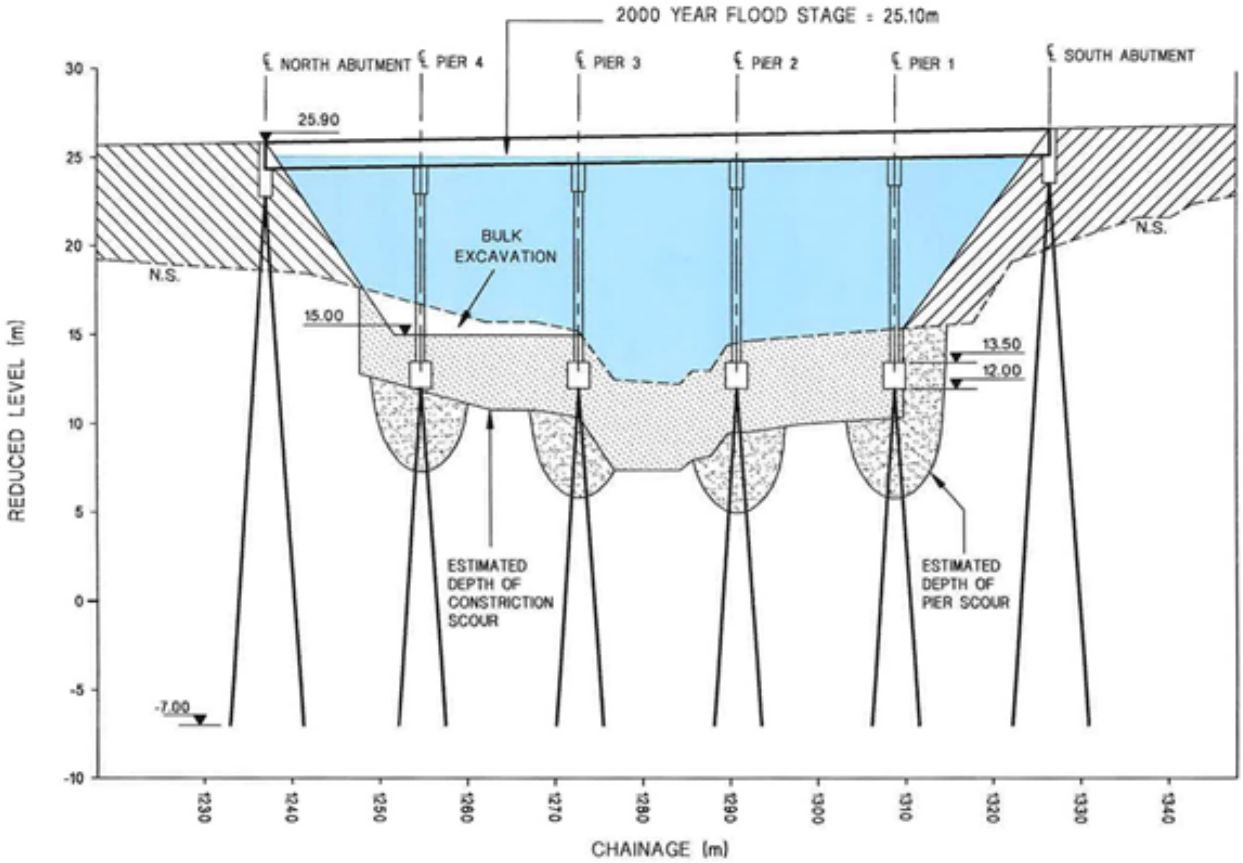
General description of problem – A 90 m long bridge with 1.5:1 spill-through abutments (see Figure 5.27) is to be constructed over an ephemeral stream in the wheat belt of Western Australia. The bridge will be on a grade with a minimum deck level of 25.90 m and have a 1.1 m deep superstructure, which will be continuous over the piers. The four round nosed piers, which will be aligned with the flow and evenly spaced, will be founded on pile caps supported on raked steel piles driven to a level of –7.00 m. The bridge abutments will also be supported on raked steel piles driven to a level of –7.00 m.

The bridge will not be overtopped with the 2000 year ARI (0.05% AEP) flood, which will be used to evaluate the foundation design. The 100 year average recurrence interval flood will be used to design the abutment protection. The 2000 year ARI (0.05% AEP) design flood discharge is 3070 m^3/s at a stage height of 25.10 m and the 100 year design flood discharge is 1870 m^3/s at a stage height of 23.70 m. Floods are flashy with durations generally less than 24 hours.

The 5350 km^2 catchment has been largely cleared for agriculture. This has occurred since about 1950 and has had a large impact on flood discharges. The largest flow experienced at the bridge site, since 1950, occurred in 1971 with a peak flow equal to that of the 100 year event.

The stream has a relatively small main channel and wide floodplains, which are reasonably well vegetated with trees and grass. The bridge crossing is located on a relatively straight reach of channel with uniform geometry upstream and downstream of the bridge site. The main channel contains a very shallow depth of fine sand and there is little bed transport. The soils beneath the shallow depth of sand in the main channel and on the floodplain comprise a mixture of silty and clayey fine to medium sands which become coarser with depth. D_{50} varies between 0.1 and 0.7 mm for the soil to a depth of 2 m and D_{x4} is 1.25 mm.

Figure 5.27: Irwin River Bridge – plot of total scour



Hydraulic characteristics – the distribution of flow in the natural channel (without the bridge) and the bridge hydraulics were determined using the computer program AFFLUX. The details are presented in Table 5.4.

Table 5.4: Irwin River – distribution of flow in natural channel

Distribution of flow	Flow (m^3/s)	Mean velocity (m/s)	Average depth of flow (m)
Q_{2000}			
Bridge opening, Q_b(1)	1610	2.1	10.4
Q_{100}			
Main channel	530	2.93	10.5
Left of bridge, Q_c(1)	690	1.37	–
Bridge opening, Q_b(1)	1145	1.98	7.82
Right of bridge, Q_a(1)	35	0.7	–

1 *See Figure 5.4.*

Contraction scour will occur in the main channel and on the left and right overbank of the bridge opening (Case 1c on Figure 5.5). Contraction scour will be clear-water scour both in the main channel and both overbanks.

Table 5.5: Irwin River – bridge hydraulics

Flow (m3/s)	Scour depth (m)	Mean velocity (m/s)	Average depth(1) of flow (m)	Backwater, h_1^* (m)
Q_{2000} = 3070	0.0	4.14	9.80	1.39
	1.0	3.82	9.80	1.19
	2.0	3.55	9.80	1.03
	3.0	3.31	9.80	0.90
	4.0	3.10	9.80	0.79
	5.0	2.93	9.80	0.70
Q_{100} = 1870	0.0	3.18	8.00	0.96
	1.0	2.72	8.00	0.70
	1.5	2.37	8.00	0.54
	2.0	2.11	8.00	0.42

1 *Without scour.*

Step 1: Long-term bed elevation changes

Clearing for agriculture has resulted in a considerable loss of top soil from the catchment, which has caused aggradation of the stream bed in its lower reaches. However, this is not a problem at the bridge site and the channel is relatively stable. The 1971 flood event did not cause any change in the alignment of the stream channel or scour to occur at the bridge site.

Step 2: Estimate magnitude of contraction scour

(a) For the 2000 year ULS flood

y_1 = 10.40 m

Q_1 = 1610 m^3/s

Q_2 = 3070 m^3/s.

For clear-water scour, the last part of Equation 33 can be ignored, and

$$\frac{y_2}{10.40} = (\frac{3070}{1610})^{0.86} = 1.74$$

$$y_2 = 1.74 \times 10.40 = 18.10 \text{ m}$$

And depth of contraction scour, $y_s = 18.10 - 10.40 = 7.70$ m.

This appears too large. Try the Mean Velocity Method (Section 5.4.8) utilising the average velocity of 2.93 m/s in the main channel that resulted from the 1971 (100 year) flood event. As noted earlier this flow did not cause any significant scour of the main channel or floodplains. From Table 5.5 it can be seen that a scour depth of 5 m is required to reduce the velocity of flow through the bridge to 2.93 m/s. Although this depth of scour also appears high, it is accepted as the bridge will be supported on piles driven to a level of −7.0 m, some 20.20 m below the average level of the bed of the main channel.

(b) For the 100 year flood for the design of protection works

y_1 = 7.82 m

Q_1 = 1 145 m^3/s

Q_2 = 1 870 m^3/s

Ignoring the last part of Equation 33:

$$\frac{y_2}{7.82} = (\frac{1870}{1145})^{0.86} = 1.52$$

$$y_2 = 1.52 \times 7.82 = 11.89 \text{ m}$$

And depth of contraction scour, $y_s = 11.89 - 7.82 = 4.07$ m.

This appears too large. With the Mean Velocity Method the scour required to reduce the velocity to 2.93 m/s is about 0.5 m. It would be conservative to assume scour will be of the order of 2 m.

Step 3: Estimate magnitude of local pier scour

(a) For the 2000 year ULS flood

Estimate local scour for pier 2, which has the lowest ground level of 14.50 m, using Equation 42 with the bridge hydraulics adjusted for a contraction scour depth of 5 m:

y_1 = 25.10 – 14.50 + 5.00 = 15.6 m

K_1 = 1.0 for a round nose (see Table 5.2)

K_2 = 1.0 for angle of attack of zero degrees (see Table 5.3)

a = width of pier = 1.00 m

V_1 = 2.93 m/s

F_{r1} = 2.93/(9.80 x 15.6)$^{0.5}$ = 0.24

$$\frac{y_{ps}}{15.6} = 2.0 \times 1.0 \times 1.0\ (\frac{1.00}{15.6})^{0.65}\ 0.24^{0.43} = 0.18$$

$$y_{ps} = 15.6 \times 0.18 = 2.81 \text{ m}$$

As the pile cap will be exposed with a contraction scour depth of 5 m, Equation 42 will be used to estimate the average velocity in the flow zone below the top of the pile cap and V_f and the local scour recalculated.

Distance from scoured bed to top of pile cap, y_f = 4.10 m.

Grain roughness of the bed, *k*, = 0.00125 m.

$$\frac{V_f}{2.93} = \frac{\ln(10.93\ \frac{4.10}{0.00125} + 1)}{\ln(10.93\ \frac{15.7}{0.00125} + 1)} = 0.89$$

$$V_f = 2.93 \times 0.89 = 2.61 \text{ m/s}$$

$$F_{r1} = 2.61 \div (9.80 \times 4.10)^{0.5} = 0.41$$

a = width of pile cap = 2.50 m

$$\frac{y_{ps}}{4.10} = 2.0 \times 1.0 \times 1.0\ (\frac{2.5}{4.10})^{0.65}\ 0.41^{0.43} = 0.99$$

$$y_{ps} = 4.10 \times 0.99 = 4.06 \text{ m}$$

Local pier scour is the largest of the two estimates = 4.06 m.

(b) For the 100 year flood for the design of protection works

Estimate local scour for pier 4 adjacent to the right (north) abutment, assuming ground level at this abutment has been excavated to a level of 15.00 m (see Step 5). Conservatively make no adjustment to bridge hydraulics for contraction scour.

y_1= 23.70 – 15.00 = 8.70 m

K_1 = 1.0 for a round nose (see Table 5.2)

K_2 = 1.0 for angle of attack of zero degrees (see Table 5.3)

a = width of pier = 1.00 m

V_1 = 3.18 m/s

$$F_{rl} = 3.18 \div (9.80 \times 8.70)^{0.5} = 0.34$$

$$\frac{y_{ps}}{8.7} = 2.0 \times 1.0 \times 1.0 \times \left(\frac{1.0}{8.7}\right)^{0.65} \times 0.34^{0.43} = 0.31$$

$$y_{ps} = 0.31 \times 8.7 = 2.70 \text{ m}$$

Step 4: Plot and evaluate total scour depths, and evaluate foundation design

The depths of contraction scour and local pier scour are plotted on Figure 5.27. They appear excessive, but as the bridge is supported on piles driven about 20 m below the bed level, there is no doubt that the bridge will survive the 2000 year ultimate limit stat flood. It should also be noted that the left (north) approach embankment will be overtopped and breached before the bridge is overtopped, which will reduce the flow through the bridge and limit the scour that might occur.

Step 5: Determine protection required at abutments

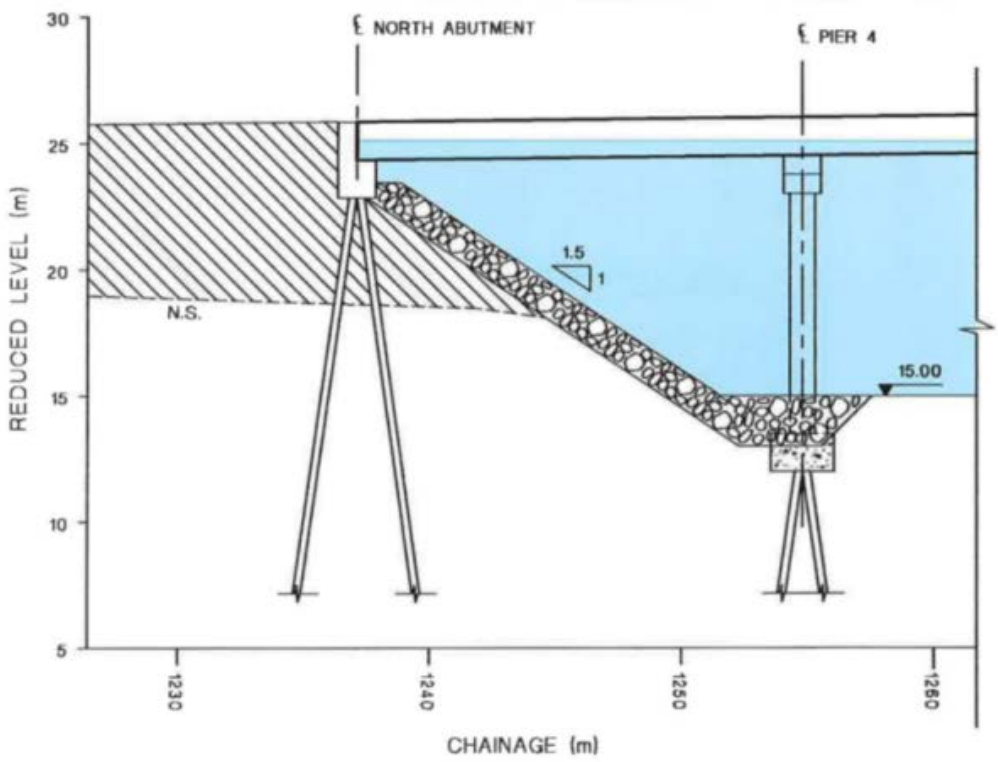
Because a large proportion of the total flow is cut off by the approach embankment at the left (north) abutment, a guide bank will be provided to limit the depth of scour. However, some local abutment scour will occur in conjunction with the contraction scour and local pier scour at pier 4, which is close to the toe of the spill-through abutment. The total depth of scour could be of the order of 5 to 6 m. Protection to this depth can be conservatively achieved by carrying out bulk excavation under the bridge to a level of 15.00 m (see Figure 5.27) and providing sufficient rock protection to protect pier 4 from scour. That is sufficient rock to protect the face of the spill-through abutment to a level of 10.00 m (see Figure 5.28). The same rock protection will be provided at the left abutment to limit the scour at pier 1.

