

FLOODWAY DESIGN GUIDE



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FLOODWAY DESIGN GUIDE

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1 SCOPE

The purpose of this document is to provide guidelines for the hydrological and hydraulic analysis and structural and civil engineering design of floodways and associated structures. Sufficient technical details are included to give guidance for the analysis and design of simple floodways to be carried out. References are included for the design of more complex or unusual floodway structures.

This document is not intended to be prescriptive in nature, however worked examples and flow charts have been included to assist the practitioner. It is intended to be used by practitioners reasonably familiar and experienced in surface water hydrology and hydraulics, or by inexperienced practitioners with the guidance and assistance of a more experienced practitioner.

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2 INTRODUCTION

Apart from bridges and culverts, a flood crossing provides for flow over a road at a specific location under conditions determined by the designer. Flood crossings can be broadly divided into two types:

- Causeway – a roadway across a watercourse or across tidal water, specifically designed to resist submergence.
- Floodway – a roadway across a shallow depression subject to flooding, specifically designed to overtop and constructed to resist the damaging effects of overtopping.

From these definitions, a floodway is a special case of causeway.

Floodways are provided, generally, under the following circumstances:

- Where flow across the road is infrequent or of short duration.
- Where traffic volumes and serviceability requirements are low, and the cost of a bridge or major culvert structure is not justified.
- In conjunction with a bridge or culvert, where the bridge or culvert is designed to pass a lesser flood than the total waterway design flood, and the floodway provides relief during large flows. The bridge or culvert may be designed not to be overtopped, or to be submerged as part of the floodway (with the associated structural implications).

3 DESIGN CRITERIA & CONSIDERATIONS

3.1 HYDROLOGY & DESIGN FLOWS

3.1.1 Hydrology

A hydrological investigation is required in order to estimate the design flows for various Average Recurrence Intervals (ARIs) for the floodway. Extreme flood events should also be considered to allow for the investigation of the floodway stability and performance under unusually large flood events.

The methods of hydrological analysis typically adopted in Western Australia are summarised below. These methods are discussed in detail in Australian Rainfall and Runoff (AR&R).

Rational Method – A statistical method used to estimate peak flows based on runoff coefficients for a reference ARI, and average rainfall intensities over a critical rainfall duration derived from the catchment area. Runoff coefficients for other ARIs are obtained by dividing C_2 , C_5 , C_{20} and C_{50} by C_{10} for each catchment and calculating the mean values for all catchments in the region.

Index Flood Method – A method based on flood frequency analysis where a relationship for a base discharge is derived for a single ARI based on multiple regression analysis of flood records, and average frequency factors are developed for other ARIs and applied to the base discharge.

Runoff-Routing Modelling – A storage network method where storages are arranged to represent the drainage network of a catchment. Design storms for various ARI are applied to the model, which calculates losses and discharges from runoff based on variables input by the user. The software program RORB is typically used for the application of this method.

The Rational and Index Flood methods are typically applied to small catchments. Runoff routing analysis is generally carried out for larger catchments. As a guide RORB should be used for catchment areas of 50 km² or greater; where hydrographs and closure period assessments are specifically required; or where the cost of the proposed structure is high and more accurate analysis is warranted.

Hydrology is not an exact science, and different hydrologic methods developed for determining flood runoff may produce different results for a particular situation. Therefore, sound engineering judgement should be exercised to select the appropriate method or methods to be applied.

For details on hydrological analysis, refer to AR&R and MRWA Technical Report 50T. An example of basic hydrological analysis for a small catchment is provided in Appendix D.

3.1.2 Design Flows

Design flows for each applicable ARI are adopted following consideration of all methods of hydrological analysis used for a particular catchment.

At times, it may be necessary to adjust design flows to match more realistic observed or historical flows. Where the waterway has been gauged with reasonable accuracy, the use of flood frequency curves to determine design flows is encouraged. The designer should check on the availability of gauged flood data from agencies such as the Department of Environment (DoE). The DoE has flood frequency and stage-discharge curves on record for some river systems in Western Australia.

The designer should also seek out any available anecdotal or historical information and records from landowners, local government agencies and the MRWA Waterways Section.

3.1.3 Hydrographs

To estimate the time of closure for a crossing, a flood hydrograph is required (a plot of discharge versus time for a given design event). Hydrographs can be obtained from runoff routing analysis using RORB, or from measured data from gauging stations. Gauged data for many crossings is available from the DoE.