Comment: It is not practical to place the top of the pile caps at the level of the contraction scour, because of problems with piling at this depth. Failing this it would be preferable to place the top of the pile caps at the same level as the thalweg. However, the water table at the site is at a level of 12.00 m at the end of summer and the bottom of the pile caps have been set at this level to make construction of the bridge easier. The banks of the main channel will be re-established with gabions when construction of the bridge is complete.

2) Bridge over Yule River on North West Coastal Highway

General description of problem – This is an existing bridge in the Pilbara Region of Western Australia, which was constructed in 1971 and has inadequate waterway capacity. An investigation into possible scour will be carried out to compare measured depths of scour with those predicted by the recommended methods.

Figure 5.28: Irwin River Bridge – rock protection at left (north) abutment



The bridge is 360 m long with 1.5:1 spill-through abutments (see Figure 5.28), has a deck level of 43.70 m and a 1.0 m deep superstructure, which is continuous over the piers. The 19 piers are evenly spaced and aligned with the flow. Every fourth pier is a strong pier comprising a solid 0.6 m wide wall with a round nose founded on a pile cap supported on raked steel piles. The intermediate piers comprise two circular columns founded on pile caps supported on vertical steel piles. The bridge abutments are also supported on vertical steel piles.

The stream, which has a catchment area of 7980 km^2 is ephemeral and is subject to flooding as a result of tropical cyclones. The overtopping flood, which is smaller than the 100 year flood event, has a discharge of 8670 m^3/s at a stage height of 42.45 m. The stream has a wide main channel lined with flood gums and wide floodplains with sparse vegetative cover. On the west side of the bridge is a secondary channel which is blocked off by the bridge approach embankment. The bridge crossing is located on a relatively straight reach of channel with uniform geometry upstream and downstream of the bridge site.

The main channel has a coarse sand bed, overlying a layer of sandy clay with some coarse gravel and cobbles, which in turn overlays a further layer of hard sandy clay into which the steel piles are driven. D_{50} of the sand is 0.8 mm and D_{84} is 1.8 mm. The soil on the floodplains is pindan (loam).

Hydraulic characteristics – the distribution of flow in the natural channel (without the bridge) and the bridge hydraulics were determined for the overtopping flood (Q = 8670 m^3/s) using the computer program AFFLUX. The details are shown in Table 5.6 and Table 5.7.

Table 5.6: Yule River – distribution of flow in natural channel

Distribution of flow	Flow (m^3/s)	Mean velocity (m/s)	Average depth of flow (m)
Main channel	7880	2.52	5.69
Left of bridge opening (Q_c[1])	1760	2.30	–
Bridge opening (Q_b[1])	4940	2.56	5.80
Right of bridge opening (Q_a[1])	1970	1.66	–

1 *See Figure 5.4.*

Table 5.7: Yule River – bridge hydraulics

Scour depth (m)	Mean velocity (m/s)	Average depth (1) of flow (m)	Backwater, h_1^* (m)
0.0	4.37	5.67	1.13
1.0	3.69	5.67	0.81
2.0	3.18	5.67	0.61
3.0	2.80	5.67	0.47
4.0	2.50	5.67	0.38
5.0	2.26	5.67	0.31

1 *Without scour.*

Live-bed scour will occur in the main channel. Although the bridge constricts the main channel (case 1a in Figure 5.5), the flow cut off by the approach embankments is carrying very little bed-load and the abutment scour will be clear-water scour. This is confirmed by measurements taken after major flood events, which show that there is a scour hole adjacent to each abutment when flow ceases.

Pier scour estimates will be made for one of the strong piers and one of the intermediate piers. The strong piers have their pile caps set 1.5 m higher than the intermediate piers.

Step 1: Long-term bed elevation changes

Although measurements of the bed in the vicinity of the bridge show that after each major flood event the bed is lower than the original bed level, it is not anticipated that degradation of the bed will be a long-term problem.

Step 2: Estimate magnitude of contraction scour

Using the overtopping flood and Equation 33.

y_1 = 5.69 m

Q_1 = 7880 m^3/s

Q_2 = 8670 m^3/s

W_1 = 550 m

W_2 = 323 m

S_1 = 0.0015

V_* = (9.80 x 5.69 x 0.0015)$^{0.15}$ = 0.29 m/s

w = 0.14 m/s for D_{50} = 0.8 mm

V_*/w = 0.29/0.14 = 2.07 – mostly suspended bed material discharge

k_1 = 0.69

$$\frac{y_2}{5.69} = (\frac{8670}{7880})^{0.86}(\frac{550}{323})^{0.69} = 1.57$$

$$y_2 = 1.57 \times 5.69 = 8.93 \text{ m}$$

And depth of contraction scour, $y_s = 8.93 - 5.69 = 3.24$ m.

Check estimate utilising the Mean Velocity Method. From Table 5.7 it can be seen that a scour depth of 4 m is required to reduce the velocity in the bridge opening to the average velocity of 2.52 m/s (which is less than the velocity with the 100 year ARI or 1% AEP flow) in the upstream main channel. A contraction scour depth of 4.00 m is accepted as a conservative estimate.

Step 3: Estimate magnitude of local pier scour

Using the overtopping flood and Equation 42, with the bridge hydraulics adjusted for a contraction scour depth of 4.0 m:

Pier at a stream bed level of 36.94 m

y_1 = 42.45 – 36.94 + 4.00 = 9.51

K_1 = 1.0 for a round nose

K_2 = 1.0 for angle of attack of zero degrees

a = width of pier = 0.6 m

V_1 = 2.50 m/s

$$F_{rl} = 2.50 \div (9.80 \times 8.70)^{0.5} = 0.34$$

$$\frac{y_s}{9.51} = 2.0 \times 1.0 \times 1.0 \times \left(\frac{0.60}{9.51}\right)^{0.65} \times 0.26^{0.43} = 0.19$$

And depth of contraction scour, $y_s = 0.19 \times 9.51 = 1.81$ m.

Strong pier – the pile cap will be exposed with a contraction scour depth of 3.24 m. Hence, the average velocity in the flow zone below the top of the pile cap, V_f will be estimated using Equation 44 and the local pier scour recalculated.

Distance from scoured bed to top of pile cap, y_t = 4.55 m

Grain roughness of the bed, k_s = 0.0018 m

$$\frac{V_f}{2.50} = \frac{\ln(10.93\ \frac{4.55}{0.0018} + 1)}{\ln(10.93\ \frac{9.51}{0.0018} + 1)} = 0.94$$

$$V_f = 0.93 \times 2.50 = 2.33\ m/s$$

$$F_{r1} = 2.33 \div (9.80 \times 4.55)^{0.5} = 0.35$$

$$\frac{y_s}{4.55} = 2.0 \times 1.0 \times 1.0 \times \left(\frac{0.90}{4.55}\right)^{0.65} \times 0.35^{0.43} = 0.44$$

$$y_{ps} = 0.44 \times 4.55 = 2.00 \text{ m}$$

Intermediate pier – the pile cap will also be exposed with the contraction scour. Using Equation 44, V_f will be calculated and the local pier scour recalculated.

Distance from scoured bed to top of pile cap, y_t = 3.05 m

$$\frac{V_f}{2.50} = \frac{\ln(10.93\ \frac{3.05}{0.0018} + 1)}{\ln(10.93\ \frac{9.51}{0.0018} + 1)} = 0.90$$

$$V_l = 0.90 \times 2.50 = 2.24 \text{ m/s}$$

$$F_{r1} = 2.50 \div (9.80 \times 3.05)^{0.5} = 0.46$$

$$\frac{y_{ps}}{3.05} = 2.0 \times 1.0 \times 1.0 \times \left(\frac{0.90}{3.05}\right)^{0.65} \times 0.46^{0.43} = 0.65$$

$$y_{ps} = 0.65 \times 3.05 = 1.98 \text{ m}.$$

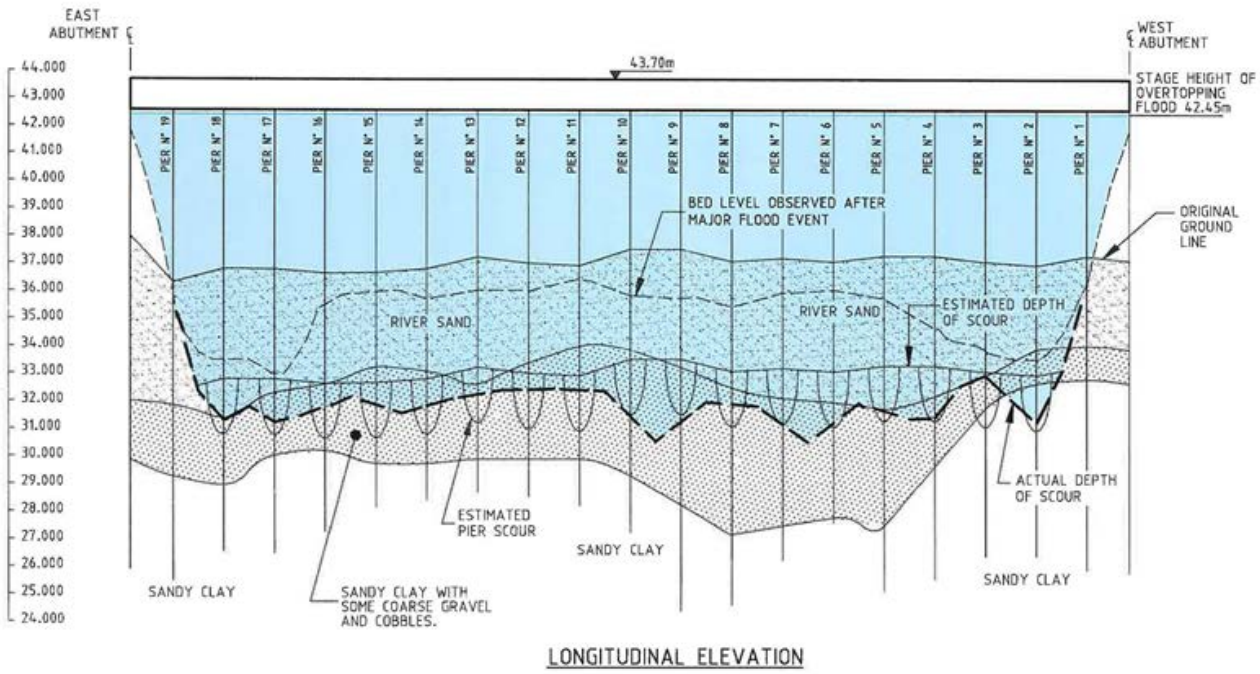
A depth of 2.00 m is accepted as the scour caused by both the strong piers and intermediate piers.

Step 4: Comparison of measured depth of scour with estimated depth

Figure 5.29 shows the estimated depths of scour compared with the measured depths, together with the soil profile under the bridge and the bed level observed after major flood events. The actual depth of scour has been ascertained by drilling to determine the extent of scour in the sandy clay layer, this material having been replaced by river sand. The drilling was carried out between each pier on the bridge centre line and at the upstream ends of piers numbers 2, 3, 4, 17 and 18. The measured scour has resulted from a number of major flood events including a flood which overtopped the bridge and its approaches. This flood is equivalent to the overtopping flood for which the scour estimates have been made.

It can be seen from Figure 5.29 that the estimated total depth of scour is generally greater than the measured depth of scour. In addition, the depth of scour at the abutments is not significantly greater than the scour over the length of the bridge. Because of the large proportion of flow cut off by the bridge approaches, it could be anticipated that the scour would be deeper at the abutments, as indicated by the level of the sand bed after major flood events. The scour in the vicinity of pier 3, adjacent to the right (west) abutment, has almost certainly been limited by the presence of larger bed material, but this does not appear to be the case at the left (east) abutment. However, it is possible that scour has been limited in the bridge opening by armouring of the scour hole. It is also possible that the depth of scour will increase with further significant flood events.

Figure 5.29: Yule River Bridge – estimated and measured depths of scour



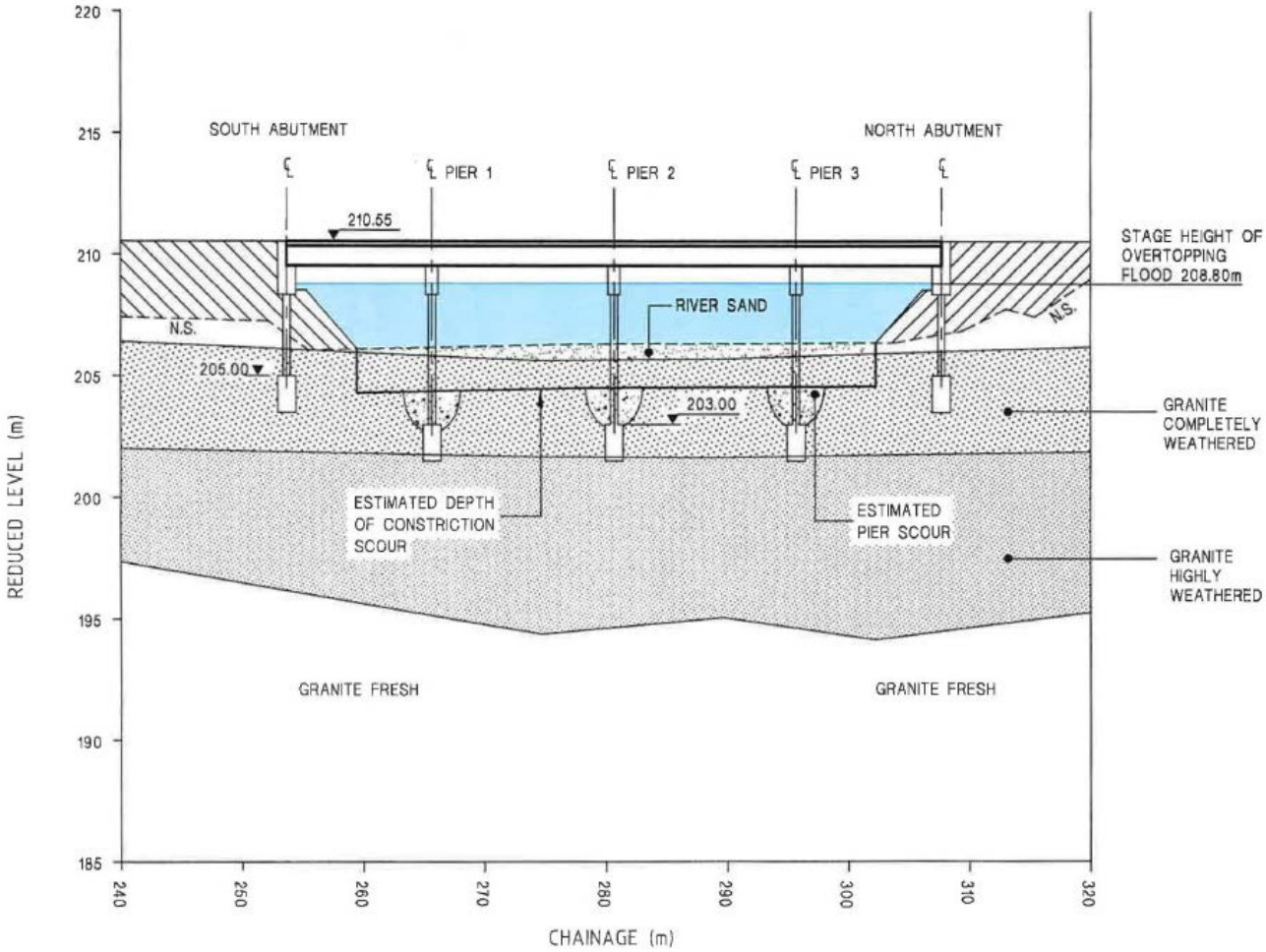
Both abutments have rock protected guide banks that have caused the concentrated flow to be directed away from the abutments. This has resulted in the scour holes being formed sufficiently far from the abutments to ensure that the rock protection has not been undermined and failure has not occurred.

Comment – Because the pier columns are relatively stiff compared with the piles and the piles are in danger of buckling under the combination of live load and temperature movement, rock riprap mats have been placed in the stream bed at both abutments to ensure lateral support of the piles.

3) Bridge over Little McPhees Creek on the Great Northern Highway

General description of problem – A 54 m long bridge with 1.5:1 spill-through abutments is to be constructed over an ephemeral stream in the Kimberley Region of Western Australia (Figure 5.30). The bridge will have a deck level of 210.55 m and a 1.0 m deep superstructure, which will be continuous over the piers. The three piers, which will be aligned with the flow and evenly spaced, will comprise three 0.6 m diameter columns founded on spread footings.

Figure 5.30: Little McPhees Creek Bridge – plot of total scour



The bridge will be overtopped with a flood with an ARI less than 500 years (an AEP of greater than 0.2%). This overtopping ULS flood will be used to evaluate the foundations and the 100 year average recurrence interval flood will be used to design the abutment protection. The overtopping flood has a discharge of 510 m^3/s at a stage height of 208.80 m and the 100 year design flood discharge is 405 m^3/s at a stage height of 208.60 m.

The stream has a catchment area of 30 km^2 and has a well-defined channel with floodplains on either side. The bridge crossing is located on a slight bend, with a moderately uniform channel upstream and downstream of the bridge site. The main channel contains a very shallow depth of coarse sand overlying weathered granite. On the floodplains there is a shallow depth of loam over weathered granite. The granite is distinctly weathered at the surface and readily breaks down into a quartz gravel. Weathering decreases with depth until fresh granite is reached at a depth of about 12 m below bed level. D_{50} is about 3 to 5 mm for the quartz gravel at the surface of the completely weathered granite.

Hydraulic characteristics – the distribution of flow in the natural channel (without the bridge) and bridge hydraulics were determined for the overtopping flood and 100 year design flood using the computer program AFFLUX. The details are shown in Table 5.8.

Table 5.8: Little McPhees Creek – distribution of flow in natural channel

Distribution of flow	Flow (m^3/s)	Mean velocity (m/s)	Average depth of flow (m)
$Q_{overtopping}$			
Main Channel	325	2.94	2.52
Q_{100}			
Main channel	290	278	2.31
Left of bridge, Q_c(1)	50	1.45	–
Bridge opening, Q_b(1)	280	2.78	2.31
Right of bridge, Q_a(1)	75	0.67	–

1 *See Figure 5.4.*

The bridge slightly constricts the main channel (case 1a in Figure 5.5). Because there is very little sand in the bed of the main channel, it is anticipated that scour will be predominately clear-water in the bridge opening.

Local pier scour will be estimated for one pier only.

Step 1: Long-term bed elevation changes

It is not anticipated that any bed elevation changes will occur.

Step 2: Estimate magnitude of contraction scour

(a) For the overtopping ULS flood

y_1 = 2.52 m

Q_1 = 325 m^3/s Q_2 = 510 m^3/s

For clear-water scour, the last part of Equation 33 can be ignored, and:

$$\frac{y_2}{2.52} = (\frac{510}{280})^{0.86} = 1.67$$

$$y_2 = 1.67 \times 2.52 = 4.21 \text{ m}$$

And depth of contraction scour, $y_s = 4.21 - 2.52 = 1.69$ m.

Try Mean Velocity Method utilising the average velocity of 2.78 m/s in the main channel for the 100 year design flood. From Table 5.9 it can be seen that a scour depth of about 1.8 m is required to reduce the velocity to 2.78 m/s. Accept 1.8 m as the depth of contraction scour.

Table 5.9: Little McPhees Creek – bridge hydraulics

Flow (m^3/s)	Scour depth (m)	Mean velocity (m/s)	Average depth[(1)] of flow (m)	Backwater, h_1^* (m)
$Q_{overtopping}$ = 500	0.0	4.43	2.52	1.74
	1.0	3.33	2.52	1.05
	2.0	2.65	2.52	0.70
	3.0	2.20	2.52	0.51
	4.0	1.89	2.52	0.38
Q_{100} = 410	0.0	3.88	2.31	1.15
	1.0	2.84	2.31	0.67
	2.0	2.23	2.31	0.44
	3.0	1.84	2.31	0.31

1 *Without scour.*

(b) For the 100 year flood for the design of protection

y_1 = 2.31 m

Q_1 = 280 m^3/s

Q_2 = 405 m^3/s

Ignoring the last part of Equation 33:

$$\frac{y_2}{2.31} = (\frac{405}{280})^{0.86} = 1.37$$

$$y_2 = 1.37 \times 2.31 = 3.16 \text{ m}$$

And depth of contraction scour, $y_s = 3.16 - 2.31 = 0.85$ m.

With the Mean Velocity method the scour required to reduce velocity to 2.78 m/s is about 1.1 m.

Step 3: Estimate magnitude of local pier scour

For the overtopping ULS flood, estimate local scour for pier 1 (on the outside of the bend) with the lowest natural ground level of 206.12 m, with the bridge hydraulics adjusted for a contraction scour depth of 1.80 m:

y_1 = 208.80 – 206.12 + 1.80 = 4.48 m

K_1 = 1.0 for a round column (see Table 5.2)

K_2 = 1.0 for piers aligned with flow and angle of attack of zero degrees (see Table 5.3)

a = width of column = 0.6 m

V_1 = 2.94 m/s for a scour depth of 1.80 m

$$F_{r1} = 2.94 \div (9.80 \times 4.48)^{0.5} = 0.44$$

$$\frac{y_{ps}}{4.48} = 2.0 \times 1.0 \times 1.0 \times \left(\frac{0.60}{4.48}\right)^{0.65} \times 0.44^{0.43} = 0.38$$

$$y_{ps} = 0.38 \times 4.48 = 1.70 \text{ m}$$

Step 4: Plot and evaluate total scour depths, and evaluate foundation design

The depths of contraction scour and local pier scour are plotted on Figure 5.30. From this plot it can be seen that the top of the footings need to be 3.50 m (1.8 + 1.7 m) below the stream bed level. This appears to be excessive given that the weathering of the granite decreases with depth and this will tend to limit the scour that can occur.

Given the uncertainties in estimating the scour depths, the top of the footings will be placed 3.12 m below the lowest point in the stream bed at a level of 203.00 m. The footings will be cast against the sides of the excavation in the weathered rock and the excavation above the footings backfilled with rock riprap. The top of the abutment footings will be placed at a level of 205.00 m.

Step 5: Determine protection required at abutments

Rock protected guide banks will be provided at both abutments. Sufficient rock will be provided at the toe of the spill through abutment and guide bank to protect against a depth of scour of 1.5 m, which is slightly greater than the depth required to achieve a velocity of 2.78 m/s (see Figure 5.28) through the bridge opening.

5.5 Scour Countermeasures

5.5.1 Introduction

Scour countermeasures are those works incorporated into an existing stream crossing to monitor, control, inhibit or minimise stream stability problems and bridge scour to make a bridge less vulnerable to damage or failure from scour. In many cases, the best countermeasure is appropriate design that avoids causing stream instability. The alternatives available for protecting an existing bridge from scour are listed below, roughly in order of increasing cost:

- providing rock protection at piers and abutments
- constructing or lengthening guide banks
- constructing channel improvements
- strengthening bridge foundations
- constructing sills or drop structures
- constructing relief bridges or lengthening existing bridges.

A guide for providing rock protection at abutments and piers follows. For the other alternatives, reference should be made to the relevant sections of this Guide.

All bridges over streams subject to scour should be regularly inspected to ensure that they are not at risk of damage or failure as a result of scour. Where a bridge is found to be vulnerable to scour it should be closely monitored and consideration given to closing it until scour countermeasures are installed. In some instances it may be appropriate to install interim or temporary measures to protect the bridge and public until suitable long-term countermeasures can be put in place.

Over the last several decades, a wide variety of countermeasure structures, armouring materials and monitoring devices have been used at to mitigate scour and stream stability problems at existing bridges. While standard countermeasures such as riprap will be familiar to most bridge inspectors and engineers, it is unlikely they are knowledgeable of the full spectrum of countermeasures available and in use. A list of countermeasures and selection criteria is presented in Table 5.10.

FHWA's HEC-23, *Bridge Scour and Stream Instability Countermeasures* (Lagasse et al. 2009) consists of a two volume manual devoted to scour countermeasures. It serves as the best available reference for scour counter measures. While this wealth of information is a useful reference to the engineer, many of the measures are not likely to be implemented in Australia.

5.5.2 Countermeasure Groups and Characteristics

Options for structural modifications such as replacement or foundation strengthening of scour susceptible bridges are limited and expensive, as they are already in place. Unless these bridges are programmed for replacement, their continued operation will ultimately require the design and installation of a scour countermeasure. Figure 5.31 is an example of bridge scour countermeasures having been installed as part of emergency repair works.

Figure 5.31: Various protection measures used in repair works

Source: TMR (2013).

Riprap is, and will remain, one of the primary scour countermeasures to resist local scour forces at abutments of typical bridges. Riprap is generally abundant, inexpensive and requires no special equipment, but proper design and placement is still essential. An adequate hydraulic opening must be maintained when designing riprap countermeasures. Improperly placed riprap may reduce the hydraulic opening significantly and create contraction scour problems. If placed improperly, riprap can increase local scour forces. The following countermeasures can be considered as alternatives to riprap:

Armouring countermeasures

Armouring countermeasures include:

- gabion boxes/rock mattresses
- sack gabions
- grouted riprap
- grout-filled mats
- articulating concrete blocks.

River training countermeasures

River training structures alter stream hydraulics to mitigate undesirable erosional and/or depositional conditions. They are commonly used on unstable stream channels to redirect stream flows to a more desirable location through the bridge. River training options include:

- spurs (both permeable and impermeable)
- bendway weirs
- guide banks
- drop structures and check dams.

Concrete retaining wall mass blocks has been used in conjunction with riprap for abutment repairs in certain conditions – especially for remedial/countermeasure applications where abutment slopes are steep and a good foundation is available. The concrete mass blocks are capable of handling high current velocities[3]. Figure 5.32 showed some examples of recent abutment repairs using the mass block/riprap solution.

Figure 5.32: Examples of recent repairs using concrete mass blocks

Collards Creek No 2 Bridge on Dawson Highway

Collards Creek No 5 Bridge on Dawson Highway

Source: TMR (n.d.).

5.5.3 Considerations for Selecting Countermeasures

An example for the selection of appropriate scour countermeasure for a specific bridge site is presented in Table 5.10.

Table 5.10: Selection of scour countermeasures

Scour countermeasure	Underwater construction	Repairs	Construction cost	Maintenance cost	Restricted access	Environmental suitability	High velocity flow	Vertical stream instability	Lateral instability
Riprap	×	✓	L	M	□	✓	✓	✓	□
Mattresses	✓	✓	M	M	□	✓	□	✓	□
Gabions	×	✓	M	M	□	✓	□	□	□
Grout filled mattress	✓	✓	H	M	✓	□	□	✓	□
Rigid grout filled bags	✓	✓	M	L	✓	□	✓	□	□
Concrete aprons	×	✓	H	L	✓	□	✓	×	□
Stone pitching	×	×	M	M	□	□	✓	□	□
Protective collars	×	×	L	L	□	✓	✓	✓	✓
Sheet piling	□	✓	M	L	□	✓	✓	✓	✓

Notes: High – H, Moderate – M, Low – L ✓ Appropriate, □ May be appropriate, × Inappropriate

Source: TMR (2013).