For small catchments that do not warrant runoff routing analysis, the typical flood duration can be estimated from anecdotal information or observed data. The MRWA Waterways Section should be consulted if such an estimation is to be made.

3.2 SERVICEABILITY LEVEL

3.2.1 Definition of Floodway Serviceability

Extensive experiments have been conducted by Bonham and Hattersley (1967) to ascertain the performance of motor vehicles negotiating flooded causeways. Buoyancy reduces the reaction between the tyres and the causeway surface and at the same time the flow of water produces a lateral pressure against the side of the car. The car proceeds until the lateral pressure exceeds the maximum frictional resistance that can be developed by the car tyres under the reduced loading.

Bonham and Hattersley found that under ideal conditions cars operate with safety up to a depth of flow of 365 mm. However, they adopted a depth of 230 mm as the limit of trafficability, due to the probable presence of debris, potholes and waves etc. Because cars have become lighter since Bonham and Hattersley carried out their experiments, MRWA adopted a *critical depth* of 200 mm as the limit of trafficability for conventional cars, beyond which the flood crossing is described as closed to conventional cars. Time of closure is then the time for which critical depth of flow over the flood crossing exceeds 200 mm. Similarly, for heavy vehicles the maximum passable critical depth adopted by MRWA is 500 mm. These values for critical depth correspond to a *total head* (depth plus velocity head) over the floodway of 300 mm and 750 mm respectively.

It is rare for a public road to be used by heavy vehicles exclusively; hence the maximum critical depth for heavy vehicles of 500 mm is generally not used as a design criterion. Where a road is used primarily by heavy vehicles, or only needs to be serviceable for heavy vehicles during flooding, a separate design criteria for heavy vehicles may be adopted. The MRWA Waterways Section should be consulted for advice in such cases.

The critical depth is always equal to two thirds of the total head (refer Figure 4.1). The serviceability of the floodway should always be assessed using the critical depth y_c (not the upstream water depth h).

3.2.2 Selection of Floodway Serviceability

It is unlikely to be economically feasible to construct a floodway that will be trafficable under all flow conditions. Thus in determining an appropriate level of serviceability, there must be a balance between the floodways' serviceability, its cost, and its sustainability against failure during greater than design flows. Various classes of National, State and Local Government roads have different serviceability standards, and it is important that the appropriate authority is consulted such that floodways are designed to meet specified requirements and standards.

Examples of typical serviceability levels are presented below, however these are indicative only, and the required serviceability for any road must be assessed on a case-by-case and site-specific basis.

Typical Serviceability Levels (Indicative Only)

Freeways and arterial roads:	50 – 100 years
Minor urban roads:	20 – 50 years
Rural main roads:	20 – 50 years
Rural minor roads:	10 – 20 years
Rural local access roads:	5 – 10 years

The level of serviceability to be provided to traffic at a particular waterway crossing may depend on the serviceability requirements of the entire road link. Although the probability of closure of a road link is dependent on the failure of the link as a whole, and not the failure of a particular stream crossing, it is normal practice to design each stream crossing on a road link for some predetermined level of serviceability.

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

- The level of service expected by the community.
- The availability of alternative routes and period of closure.
- Anecdotal and historical information relating to road closure and damage.
- The importance of the road as access in emergency and post-disaster recovery situations, such as to hospitals, airports etc.

- The relationship between traffic density and composition, and the wet season, especially in northern Australia.

3.2.3 Economic Considerations

As the provision of higher serviceability levels typically results in an increase in the cost of the structure, in Western Australia the selection of the serviceability level for a new floodway should be made on the advice of the MRWA Waterways Section and the asset owner. Also, it is important to investigate the floodways' performance when greater than design flows eventuate.

3.2.4 Closure Period

In addition to defining an ARI for which the crossing must remain serviceable (open to traffic), maximum closure period criteria also require consideration (generally for highways and key main roads only). Such criteria typically take the form: "The crossing shall not be closed for more than X hours for the Y-year ARI design flow". The estimation of the closure period for a crossing is discussed in Section 4.5.

3.3 SURVEY INFORMATION

Calculation of the floodway capacity, velocities at the floodway, and the capacities of any associated structures requires reliable information on the *upstream* water level and the *stage / tailwater* level for each design flow. Field survey is required in order to obtain this information. As a minimum, the following are typically required:

- A cross section(s) across the river, extending beyond the water level for the discharge being considered. This section is typically taken along (or very near) the line of the existing or proposed road. Cross sections upstream and downstream of the proposed structure are also required.
- A long section (profile) along the streambed, including the water surface profile if available, in order to estimate the hydraulic gradient of the waterway.
- A long section (profile) on the road centreline if an existing road is being analysed.