3 Comments from Chris Russell (TMR) as part of the BTF review (2017).

5.5.4 Design of Countermeasures

Filter layer

Many scour countermeasures consist of a filter layer (geotextile or granular) overlain by a heavy-duty armour (usually rock riprap) and possibly a form of containment (basket or cables) holding it together. Correct design of the filter layer is essential and often overlooked. Filters limit the loss of fines, while providing a free-flowing interface. To achieve this, the permeability of the geotextile should be ten times that of the underlying soil. If the filter is too broad the rock riprap will roll off the filter and compromise the countermeasure.

Various considerations for filter design are required, including base soil properties, particle size distribution, plasticity, porosity and hydraulic conductivity of the soil and filter materials.

There are two main filter materials, including granular and geotextile. Refer to TMR (2013) for details.

Rock riprap at bridge piers

Properly designed riprap used for erosion protection has an advantage over rigid structures due to its flexibility when under attack by river currents, as it can remain functional even if some individual stones are lost, and can be repaired relatively easily. Properly constructed riprap can provide long-term protection if it is inspected and maintained on a periodic basis as well as after flood events. This section considers the application of riprap as a pier scour countermeasure.

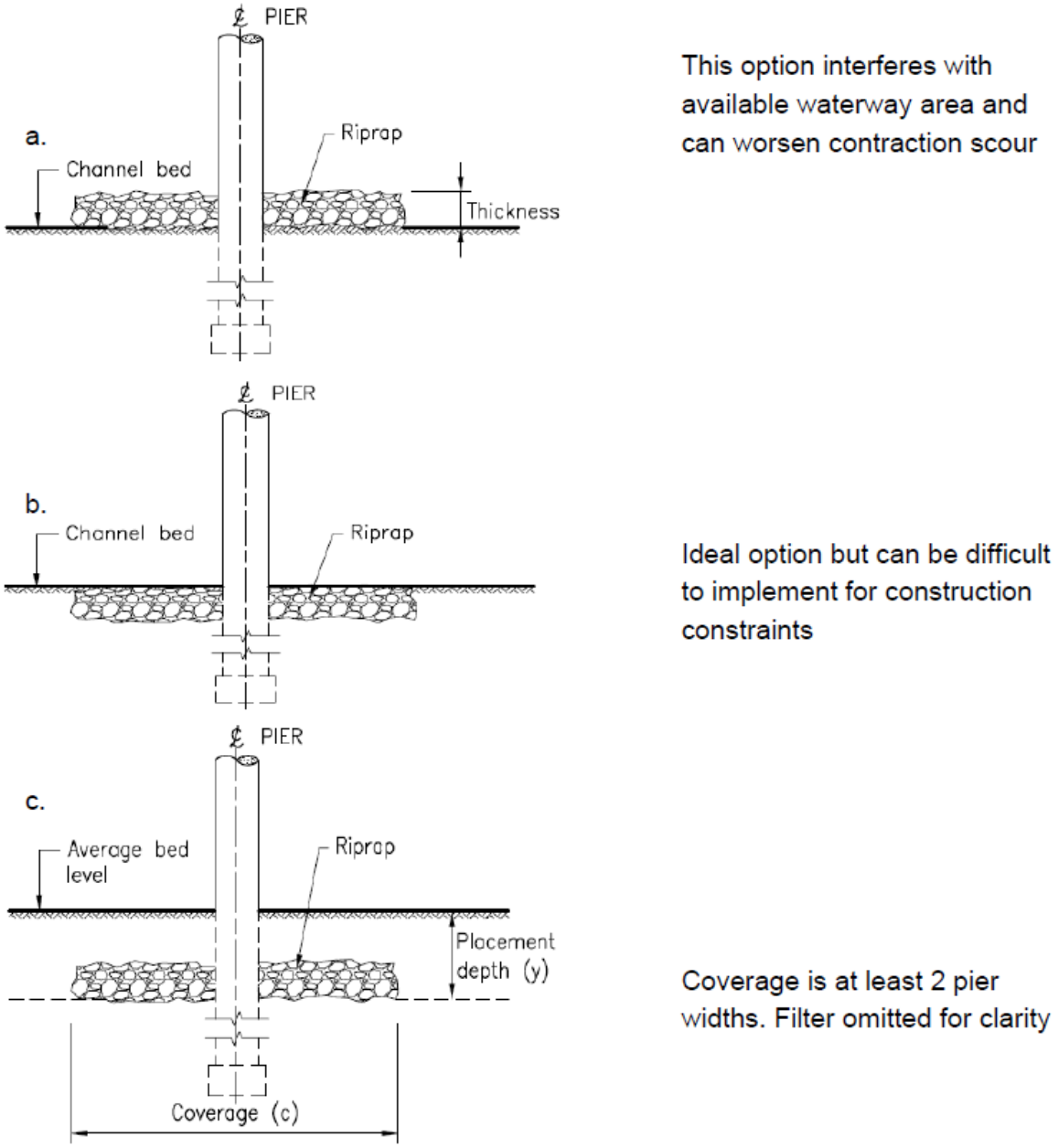
Design of a pier scour countermeasure system using riprap requires knowledge of the:

- river bed and foundation material
- flow conditions including velocity, depth and orientation
- riprap characteristics of size, density, durability, and availability
- pier size, shape, and skew with respect to flow direction
- type of interface material between the riprap and underlying foundation.

The system deployed will typically include a filter layer, either a geotextile fabric or a filter of sand and/or gravel, specifically selected for compatibility with the subsoil. The filter allows infiltration and ex-filtration to occur while providing particle retention. Typical pier riprap configurations are shown in Figure 5.33.

It is worth noting that, for bridge piers, rock riprap is not a permanent countermeasure for scour at piers at existing bridges and should not be used to protect piers at new bridges. The class of rock protection required to protect a pier from scour is determined from the velocity obtained by multiplying the velocity of flow approaching the pier by a coefficient, K_p for pier shape.

Figure 5.33: Typical pier riprap configurations (filter omitted for clarity)



Source: TMR (2013).

The average velocity of flow, V, approaching the pier is estimated by taking the average velocity in the bridge opening and multiplying it by a coefficient that ranges from 0.9 for a pier near the bank in a straight uniform reach of the stream to 1.7 for a pier in the main current of flow around a bend. For piers located on the floodplain the velocity on the floodplain should be used. Where a pier is located adjacent to an abutment and the bridge approach is cutting off significant flow, then some allowance should be made for the resulting increase in velocity caused by the cut-off flow entering the bridge opening.

Coefficient for pier shape, K_p can be taken as 1.5 for round-nose pier, and 1.7 for rectangular pier.

Rock protection should be provided as follows:

- The class and thickness of rock is determined from Table 5.11 for the velocity given by $V \times K_p$.
- The riprap mat should extend horizontally at least twice the pier width, measured from the pier face.

- The top of the riprap mat should be placed at the same elevation as the stream bed.
- In some conditions a filter cloth or gravel filter will be required under the riprap mat. If a well-graded riprap is used a filter may not be needed.

Bridge pier riprap design is primarily based on research conducted under laboratory conditions with little field verification. Flow turbulence and velocities around a pier are of sufficient magnitude that large rocks move over time. The removal of riprap at piers by turbulent high velocity flow has led to the loss of bridges. This is typically the result of a cumulative effect of a sequence of high flows rather than during one storm. Therefore, rock riprap pier scour protection should be monitored and inspected to ensure stability after each high-flow event.

Properly designed rock riprap will afford protection against progressive erosion for abutments and piers where scour is expected. This type of protection has generally been found to be the most practical and economic solution for the protection of spill-through abutments and guide banks. Alternatively, rock filled wire mattresses or hydraulically filled concrete mattresses may be used.

Sizing rock riprap at bridge piers

The required size of stone for riprap at bridge piers is determined by the rearranged Isbash equation (Equation 47), as recommended by Lagasse et al. (2009):

$$d_{50} = \frac{0.692(V_{des})^2}{(S_g - 1)2g} \qquad 47$$

where

d_{50}	=	particle size for which 50% is finer by weight, (m)
V_{des}	=	design velocity for local conditions at the pier, (m/s)
S_g	=	specific gravity of riprap (usually taken as 2.65)
g	=	acceleration due to gravity, (9.81 m/s2)

It is important that the velocity used is representative of conditions in the immediate vicinity of the bridge pier including the constriction caused by the bridge. If the cross-section or channel average velocity, V_{avg} is used, then it must be multiplied by factors that are a function of the shape of the pier and its location in the channel (Equation 48):

$$V_{des} = K_1 K_2 V_{avg} \qquad 48$$

where

K_1	=	shape factor, equals to 1.5 for round-nose piers or 1.7 for square-faced piers
K_2	=	velocity adjustment factor for location in the channel (ranges from 0.9 for a pier near the bank in a straight reach, to 1.7 for a pier located in the main current of flow around a sharp bend)
V_{avg}	=	channel average velocity at the bridge, m/s

If a velocity distribution is available from the flow distribution output of a 1D model or directly from a 2D model, then only the pier shape coefficient (K_1) should be used. In this case, the maximum velocity in the active channel V_{max} is often used since the channel could shift and the highest velocity could impact any pier (Equation 49):

$$V_{des} = K_1 V_{max} \qquad 49$$

where

V_{max}	=	maximum velocity in the active channel, m/s

Once a design size is established, a standard gradation class can be selected, if design criteria and economic considerations permit. Using standard sizes the appropriate gradation can be achieved by selecting the next larger size class, thereby creating a slightly over-designed riprap installation, but economically a less expensive one.

Rock riprap at bridge abutments

Two research studies in a hydraulic flume to were conducted by the FHWA to determine equations for sizing rock riprap for protecting abutments from scour (Pagán-Ortiz 1991, Parola 1993). Pagán-Ortiz (1991) investigated vertical wall and spill-through abutments which encroached 28 and 56% on the floodplain, respectively. Parola (1993) investigated spill-through abutments which encroached on a floodplain with an adjacent main channel. Observed encroachment varied from the largest encroachment used in the first study to a full encroachment to the edge of main channel bank. For spill-through abutments in both studies, the rock riprap consistently failed at the toe downstream of the abutment centreline. For vertical wall abutments, the first study consistently indicated failure of the rock riprap at the toe upstream of the centreline of the abutment.

The class of rock protection required to protect an abutment (without a guide bank) is determined from the velocity given by multiplying the average velocity, V, in the bridge opening by a factor of 1.33, to allow for the turbulently mixing flow action at bridge abutments.

Rock protection should be provided as follows:

- The class and thickness of rock required to protect the embankment slope and the toe of the embankment should be determined from Table 5.11 for the velocity given by 1.33*V.
- The apron at the toe of the abutment slope should extend along the entire length of the abutment toe, around the curved portions of the abutment to the point of tangency with the plane of the embankment slopes.
- The top of the riprap mat should be placed at the same elevation as the stream bed.
- In some conditions a permeable geotextile fabric or gravel filter will be required under the riprap mat. If a well-graded riprap is used a filter may not be needed.

Additional requirements for rock riprap are as follows.

Grading of rock – the grading of rock riprap affects its resistance to erosion. The stone should be reasonably well graded throughout the riprap layer thickness. The grading of the various standard classes of rock protection should be in accordance with Table 5.12.

The riprap should be well graded from the smallest to the maximum specified. Stones smaller than the specified 10% size should not be present in an amount exceeding 20% by weight of each riprap load.

Quality of rock – the riprap should be hard, dense and durable, as well as being resistant to weathering, free from overburden, spoil, shale and organic matter. Rock that is laminated, fractured, porous, or otherwise physically weak is unacceptable to be used as scour protection.

Stone shape is another important factor in the selection of an appropriate riprap material. Riprap constructed with angular material generally has the best performance. Round material can be used as riprap provided it is not placed on slopes greater than 3:1. Flat slab-like stones can be easily dislodged by flow and should be avoided. The breadth or thickness of a single stone should be not less than one-third its length as an approximate guide for good stone shape.

Method of placement of rock protection – the thickness of the rock protection has been determined assuming the following method of placement.

A footing trench should be excavated, along the toe of the slope as shown on Figure 5.36. Rock should be placed so as to provide a minimum of voids. The larger rocks should be placed in the foundation course and on the outside surface of the slope protection. The rock may be placed by dumping and may be spread in layers by bulldozers or other similar equipment.

The embankment and rock protection should be raised in progressive horizontal layers in order to obtain good results where filter fabrics are not used. The larger rocks are placed at the face by bulldozer, and a graded sand/gravel filter material pushed tightly in behind the rock protection where required. The general level of the embankment is then raised to the next level. Local surface irregularities of the slope protection should not vary from the planned slopes by more than 300 mm measured at right angles to the slope.

Filter material – Filter material is material placed between the embankment fill and the rock slope protection to that is designed to prevent fine embankment material from being washed out through the voids of the riprap. The filter may be a permeable geotextile fabric membrane or a graded sand/gravel filter.

Geotextile filter fabrics have generally replaced sand/gravel filters in the construction of roadworks, but they do have a use in the construction of rock protected spill-through abutments as described above. The manufacturer's instructions should be followed when designing geotextile fabric filters. For the design of sand/gravel filters reference should be made to Hydraulic Engineering Circular No 11 (Federal Highway Administration 1989).

When rock slope protection consists of quarry-run rock dumped into place, most of the finer material will naturally settle against the embankment face and the coarser stones will work to the outside, avoiding the need for a filter. But where the face stones are nearly uniform in size and embankment material is vulnerable to scour, a filter will be necessary.

Embankment material should never be carried out over the rock slope protection so that the rock becomes a part of the fill. With this type of construction fill material will filter down through the voids of the large stones and the portion of fill above the rock will be lost.