A detailed contour plot is also very useful (but not essential) as it allows the designer to determine the flow patterns at the crossing, and the optimum locations for drainage culverts or other waterways structures.

A guide for the provision of survey data for waterways investigations is available from the MRWA Survey and Mapping Section – "Survey and Mapping Standard 67-08-42 Waterways Investigation Surveys".

3.4 BACKWATER & UPSTREAM FLOODING

In many cases, the existence of a waterways structure, such as a bridge, culvert or floodway, will cause water levels to increase on the upstream side. This increase in water level is known as the *backwater*, or *afflux*. The distance upstream that is affected depends on the size of the backwater, and the hydraulic gradient of the watercourse. The acceptability of this increase is defined by the existence of upstream property or infrastructure. Upstream assets that cannot cope with increased flood levels will typically necessitate a higher capacity waterways structure, in order to minimise the backwater effects.

In some waterways in the urban areas of Western Australia, the Department of Water (DoW), Water Investigation and Assessment branch, has in place Flood Management Plans, with established allowable backwater levels. Appropriate agencies such as the Department of Water, should also be consulted regarding acceptable backwater levels.

3.5 ASSOCIATED WATERWAYS STRUCTURES

Floodways are often constructed as secondary waterways structures to provide relief and to act in conjunction with bridges or major culverts.

Where a floodway is designed as the primary waterways structure (designed to take the majority of the design flow), it should be constructed with drainage culverts (unless it is constructed at ground level). These drainage culverts are required for one or more of the following functions:

- To reduce the backwater.
- To raise the tailwater level in order to reduce the head across the floodway.
- To facilitate drainage and prevent ponding behind the embankment. Ponding on the upstream side of the floodway can result in piping or sediment transport phenomenon, affecting the structural integrity of the floodway.
- To facilitate drainage and prevent overtopping for smaller, more frequent flows.

For very low-lying floodways, only nominal drainage culverts may be required. As the height of the embankment and the volume of water that can be contained behind the embankment increase, the size of the culvert required also increases.

Nominal drainage culverts are typically ignored in hydraulic calculations, however as the size of the culvert increases, the proportion of the total discharge passed through the culvert increases, and must be included in the analysis. Software programs such as CulvertW allow analysis of combined culverts and floodway models. AFFLUX allows analysis of combined bridge and floodway systems. In some situations (such as when a floodway invert level is below the culvert obvert level) these programs may not function. In such cases, an iterative method is required, using the program to analyse the culvert, and hand calculations for the floodway (refer Section 4.4), iterating to balance the Upstream Water Level (USWL).

For long floodway lengths, drainage culverts may be required at several locations, to avoid excessive backwaters, and to minimise lateral flow at the upstream embankment batter slope. Guidance on the placement of multiple drainage culverts can be provided by the MRWA Waterways Section.

3.6 ROAD GEOMETRY & VEHICLE SAFETY

3.6.1 Horizontal Alignment

Floodways should not be located on horizontal curves for the following reasons:

- There are problems in defining the edge of the pavement for motorists.
- Any superelevation may change the normal flow distribution, i.e. push more water to the non-superelevated sections of road.
- The water depth will be deeper on one side of the road than the other in a superelevated section of road and there is the possibility of the high side being trafficable but not the other, thus creating a vehicle safety problem.

3.6.2 Vertical Alignment

The vertical alignment of a floodway is determined by:

- Hydraulic capacity.
- Geometric road design standards.
- Vehicle safety.
- Effect of backwater upon land use.
- Structural stability.

Floodways should be designed with a horizontal longitudinal profile (except for the sag curves to the approach ramps) so that the depth of water over the road is as uniform as possible over the flooded section. Building a floodway on a level grade avoids the possibility of a driver encountering concealed

changes in flow depth. An exception to this is a skew crossing of a major stream, where the natural stream grade must be estimated and applied in proportion to the flood crossing, to avoid variations in flow depth over the length of the floodway.

The length of floodways should be limited to about 300 m, so that drivers do not become disorientated when confronted with wide, open stretches of water. Where wide flood sections have to be crossed, floodways should be broken into shorter lengths by the provision of sections of road that are raised above the maximum flood level. However, care should be taken to avoid the creation of isolated islands. The level of serviceability must be the same on either side. The use of such raised 'islands' may also result in vehicles being trapped while crossing during rising flood levels.