Table 5.11: Design of rock slope protection

Velocity (m/s)	Class of rock protection, Wc (tonne)	Section thickness, T (m)
< 2	None	–
2.0–2.6	Facing	0.50
2.6–2.9	Light	0.75
2.9–3.9	¼	1.00
3.9–4.5	½	1.25
4.5–5.1	1.0	1.60
5.1–5.7	2.0	2.00
5.7–6.4	4.0	2.50
> 6.4	Special	–

Table 5.12: Standard classes of rock slope protection

Rock class	Rock size (1) (m)	Rock mass (kg)	Minimum percentage of rock larger than
Facing	0.40	100	0
	0.30	35	50
	0.15	2.5	90
Light	0.55	250	0
	0.40	100	50
	0.2	10	90
¼ tonne	0.75	500	0
	0.55	250	50
	0.3	35	90

Rock class	Rock size (1) (m)	Rock mass (kg)	Minimum percentage of rock larger than
½ tonne	0.90 0.70 0.40	1000 450 100	0 50 90
1 tonne	1.15 0.90 0.55	2000 1000 250	0 50 90
2 tonne	1.45 1.15 0.75	4000 2000 500	0 50 90
4 tonne	1.80 1.45 0.90	8000 4000 1000	0 50 90

1 *Assuming a specific gravity of 2.65 and spherical shape.*

Sizing rock riprap at abutments

It is recommended that the following equations be used to determine the size of rock riprap for protecting abutments from scour for spill-through and vertical wall abutments (Lagasse et al. 2009). Depending on the Froude Numbers ($V/(gy)^{\frac{1}{2}}$), the median stone diameter can be calculated as follows:

For Froude Numbers ≤ 0.80 (Equation 50):

$$\frac{d_{50}}{y} = \frac{K}{(S_g - 1)}\left[\frac{V^2}{gy}\right] \qquad 50$$

where

d_{50} = median stone diameter, (m)

V = characteristic average velocity in the contracted section, (m/s)

S_g = specific gravity of riprap (usually taken as 2.65)

g = acceleration due to gravity, (9.81 m/s2)

y = depth of flow in the contracted bridge opening (m)

K = velocity multiplier to account for the apparent local acceleration of flow at the point of rock riprap failure, equals 0.89 for a spill-through abutment; 1.02 for a vertical wall abutment

For Froude Numbers > 0.80 (Equation 51):

$$\frac{d_{50}}{y} = \frac{K}{(S_g - 1)}\left[\frac{V^2}{gy}\right]^{0.14} \qquad 51$$

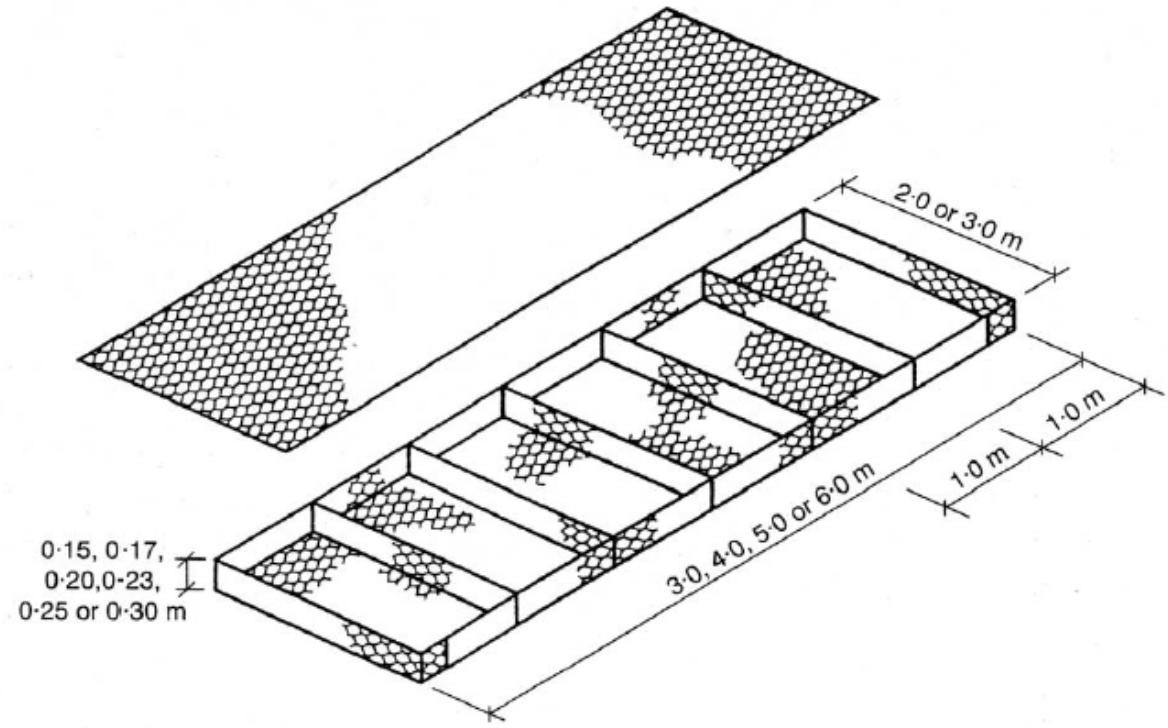
where

K = 0.61 for a spill-through abutment; 0.69 for a vertical wall abutment

Steel-wire gabion and mattresses

Gabion and mattresses are containers constructed of steel-wire mesh and filled with rocks. The length of a gabion mattress is greater than the width, and the width is greater than the thickness, refer Figure 5.34 for typical dimensions. Diaphragms are inserted width wise into the mattress to create compartments. Galvanized or polyvinyl chloride coated wire is used to resist corrosion, and either welded or twisted into a lattice. Angular rock is preferred to fill the containers due to the higher degree of natural interlocking of the stone fill, however rounded cobbles can also be used. During installation, individual mattresses are connected together by lacing wire or other connectors to form a continuous structure.

Figure 5.34: Typical gabion mattress dimensions



Source: Lagasse et al. (2009).

Using wire mesh allows the gabions to deform and adapt to changes in the subgrade while maintaining stability. Less excavation of the bed is required and smaller stones can be used when compared to riprap, leading to a more economical solution. The smaller stones used in gabion mattresses are those that are smaller than those that would individually be too small to withstand the hydraulic forces of a stream.

The following types of gabions are commonly used as armouring countermeasures:

- Gabion sacks: are used where it is not possible to dewater the construction area. In the absence of cofferdams, gabion sacks are placed directly in water. The size of a gabion sack can range between 500 mm to 900 mm.
- Gabion boxes or baskets: are larger in size than sacks and more suitable for higher velocities. The minimum dimension of a gabion box ranges between 600 mm to 1.2 m.
- Rock filled mattresses: are the most commonly used form of gabion. They are thinner than sacks or boxes and have less weight per unit area. Minimum thickness varies between 200 mm to 450 mm. The mattress is manufactured in greater lengths and tied together. For higher scour depths, two mattresses can be placed on top of each other.
- Wire enclosed riprap: differs from mattresses in that it is larger in size and is a continuous framework rather than individual interconnected boxes or baskets. They can be used for slope protection at riverbanks or as guide banks. Riprap sizes that are used are less uniform when compared to other three types discussed above.

It should be noted that gabions and mattresses have durability concerns due to the durability of the steel wire mesh. The maximum life for gabion is 50 years as claimed by the manufacturers.

Grout-filled mattresses

Grout-filled mattresses (mats) are comprised of a double layer of strong synthetic (woven nylon or polyester) fabric, sewn into a series of pillow-shaped compartments that are connected by internal ducts. An example is shown in Figure 5.35. The compartments are filled with a concrete grout that flows from compartment to compartment via the ducts. Mats are typically sewn together or otherwise connected prior to filling.

Figure 5.35: A possible flexible collar arrangement at a pile to seal joint with a mattress

Source: Lagasse et al. (2009).

The grout forms a mat made up of a grid of interconnected blocks when set. The mats are reinforced by cables laced through the mat before the concrete is pumped into the form. This is typically called an articulating block mat (ABM). As flexibility and permeability are important functions for stream instability and bridge scour countermeasures, systems that incorporate filter points or weep holes (allowing for pressure relief across the mat) combined with small-diameter ducts (to allow breakage and articulation between the blocks) are preferred.

Grout-filled mat systems can range from very smooth, uniform surface conditions that approach cast-in-place concrete in terms of surface roughness, to extremely irregular surfaces exhibiting the roughness of moderate size rock riprap. Comprehensive technical information on mat types and configurations are available from manufacturers due to the specialised nature of this type of revetments. Mats are typically available in standard nominal thicknesses of 100, 150 and 200 mm and are occasionally produced up to 300 mm thick.

There has been limited field use of these systems as a scour countermeasure for bridge piers. They have typically been used for shoreline protection, protective covers for underwater pipelines, bridge abutment spill slopes, and channel armouring where the mat is placed across the entire channel width and keyed into bridge abutments or stream banks. This type of scour protection, however, is not recommended for large river where the river bank is very high.

The benefits of grout-filled mats are that the fabric installation can be completed quickly, without the need for dewatering. Because of the flexibility of the fabric prior to filling, laying out the forms and pumping those with concrete grout can be performed in areas where room for construction equipment is limited.

Guide banks

As indicated in Section 5.2.7 guide banks are an effective means of decreasing the risk from scour at bridge abutments. This is achieved by moving the contraction of the streamlines and the generated vortices away from the abutment to the upstream end of the guide bank. Guide banks also protect approach embankments by reducing the flow along the face of the embankment, minimising scour.

Three principal considerations are involved in proportioning a guide bank:

- geometry
- height
- length.

Geometry – Karaki (1960) found that a guide bank in the form of a quarter ellipse, with ratio of major (length) to minor (offset) axes of 2.5:1 performed as well or better than any other shape tested. The equation for this ellipse (see Figure 5.36) is (Equation 52):

$$\frac{X^2}{L_s^2} + \frac{Y^2}{(0.4L_s)^2} = 1 \qquad 52$$

where

X = length of major axis, m

Y = length of minor axis, m

L_s = length of guide bank, m

Height – is based on the anticipated high water level. The guide bank should have sufficient height and freeboard to avoid overtopping and be protected from wave action.

Length – is estimated using the method recommended in Bradley (1978) in which the length of guide bank, L_s, is determined from the discharge ratio Q_f/Q_{30}, relating the flow over the left or right floodplain to a specific portion of the flow under the bridge, a representative velocity adjacent to the abutment of the bridge, and the length of the guide bank needed. L_s is determined from Figure 5.37.

Definition of the symbols used are:

- Q = total stream discharge (m^3/s)
- Q_f = lateral or floodplain flow (one side) measured at the upstream reach of the structure (m^3/s)
- $Q_{30} = Q/b \times 30$ = discharge (m^3/s) in 30 m of stream adjacent to abutment, measured at section 1 in Figure 5.4
- b = length (m) of bridge opening
- A_{n2} = water area (m^2) under bridge referred to normal stage
- $V_{n2} = Q/A_{n2}$ = average velocity (m/s) through bridge opening
- Q_f/Q_{30} = guide bank discharge ratio
- L_s = top length (m) of guide bank (measured as shown on Figure 5.37).

The length of the guide bank should be increased with an increase in floodplain discharge (as observed in Figure 5.37), with an increase in velocity under the bridge, or both. In order read the chart, the ordinate with the proper value of Q_f/Q_{30} should be entered, moving horizontally the appropriate V_{n2} curve, then downward from the abscissa to obtain the required bank length. If the length read is less than 10 m, a guide bank is generally not required. A guide bank not less than 30 m be constructed for chart lengths between 10 m and 30 m. This length is required to direct curvilinear flow around the end of the guide bank to merge with the main channel flow and establish a straight course down the river before reaching the bridge abutment.

This type of flow can have several time the scour capacity than that of parallel flow, depending on factors such as the radius of curvature, velocity, depth and others. Holding all factors constant, the depth of scour will increase with a corresponding decrease in radius of curvature. Therefore, the deepest scour produced by a guide bank will occur near the nose where the radius of curvature is least.

Rock protection should be provided for the guide bank, and should be extended out from the toe of the bank on the bed such that it will fall into the scour hole as it forms, preventing undermining of the guide bank. Figure 5.36 shows this and further details for the construction of guide banks.

The selection of Class of rock protection required for impinging and parallel flow is based on the following assumption:

The velocity ratios are, V_p: V_m: V_i = 2:3:4

where V_p = velocity of parallel flow along tangent bank

V_m = mean velocity through bridge opening

V_i = velocity of impinging flow against curved bank.

The classes of the rock riprap required can be obtained from Table 5.12. The rock protection for parallel and impinging flow should be distributed along the guide bank as shown on Figure 5.36. The level to which the toe of the rock is to be carried will be dependent upon the anticipated depth of scour. The grading of the various Class of rock should be in accordance with Table 5.12. Where necessary a filter should be placed between the embankment fill and the rock slope protection.

Example – given that the face slope of the bridge abutment is 1.5:1, the specific gravity of the rock is approximately 2.65 and the mean velocity of flow through the bridge for the design flow is 3.5 m/s:

V_p = 2/3 x 3.5 = 2.33 m/s

V_i = 4/3 x 3.5 = 4.67 m/s.

From Table 5.12 the rock protection required for parallel flow is Facing Class with a thickness of 0.5 m and for impinging flow is 1 tonne Class with a thickness of 1.6 m.

Spurs

Spurs, retards or groynes are structures or embankments projecting into a stream from the bank at some angle to deflect flowing water away from critical zones, to prevent erosion of the bank, and establish a more desirable channel alignment or width. By deflecting the current from the bank, a spur or a series of spurs may protect the stream bank more effectively and at less cost than rock protecting the bank. Also, by moving the location of any scour away from the bank, failure of the rock protection on the spur can often be repaired before damage is done to structures along and across the river. ‘Guidelines for Stabilising Waterways’ (Standing Committee on Rivers and Catchments Victoria 1991) and ‘Stream Stability at Highway Structures’ (Lagasse et al. 2012) give details of the design of these structures as they relate to stream alignment and bank protection.

Spurs are also used to protect road embankments that form the approaches to a bridge crossing. Often these embankments cut off the overbank flood flows causing these flows to run parallel to the embankment en route to the bridge opening. Spurs constructed perpendicular to the embankment keep the potentially erosive current away to the direction of flow, as shown on Figure 5.38. Therefore, spurs of equal length need not be spaced closer than three times their projected length. For a group of four or more, the spacing may be up to four times their projected length.