Adequate sight distance to allow drivers approaching a floodway to stop when there is water over the road is of utmost importance. Crest curves (see Figure 3.1) should be designed, therefore, to provide for adequate visibility.

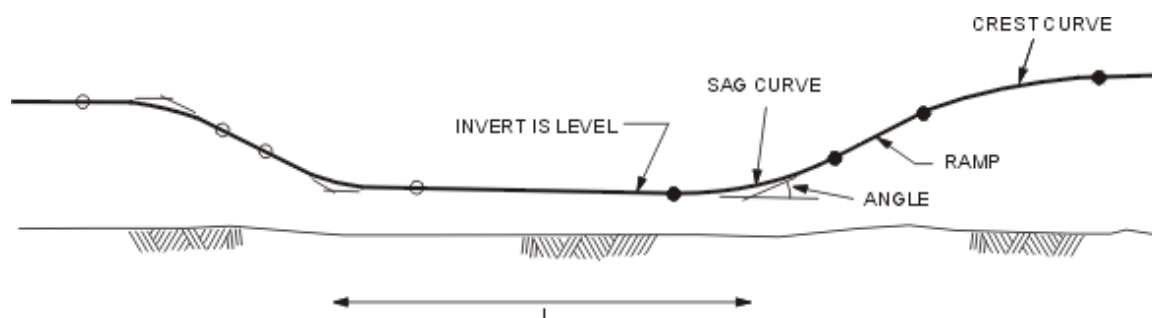


Figure 3.1 - Long Section of a Typical Floodway

3.6.3 Embankment Cross-Section

MRWA generally designs floodways with the trafficable depth of flow of 200 mm occurring at the crown of the road. A two-way crossfall is generally preferred for the following reasons:

- Vehicles traversing floodways under flood conditions generally travel down the middle of the road. It is easier to travel down the middle of a floodway with a two-way crossfall.
- The two-way crossfall generally matches the shape of the approach road.

A one-way crossfall induces smooth, stable flow over the floodway, but may result in a hydraulic jump forming on the road surface (for down-slope flow). In cases where the use of a one-way crossfall is being considered (where a floodway must be constructed on a horizontal curve), advice should be sought from the MRWA Waterways Section.

Examples of typical floodway cross sections are provided in Section 5.7.

3.6.4 Safety Issues & Floodway Signage

Floodways should be designed so that they are not covered by water, from ponding or backwater, for any significant period after a flood event. Adequate drainage culverts must be provided to ensure this does not occur.

The signing of floodways should be in accordance with the Australian Standard 1742.2-1994, Manual of Uniform Traffic Control Devices-Part 2-Traffic Control Devices for General Use. A warning sign, FLOODWAY, should be provided and depending on the depth of flooding, a sign, ROAD SUBJECT TO FLOODING INDICATORS SHOW DEPTH, and a depth indicator may also be required. In addition, a pair of guide posts should be provided approximately every 25m to delineate the edge of the road pavement.

Guardrailing and other barriers represent a significant obstruction to the flow over the floodway, and should not be used wherever possible. For this reason, the embankment height and road geometry should be designed (as much as possible) such that guardrailing is not required. That is, to minimise the hazards to vehicles at the floodway, during wet and dry conditions.

3.7 ENVIRONMENTAL & ETHNOGRAPHIC ISSUES

Environmental considerations must be taken into account throughout the design process. The issues that may be required to be considered include (but are not exclusive to) the following:

Environmental Impact - The site should be investigated for possible problems that might occur for a range of flood events, with emphasis on the more frequent events. Factors to be considered include site selection; limiting backwater levels and velocities; erosion of banks from flow redirection and turbulence; protection of vegetation; control of road drainage; provision of adequate water flow to downstream areas; fauna passage; and fish migration.

Construction Effects - The impact of each of the construction activities 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. A preliminary Environmental Assessment and Management Plan (EAMP) at the design stage should be considered at potentially sensitive sites. Construction contract documents should require contractors to prepare an EAMP for all construction activities.

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. Environmental concerns for stream velocities, flow depths 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.

Ethnographic Issues - An Aboriginal Heritage Investigation may be required as part of the design process. The need for a heritage investigation can be clarified by liaison with the MRWA Indigenous Heritage Officer, review of existing information on heritage issues in the area, and liaison with the Department of Indigenous Affairs. The Main Roads document, Environmental Guideline - Aboriginal Heritage (Document No. 6707/006), should be consulted for details of Aboriginal Heritage issues.

4 HYDRAULIC ANALYSIS

4.1 GENERAL

The objective of the design is to determine the configuration, size and extent of the floodway; and to assess the need for culverts or other associated waterways structures. Hydraulic analysis is required to meet the following objectives:

- Obtain a stage-discharge curve for the natural section, in order to estimate stage and tailwater levels for various flows.
- Assess the capacity of the floodway and determine the backwater caused by the floodway.
- Assess the closure period for various flood events. The stage-discharge curve and flood hydrograph can be used to estimate the time of closure for the design flood events.
- Calculate the velocities at the floodway in order to determine the type, size and extent of scour protection required.

Where a floodway is acting as a secondary waterways structure, in conjunction with a bridge or major culvert, the floodway may be analysed using the in-built features of the bridge or culvert analysis program. Examples of currently available hydraulic analysis programs are AFFLUX by MRWA; HEC-RAS by the US Army Corps of Engineers Hydrologic Engineering Centre; Culvert W by Iceminster Pty Ltd; and the XP range of programs by XP Software.

Where a floodway is acting as a primary waterways structure, manual calculations are typically used. If the floodway is constructed with a small drainage culvert, or if the associated culvert takes a small proportion of the total flow, the contribution of the culvert is typically ignored. This approach will produce slightly conservative results, but will simplify the calculations.

4.2 NATURAL SECTION DISCHARGE

Manning's formula for open channel flow is typically used to determine the stage-discharge curve for the natural section. The equation, shown below in its simplest form is applicable only to prismatic channels, but can be used to estimate the velocity and discharge in natural channels.

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}, \text{ with the discharge given by } Q = AV \quad \text{-----Equation (1)}$$

Where:

- V = velocity (m/s)
- n = Manning's roughness coefficient (dimensionless)
- A = cross-sectional area of flow, (m²)
- R = hydraulic radius (m) = Area of flow (A) / Wetted Perimeter (P)
- S = stream hydraulic gradient (m/m)
- Q = flow (m³/s)

Values for the Manning roughness coefficient 'n' should be estimated during a site inspection, or from current photographs of the site. A roughness coefficient is selected for each part of the stream cross section. Typically, a different roughness coefficient is selected for the main channel, each immediate riverbank, and the extended floodplain. A separate value should be used wherever there is marked change in ground or vegetation type. A guide to estimating the roughness coefficients is presented in Appendix B.

For natural sections, it is preferable for the hydraulic gradient 'S' of the stream to be estimated from the water surface profile, by survey of running water levels or reliable debris marks. If estimation of the water surface profile is not practical, the gradient can be estimated from the slope of the streambed at the proposed crossing. Estimation of the hydraulic gradient from the streambed levels must be carried out over a relatively long section of the streambed to avoid the influence of localised scouring and

sediment transport effects near existing waterways structure, and to get a true indication of the streambed gradient.

Natural sections are typically non-prismatic. Such sections can be analysed by breaking the section into a number of smaller sub-sections and applying Manning's equation to each sub-section. The MRWA software program AFFLUX can be used for this analysis.

Care should be exercised in the survey of cross-sections and stream gradients, if realistic results are to be obtained with this method. Whilst the survey detail level required is typically not high, the quality and accuracy of the survey should always be checked prior to use, to ensure the section used is an appropriate representation of the site.

The design engineer should also be aware that backwater caused by downstream structures, obstructions or constrictions can affect the floodway being designed. As simple analysis programs such as AFFLUX consider a single cross section only, and do not allow for existing downstream conditions, the output must be manipulated and interpreted with care to ensure that the desired results are obtained. For example, where two waterways structures are located close together along a stream, the downstream structure should be analysed first under the adopted design flows, and then the upstream water levels from the downstream structure should be used as the stage levels for the upstream structure. That is, the upstream structure is designed for a higher '*apparent flow*' to allow for the effect of the downstream structure. In such cases consideration should be given to the use of water surface profile computer programs such as HEC-RAS.

4.3 STAGE & TAILWATER LEVELS

The stage is defined as the water level at the floodway or structure on the natural section for the design flow, that is, the water level assuming there are no structures present at the crossing. It is typically taken at (or close to) the road centreline.