Figure 5.36: Guide bank details

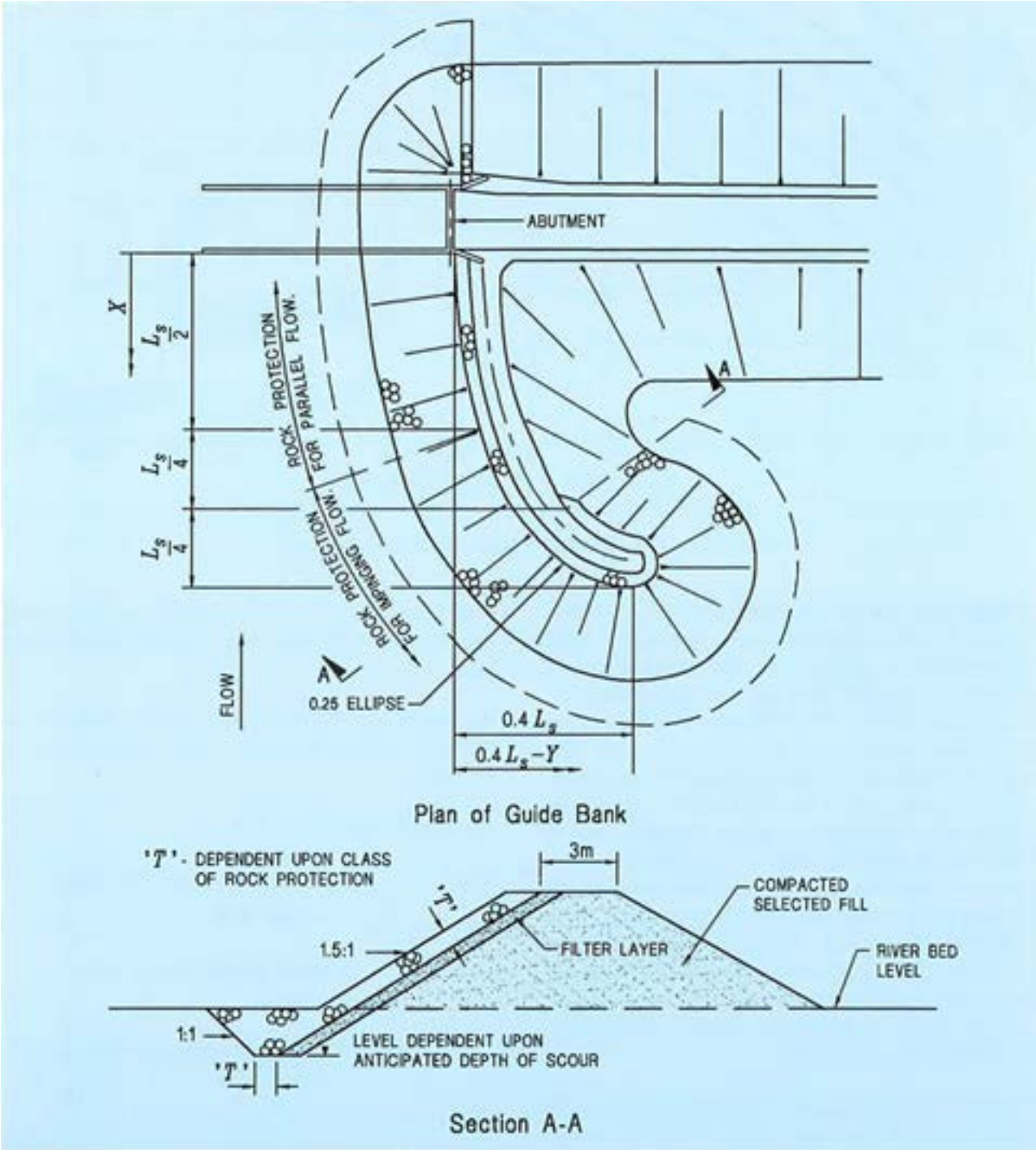


Figure 5.37: Chart for determining length of guide banks

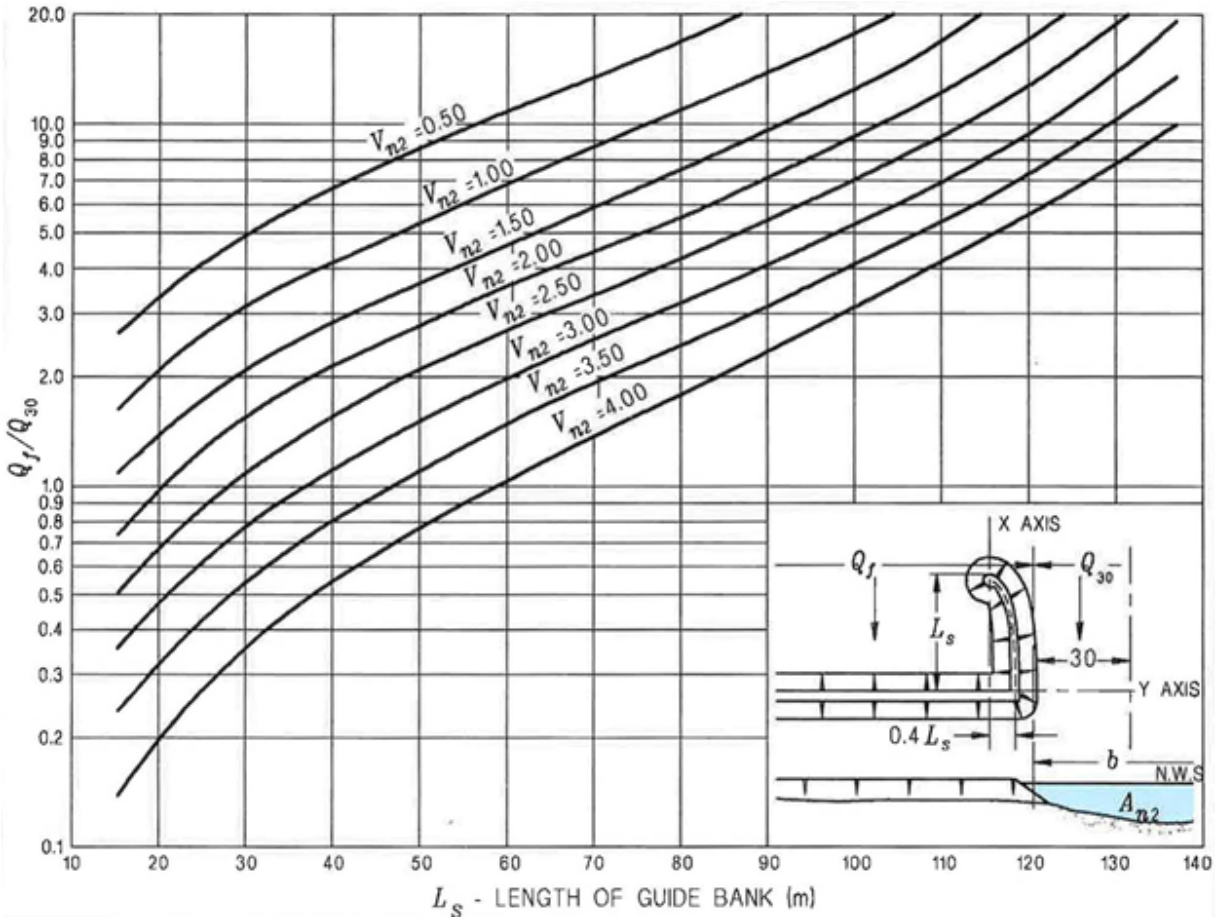
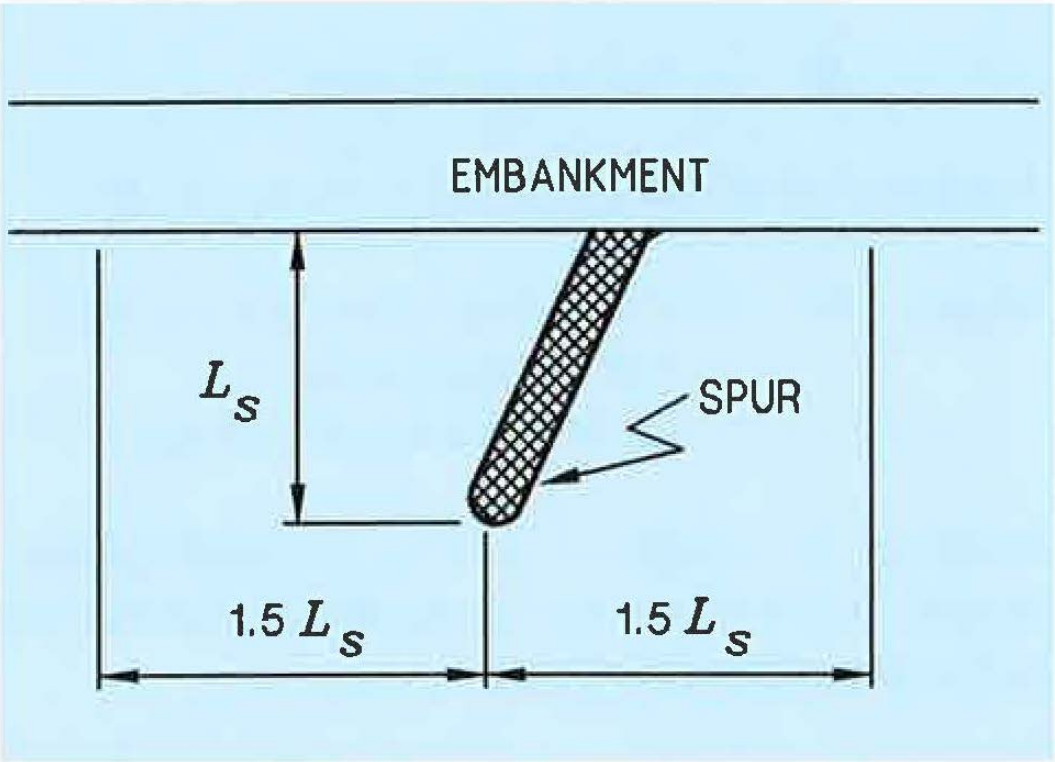


Figure 5.38: Approximate length of embankment protected by spurs



5.6 Monitoring Bridges for Scour

Monitoring of bed levels at each pier and abutment location should be a required maintenance activity. The bed profile along the length of the bridge and the bed level upstream and downstream at each pier and abutment should be recorded at the time of construction. This information will provide base data to monitor long-term trends in the stream bed profile (contraction scour) and detect any changes in bed depth at piers and abutments (local scour).

In one instance the fact that serious scour had occurred was not detected because successive underwater inspections were comparing the bed profile from the previous inspection and not comparing them to the as-built bed profile. The changes in bed profile from one inspection to the next were not considered significant.

Visual inspections are the most common method for monitoring bridge scour. This method involves use of divers to inspect the condition of the foundation and measure the depth and extent of scour. While this method is used widely, it cannot be carried out during the times of flooding when the scour damage is the most visible. In addition, the scour damages may not be measured accurately, as normally the scour hole is filled in when the flood water subsides.

Use of fixed or discrete scour monitoring instrumentation has proved to be the more effective method than visual inspections. Various techniques have been used to monitor the scour in ‘real time’ as discussed in the following section. The use of these devices enables long-term monitoring to determine long-term trends. They also provide information on bed movements in flood. The change in bed levels in flood conditions is considered by some hydrologists to be under-estimated.

5.6.1 Sonar Scour Monitor

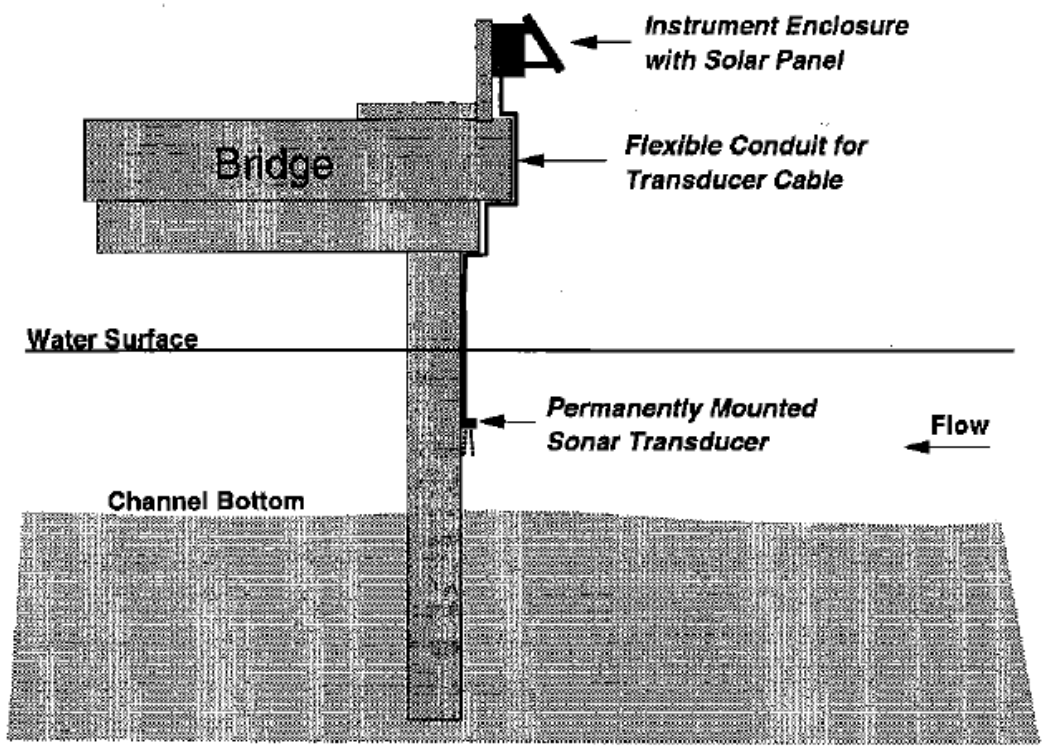
Sonar scour monitoring is a system whereby a conventional sonar instrument is connected to a data logger to provide of continuous records of scour depth by measuring the distance based on the travel time of a soundwave through water (Schall et al. 1997a).

Sonar monitoring for bridge structures uses low-cost, commercially available sonic fathometers point at anticipated scour locations or those where critical failure is possible. The probe is typically mounted to the bridge substructure with the data logger placed on the bridge deck. This can technique can measure either aggradation or degradation.

This system can track both scour and refill processes and has the advantage of being able to be mounted in a variety of elevations out of the way of debris and at various angles of inclination without affecting function. Measurements are, however, affected by aerated flow and bed load.

An example of a sonar scour monitoring system is shown in Figure 5.39.

Figure 5.39: Sonar scour monitoring with permanently mounted transducer



Source: Schall et al. (1997a).

5.6.2 Magnetic Sliding Collar Monitor

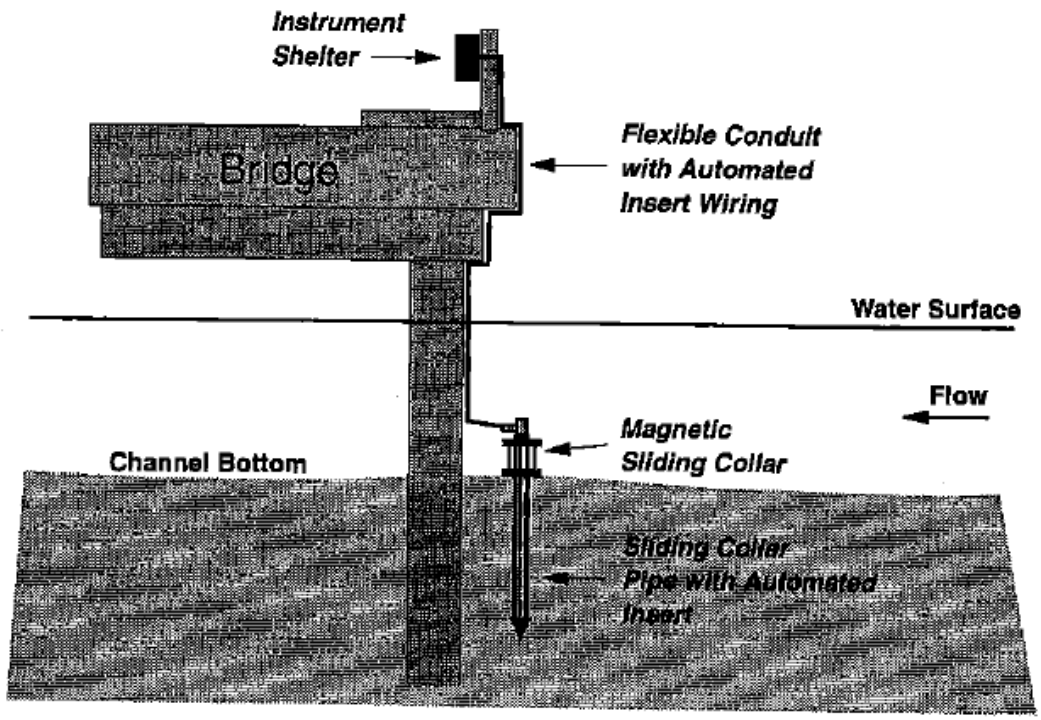
Magnetic sliding collars are collars that slide down a rod driven into the streambed. The collar sits on the riverbed, and as the local bed erodes, the collar follows the bed. This method of monitoring only measures maximum scour depth. Therefore, this system is affected by process that interfere with the collar moving freely up and down the rod, such as debris or aggradation. The system may be automated or manually read. With the automated set-up, magnetic switches inside the driven rod locate the collar and a data logger records the collar's location. In the manual set-up, a magnetic switch is lowered until it is triggered by the collar (Schall et al. 1997b). An example of the magnetic sliding collar monitoring system set-up is shown in Figure 5.40.

5.6.3 Float-out Devices

Float-out devices are buried at appropriate points near the bridge. They are activated when scour exposes the sensor. The sensor floats to the stream surface and an on-board transmitter is activated that transmits the device's digital identification number to a data logger.

A disadvantage of float-out systems is that they only provide a measurement if the scour has progressed past a certain level. Float-outs can be installed at increasing depths to provide hierarchical intervention points. There is a power requirement, but it is minimal. However, the operational capability cannot be verified and the power system must be reliably for long, dormant periods. The interface with the data logger is wireless.

Figure 5.40: Automatic magnetic sliding scour monitoring



Source: Schall et al. (1997b).

5.6.4 Sounding Rods

Sounding-rod or falling-rod instruments are manual or automated gravity-based physical probes. As the streambed scours, the rod, with its foot resting on the streambed, follows the streambed and causes the system counter to record the change. However, the foot must be of sufficient size to prevent penetration of the streambed due to gravity and/or the vibration of the rod from flowing water. This is a major limitation for sand bed channels.

5.6.5 Time Domain Reflectometry (TDR)

TDR operates by sending an electromagnetic pulse down a rod buried vertically in the streambed. When the pulse runs into a change in the boundary conditions, such as the soil-water interface, a portion of the pulse's energy is reflected back to the source from the boundary. The remainder of the pulse's energy propagates through the boundary until another boundary condition (or the end of the probe) causes part or all of the energy to be reflected back to the source. By monitoring the round trip travel time of a pulse in real time, the distance to the respective boundaries can be calculated. This provides information on any changes in streambed elevation.

5.6.6 Ground-penetrating Radar (GPR)

Radar pulses are used to determine the water-sediment interface and the depth of scour. This method uses similar principles to the TDR method in using high frequency electromagnetic waves, however, it uses a floating GPR transmitter instead of using probes installed into the stratum. The floating GPR transmitter is pulled along the water surface to obtain a geophysical profile of the riverbed a long its path. This method is easy to implement and can provide very accurate information about the sub-surface ground conditions. However, it requires manual operation, thus cannot be used during the times of flooding as well as cannot be used for continuous monitoring (Anderson, Ismael & Thitimakorn 2007).

5.6.7 Tiltmeter Arrays

Tiltmeter arrays can be used to measure movement and rotation of the structure. Tiltmeters installed on the bridge pier(s) of concern and superstructure above allow monitoring of the bridge position. In order to monitor all potential ranges of movement, the optimal installation will include sensors installed on pier faces parallel and perpendicular to the direction of travel as well as to the superstructure directly above the pier(s). A system of this nature allows for the monitoring of a structure's stability not only due to scour but for other events, including earthquakes, long-term settlement, impact, construction adjacent to the structure and the effects of 'locked' bearings.

5.6.8 Operational Considerations

All systems suggested above will be required to operate reliably during periods of high river levels and potential overtopping of structures. The design of these systems must therefore include a reliable power supply, robust sensors and equipment as well as being installed in a sufficiently durable manner. This will minimise the potential for damage/malfunction due to inundation, submersion and impact from debris.

It is worth noting that monitoring is not a long-term solution and does not make a scour critical bridge a non-scour critical bridge (Arneson et al. 2012).

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Appendix A Manning's 'n' Values

The following tables are adopted from Austroads Guide to Road Design Part 5B (Austroads 2013a).

Table A 1: Manning's 'n' values for artificial channels/conduits

Type of structure	'n'
Closed conduits	
1. Concrete pipe	0.011–0.013
2. Corrugated metal pipe or pipe-arch (small corrugation)	
(a) Plain or unpaved	0.024
(b) Paved invert – full flow	
25% circumference paved	0.021
50% circumference paved	0.018
Fully paved	0.012
3. Structural plate pipe or pipe-arch	0.030–0.033
4. Monolithic concrete (box culvert)	0.012
5. Vitrified clay pipe	0.012
Open channels – lined	
1. Concrete – smooth forms or trowelled	0.012
2. Asphalt	
(a) Smooth	0.013
(b) Rough	0.016
Open channels – excavated (straight alignment and natural lining)	
1. Earth – uniform section	
(a) Clean – new to weathered	0.016–0.020
(b) With short grass, few weeds	0.022–0.027
(c) In gravelly soil, clean	0.022–0.025
2. Earth – uniform section	
(a) No vegetation	0.022–0.025
(b) Grass, some weeds	0.025–0.030
(c) Dense weeds or plants in deep channel	0.030–0.035
(d) Sides clean, gravel bottom	0.025–0.030
(e) Sides clean, cobble bottom	0.030–0.040
3. Dragline excavated or dredged	
(a) No vegetation	0.028–0.033
(b) Light bush on banks	0.035–0.050
4. Channels not maintained, weeds and brush uncut	
(a) Dense weeds, high as flow depth	0.08–0.12
(b) Clean bottom, brush on sides	0.05–0.08
(c) Same, highest stage of flow	0.07–0.11
(d) Dense brush, high stage	0.10–0.14

Table A 2: Manning's 'n' values for natural channels

Type of channel	'n'
Main channel	
1. Fairly regular section	
(a) Some grass and weeds, little or no brush	0.030–0.035
(b) Dense growth of weeds, depth of flow materially greater than weed height	0.035–0.05
(c) Some weeds, light brush on banks	0.035–0.05
(d) Some weeds, heavy brush on banks	0.05–0.07
(e) Some weeds, dense willows on banks	0.06–0.08
(f) Trees within channel with branches submerged at high stage	Add 0.01–0.02
2. Irregular section, with pools, slight channel meander To (a) to (f) above as applicable	Add 0.01–0.02
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage	
(a) Bottom, gravel, shingle and few boulders	0.04–0.05
(b) Bottom, shingle with large boulders	0.05–0.07
Adjacent flood channels	
1. Pasture, no brush	
(a) Short grass	0.030–0.035
(b) High grass	0.035–0.05
2. Cultivated areas	
(a) No crop	0.03–0.04
(b) Mature row crops	0.035–0.045
(c) Mature field crops	0.04–0.05
3. Heavy weeds, scattered brush	0.05–0.07
4. Light brush and trees	0.06–0.08
5. Medium to dense brush	0.10–0.16
6. Dense willows, summer, not bent over by current	0.15–0.20
7. Cleared land with tree stumps (250 to 450 per ha)	
(a) No sprouts	0.04–0.05
(b) With heavy growth of sprouts	0.06–0.08
8. Heavy stand of timber, a few fallen trees, little undergrowth	
(a) Flood depth bellow branches	0.10–0.12
(b) Flood depth reaches branches	0.12–0.16
Major streams (surface width a flood stage > 30 m) Roughness coefficient is usually less than for minor streams of similar description on account of less effect offered by irregular banks or vegetation on banks. Values of 'n' may be somewhat reduced. The value of 'n' for large streams of mostly regular section may be in the range.	0.12–0.16

Glossary

The definitions below were adopted from Arneson et al. (2012).

Term	Definition
abrasion	Removal of streambank material due to entrained sediment or debris rubbing against the bank.
aggradation	General and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition.
alluvial stream	A stream which has formed its channel in cohesive or noncohesive materials that have been and can be transported by the stream.
apron	Protective material placed on a streambed to resist scour.
armour (armouring)	Surfacing of channel bed, banks, or embankment slope to resist erosion and scour. (a) natural process whereby an erosion- resistant layer of relatively large particles is formed on a streambed due to the removal of finer particles by streamflow; (b) placement of a covering to resist erosion.
average velocity	Velocity at a given cross-section determined by dividing discharge by cross-sectional area.
backfill	The material used to refill a ditch or other excavation, or the process of doing so.
backwater	The increase in water surface elevation relative to the elevation occurring under natural channel and floodplain conditions. It is induced by a bridge or other structure that obstructs or constricts the free flow of water in a channel.
backwater area	The low-lying lands adjacent to a stream that may become flooded due to backwater.
bank	The sides of a channel between which the flow is normally confined.
bank, left (right)	The side of a channel as viewed in a downstream direction.
bank protection	Engineering works for the purpose of protecting streambanks from erosion.
bank revetment	Erosion-resistant materials placed directly on a streambank to protect the bank from erosion.
bar	Elongated deposit of alluvium within a channel, not permanently vegetated.
base floodplain	Floodplain associated with the flood with a 100-year recurrence interval.
bed	Bottom of a channel bounded by banks.
bed load	Sediment that is transported in a stream by rolling, sliding, or skipping along the bed or very close to it; considered to be within the bed layer (contact load). Or The quantity of bed load passing a cross-section of a stream in a unit of time.
bed material	Material found in and on the bed of a stream (May be transported as bed load or in suspension).
bed shear (tractive force)	The force per unit area exerted by a fluid flowing past a stationary boundary.
bed slope	The inclination of the channel bottom.
braid	A subordinate channel of a braided stream.
braided stream	A stream whose flow is divided at normal stage by small mid-channel bars or small islands; the individual width of bars and islands is less than about three times water width; a braided stream has the aspect of a single large channel within which are subordinate channels.
bridge opening	The cross-sectional area beneath a bridge that is available for conveyance of water.
bridge opening ratio	The ratio of the bridge opening width to the total floodplain width
bridge waterway	The area of a bridge opening available for flow, as measured below a specified stage and normal to the principal direction of flow.
catchment	See drainage basin.
causeway	Rock or earth embankment carrying a roadway across water.

Term	Definition
channel	The bed and banks that confine the surface flow of a stream.
channel pattern	The aspect of a stream channel in plan view, with particular reference to the degree of sinuosity, braiding, and anabranching.
check dam	A low dam or weir across a channel used to control stage or degradation.
clay (mineral)	A particle whose diameter is in the range of 0.00024 to 0.004 mm.
clear-water scour	Scour at a pier or abutment (or contraction scour) when there is no movement of the bed material upstream of the bridge crossing at the flow causing bridge scour.
cobble	A fragment of rock whose diameter is in the range of 64 to 250 mm.
confluence	The junction of two or more streams.
constriction	A natural or artificial control section, such as a bridge crossing, channel reach or dam, with limited flow capacity in which the upstream water surface elevation is related to discharge.
contact load	Sediment particles that roll or slide along in almost continuous contact with the streambed (bed load).
contraction	The effect of channel or bridge constriction on flow streamlines.
contraction scour	Contraction scour, in a natural channel or at a bridge crossing, involves the removal of material from the bed and banks across all or most of the channel width. This component of scour results from a contraction of the flow area at the bridge which causes an increase in velocity and shear stress on the bed at the bridge. The contraction can be caused by the bridge or from a natural narrowing of the stream channel.
countermeasure	A measure intended to prevent, delay or reduce the severity of hydraulic problems.
critical shear stress	The minimum amount of shear stress required to initiate soil particle motion.
crossing	The relatively short and shallow reach of a stream between bends; also crossover or riffle.
cross-section	A section normal to the trend of a channel or flow.
current	Water flowing through a channel.
cutoff	(a) A direct channel, either natural or artificial, connecting two points on a stream, thereby shortening the original length of the channel and increasing its slope; (b) A natural or artificial channel which develops across the neck of a meander loop (neck cutoff) or across a point bar (chute cutoff).
debris	Floating or submerged material, such as logs, vegetation, or trash, transported by a stream.
degradation (bed)	A general and progressive (long-term) lowering of the channel bed due to erosion, over a relatively long channel length.
depth of scour	The vertical distance a streambed is lowered by scour below a reference elevation.
design flow (design flood)	The discharge that is selected as the basis for the design or evaluation of a hydraulic structure including a hydraulic design flood, scour design flood, and scour design check flood.
discharge	Volume of water passing through a channel during a given time.
drainage basin	An area confined by drainage divides, often having only one outlet for discharge (catchment, watershed).
drift	Alternative term for vegetative ‘debris’.
eddy current	A vortex-type motion of a fluid flowing contrary to the main current, such as the circular water movement that occurs when the main flow becomes separated from the bank.
ephemeral stream	A stream or reach of stream that does not flow for parts of the year. As used here, the term includes intermittent streams with flow less than perennial.
erosion	Displacement of soil particles due to water or wind action.
fall velocity	The velocity at which a sediment particle falls through a column of still water.
filter	Layer of fabric (geotextile) or granular material (sand, gravel, or graded rock) placed between bank revetment (or bed protection) and soil for the following purposes: (1) to prevent the soil from moving through the revetment by piping, extrusion, or erosion; (2) to prevent the revetment from sinking into the soil; and (3) to permit natural seepage from the streambank, thus preventing the buildup of excessive hydrostatic pressure.

Term	Definition
filter fabric (cloth)	Geosynthetic fabric that serves the same purpose as a granular filter blanket.
flood immunity	Provision of flood immunity is accomplished by ensuring that major networks are designed to maintain flood levels below predetermined levels for facilities in adjacent areas. The appropriate flood levels vary depending upon the facility in question and the general terrain.
flow-floodplain	A nearly flat, alluvial lowland bordering a stream that is subject to frequent inundation by floods.
fluvial geomorphology	The science dealing with the morphology (form) and dynamics of streams and rivers.
fluvial system	The natural river system consisting of (1) the drainage basin, watershed, or sediment source area, (2) tributary and mainstream river channels or sediment transfer zone, and (3) alluvial fans, valley fills and deltas, or the sediment deposition zone.
freeboard	The vertical distance above a design stage that is allowed for waves, surges, drift, and other contingencies.
Froude Number	A dimensionless number that represents the ratio of inertial to gravitational forces in open channel flow.
geomorphology/morphology	That science that deals with the form of the Earth, the general configuration of its surface, and the changes that take place due to erosion and deposition.
gravel	A rock fragment whose diameter ranges from 2 to 64 mm.
grout	A fluid mixture of cement and water or of cement, sand, and water used to fill joints and voids.
guide bank	A dike extending upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening. Some guide banks extend downstream from the bridge.
hydraulics	The applied science concerned with the behaviour and flow of liquids, especially in pipes, channels, structures, and the ground.
hydraulic model	A small-scale physical or mathematical representation of a flow situation.
hydraulic problem	An effect of streamflow, tidal flow, or wave action such that the integrity of the highway facility is destroyed, damaged, or endangered.
hydraulic radius	The cross-sectional area of a stream divided by its wetted perimeter.
hydraulic structures	The facilities used to impound, accommodate, convey or control the flow of water, such as dams, weirs, intakes, culverts, channels, and bridges.
hydrograph	The graph of stage or discharge against time.
hydrology	The science concerned with the occurrence, distribution, and circulation of water on the earth.
incised stream	A stream which has deepened its channel through the bed of the valley floor, so that the floodplain is a terrace.
island	A permanently vegetated area, emergent at normal stage, that divides the flow of a stream. Islands originate by establishment of vegetation on a bar, by channel avulsion, or at the junction of minor tributary with a larger stream.
levee	An embankment, generally landward of top bank, that confines flow during high-water periods, thus preventing overflow into lowlands.
live-bed scour	Scour at a pier or abutment (or contraction scour) when the bed material in the channel upstream of the bridge is moving at the flow causing bridge scour.
load (or sediment load)	Amount of sediment being moved by a stream.
local scour	Removal of material from around piers, abutments, spurs, and embankments caused by an acceleration of flow and resulting vortices induced by obstructions to the flow.
longitudinal profile	The profile of a stream or channel drawn along the length of its centreline. In drawing the profile, elevations of the water surface or the thalweg are plotted against distance as measured from the mouth or from an arbitrary initial point.
lower bank	That portion of a streambank having an elevation less than the mean water level of the stream.

Term	Definition
mattress	A blanket or revetment of materials interwoven or otherwise lashed together and placed to cover an area subject to scour.
meander or full meander	A meander in a river consists of two consecutive loops, one flowing clockwise and the other counter-clockwise.
meander loop	An individual loop of a meandering or sinuous stream lying between inflection points with adjoining loops.
median diameter	The particle diameter of the 50th percentile point on a size distribution curve such that half of the particles (by weight, number, or volume) are larger and half are smaller (D_{50}.).
mid-channel bar	A bar lacking permanent vegetal cover that divides the flow in a channel at normal stage.
migration	Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.
normal stage	The water stage prevailing during the greater part of the year.
overbank flow	Water movement that overtops the bank either due to stream stage or to overland surface water runoff.
oxbow	The abandoned former meander loop that remains after a stream cuts a new, shorter channel across the narrow neck of a meander. Often bow-shaped or horseshoe-shaped.
pavement	Streambank surface covering, usually impermeable, designed to serve as protection against erosion. Common pavements used on streambanks are concrete, compacted asphalt, and soil-cement.
paving	Covering of stones on a channel bed or bank (used with reference to natural covering).
perennial stream	A stream or reach of a stream that flows continuously for all or most of the year.
pile	An elongated member, usually made of timber, concrete, or steel, that serves as a structural component of a river-training structure or bridge.
piping	Removal of soil material through subsurface flow of seepage water that develops channels or ‘pipes’ within the soil bank.
point bar	An alluvial deposit of sand or gravel lacking permanent vegetal cover occurring in a channel at the inside of a meander loop, usually somewhat downstream from the apex of the loop.
pressure flow/scour	See vertical contraction scour.
probable maximum flood	A very rare flood discharge value computed by hydro-meteorological methods, usually in connection with major hydraulic structures.
reach	A segment of stream length that is arbitrarily bounded for purposes of study.
recurrence interval	The reciprocal of the annual probability of exceedance of a hydrologic event (also return period, exceedance interval).
regime	The condition of a stream or its channel with regard to stability. A stream is in regime if its channel has reached an equilibrium form as a result of its flow characteristics. Also, the general pattern of variation around a mean condition, as in flow regime, tidal regime, channel regime, sediment regime, etc. (used also to mean a set of physical characteristics of a river).
relief bridge	An opening in an embankment on a floodplain to permit passage of overbank flow.
revetment	Rigid or flexible armor placed to inhibit scour and lateral erosion.
riffle	A natural, shallow flow area extending across a streambed in which the surface of flowing water is broken by waves or ripples. Typically, riffles alternate with pools along the length of a stream channel.
riparian	Pertaining to anything connected with or adjacent to the banks of a stream (corridor, vegetation, zone, etc.).
riprap	Layer or facing of rock or broken concrete dumped or placed to protect a structure or embankment from erosion; also the rock or broken concrete suitable for such use. Riprap has also been applied to almost all kinds of armor, including wire-enclosed riprap, grouted riprap, sacked concrete, and concrete slabs.

Term	Definition
river training	Engineering works with or without the construction of embankment, built along a stream or reach of stream to direct or to lead the flow into a prescribed channel. Also, any structure configuration constructed in a stream or placed on, adjacent to, or in the vicinity of a streambank that is intended to deflect currents, induce sediment deposition, induce scour, or in some other way alter the flow and sediment regimes of the stream.
rock	Indurated geomaterial that requires drilling, wedging, blasting, or other methods of applying force for excavation.
roughness coefficient	Numerical measure of the frictional resistance to flow in a channel, as in the Manning or Chezy's formulas.
rubble	Rough, irregular fragments of materials of random size used to retard erosion. The fragments may consist of broken concrete slabs, masonry, or other suitable refuse.
runoff	That part of precipitation which appears in surface streams of either perennial or intermittent form.
sand	A rock fragment whose diameter is in the range of 0.062 to 2.0 mm.
scour	Erosion of streambed or bank material due to flowing water; often considered as being localized (see local scour, contraction scour, total scour).
scour prism	Total volume of stream bed material removed by scour in the bridge reach for design flood conditions.
sediment or fluvial sediment	Fragmental material transported, suspended, or deposited by water.
sediment discharge	The quantity of sediment that is carried past any cross-section of a stream in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross-section.
sediment load	Amount of sediment being moved by a stream.
sediment yield	The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.
seepage	The slow movement of water through small cracks and pores of the bank material.
serviceability limit state	Refer to AS 5100.1 for the definition with regard to bridges and structures.
shear stress	See unit shear force.
silt	A particle whose diameter is in the range of 0.004 to 0.062 mm.
sinuosity	The ratio between the thalweg length and the valley length of a stream.
slope (of channel or stream)	Fall per unit length along the channel centreline or thalweg.
slope protection	Any measure such as riprap, paving, vegetation, revetment, brush or other material intended to protect a slope from erosion, slipping or caving, or to withstand external hydraulic pressure.
slope area method	A method of estimating unmeasured flood discharges in a uniform channel reach using observed high-water levels.
soil	Any unconsolidated geomaterial composed of discrete particles with gases and liquids in between.
spill-through abutment	A bridge abutment having a fill slope on the streamward side. The term originally referred to the ‘spill-through’ of fill at an open abutment but is now applied to any abutment having such a slope.
spread footing	A pier or abutment footing that transfers load directly to the earth.
spur (dike, groin, jetty)	A structure extending from a bank into a channel that is designed to: (a) reduce the stream velocity as the current passes through the dike, thus encouraging sediment deposition along the bank (permeable dike); or (b) deflect erosive current away from the streambank (impermeable dike).
stability	A condition of a channel when, though it may change slightly at different times of the year as the result of varying conditions of flow and sediment charge, there is no appreciable change from year to year; that is, accretion balances erosion over the years.

Term	Definition
stable channel	A condition that exists when a stream has a bed slope and cross-section which allows its channel to transport the water and sediment delivered from the upstream watershed without aggradation, degradation, or bank erosion (a graded stream).
stage	Water-surface elevation of a stream with respect to a reference elevation.
stone riprap	Natural cobbles, boulders, or rock dumped or placed as protection against erosion.
stream	A body of water that may range in size from a large river to a small rill flowing in a channel. By extension, the term is sometimes applied to a natural channel or drainage course formed by flowing water whether it is occupied by water or not.
subcritical, supercritical flow	Open channel flow conditions with Froude Number less than and greater than unity, respectively.
thalweg	The line extending down a channel that follows the lowest elevation of the bed.
tidal waterways	Scour at bridges over tidal waterways, i.e., in the coastal zone.
toe of bank	That portion of a stream cross-section where the lower bank terminates and the channel bottom or the opposite lower bank begins.
toe protection	Loose stones laid or dumped at the toe of an embankment, groin, etc., or masonry or concrete wall built at the junction of the bank and the bed in channels or at extremities of hydraulic structures to counteract erosion.
total scour	The sum of long-term degradation, general (contraction) scour, and local scour.
tractive force	The drag or shear on a streambed or bank caused by passing water which tends to move soil particles along with the streamflow.
turbulence	Motion of fluids in which local velocities and pressures fluctuate irregularly in a random manner as opposed to laminar flow where all particles of the fluid move in distinct and separate lines.
ultimate limit state	Refer to AS 5100.1 for the definition with regard to bridges and structures.
ultimate scour	The maximum depth of scour attained for a given flow condition. May require multiple flow events and in cemented or cohesive soils may be achieved over a long time period.
uniform flow	Flow of constant cross-section and velocity through a reach of channel at a given time. Both the energy slope and the water slope are equal to the bed slope under conditions of uniform flow.
unit discharge	Discharge per unit width (may be average over a cross-section, or local at a point).
unit shear force (shear stress)	The force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually in units of stress, Pa (N/m^2).
unsteady flow	Flow of variable discharge and velocity through a cross-section with respect to time.
velocity	The time rate of flow usually expressed in m/s. The average velocity is the velocity at a given cross-section determined by dividing discharge by cross-sectional area.
vertical contraction scour	Scour resulting from flow impinging on bridge superstructure elements (e.g. low chord).
vortex	Turbulent eddy in the flow generally caused by an obstruction such as a bridge pier or abutment (e.g. horseshoe vortex).
wash load	Suspended material of very small size (generally clays and colloids) originating primarily from erosion on the land slopes of the drainage area and present to a negligible degree in the bed itself.
watershed	See drainage basin.
waterway opening width (area)	Width (area) of bridge opening at (below) a specified stage, measured normal to the principal direction of flow.

Guide to Bridge Technology consists of eight parts. It provides guidance to bridge owners and agencies on issues related to design procurement, vehicle and pedestrian accessibility and bridge maintenance and management practices, including the use and application of Australian and New Zealand bridge design standards.

Guide to Bridge Technology Part 8: Hydraulic Design of Waterway Structures covers issues related to the waterway design of bridges including design flood standards and estimation methods, general considerations in waterway design and design considerations of waterway structures. It includes the design of new bridges for scour, scour countermeasures, and monitoring.

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Austroads

Austroads is the association of Australasian road and transport agencies.

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