The tailwater level is similar to the stage level, but is taken at the outlet position of the culvert or floodway (ie. tailwater = downstream stage). As the structure outlet is generally only a few metres downstream of the road centreline, and the hydraulic gradient is typically very small, there is often very little difference between the stage and tailwater levels, and the same figure can be used for both levels. The stage and tailwater levels can be estimated using the MRWA software program AFFLUX. A survey cross section across the stream, and an estimation of the longitudinal streambed gradient, are required as input for AFFLUX. The cross section must extend beyond the water level for the discharge being considered.

4.4 FLOODWAY CAPACITY

4.4.1 General

For floodways constructed at ground level (that is, effectively no embankment) that do not represent a hydraulic control (effectively no energy loss), the capacity, discharge and velocities at the floodway can be estimated using Manning's equation (or the AFFLUX program).

Methods for calculating the capacity of raised floodways are detailed in the following sections. The first method is more detailed and is applicable to submerged and unsubmerged flows. The second is a simplified version of the first method, and is applicable only to unsubmerged flows.

Unsubmerged floodways are assumed to act as *broad-crested weirs*. That is, the fluid pressure over the floodway may be considered to be hydrostatic, and that nearly uniform critical flow is achieved over the floodway. Submerged floodways may be analysed by modifying the broad crested weir analysis with a submergence factor in the early stages of submergence, and can be modelled as open channels when fully submerged and the floodway no longer constitutes a hydraulic control.

The nomenclature used in floodway analysis is shown on Figure 4.1 below, and is also listed in Appendix A.

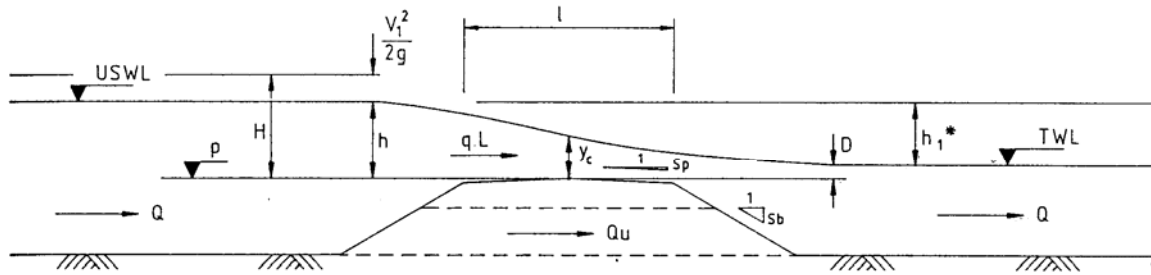


Figure 4.1 – Floodway Nomenclature

4.4.2 Detailed Method

The discharge over a floodway can be determined using Figure 4.2 and the following procedure:

1. Determine stage-discharge curve for the stream using Manning's Equation. For the design discharge, obtain the tailwater level and the average velocity of flow, V , approaching the floodway, from an open channel analysis.
2. Select a floodway crest level and length of floodway, L . Assume a height of headwater, h , above the floodway crest.

The length of floodway is usually taken as the length between intersection points of the sag vertical curves on the approaches into the floodway, as shown on Figure 3.1. The extra capacity gained on the ramps and the loss of capacity due to the sag curves is generally ignored, as the two effects tend to cancel each other out.

Where the floodway is located on a vertical curve, or the design flow considered results in the extent of flooding being beyond the main floodway section, the overall floodway should be modelled as a series of smaller floodways, with a separate invert level and length for each.

In determining h , the designer should bear in mind that once the upstream water level rises above the top of the ramp(s) and approach(s) to the floodway, then the flow will spread out over the road and there will be little rise in the upstream water level for larger flows. This provides an upper bound to the backwater that can occur.

3. Calculate H/l

Where, $H = \text{total head (static plus velocity)} = h + \frac{V^2}{2g}$

$h = \text{height (m) of headwater above floodway crest.}$

$V = \text{average velocity (m/s) of flow approaching floodway.}$

$g = \text{acceleration due to gravity} = 9.81 \text{ m/s}^2$

$l = \text{width (m) of floodway}$

4. Enter curve B (Figure 4.2) with H/l and obtain free flow coefficient of discharge, C_f . Should the value of H/l be less than 0.15, C_f should be read from curve A.
5. If submergence is present (eg. If, $D/H > 0.76$) calculate percent submergence $D/H \times 100$ and read off the submergence factor C_s/C_f .

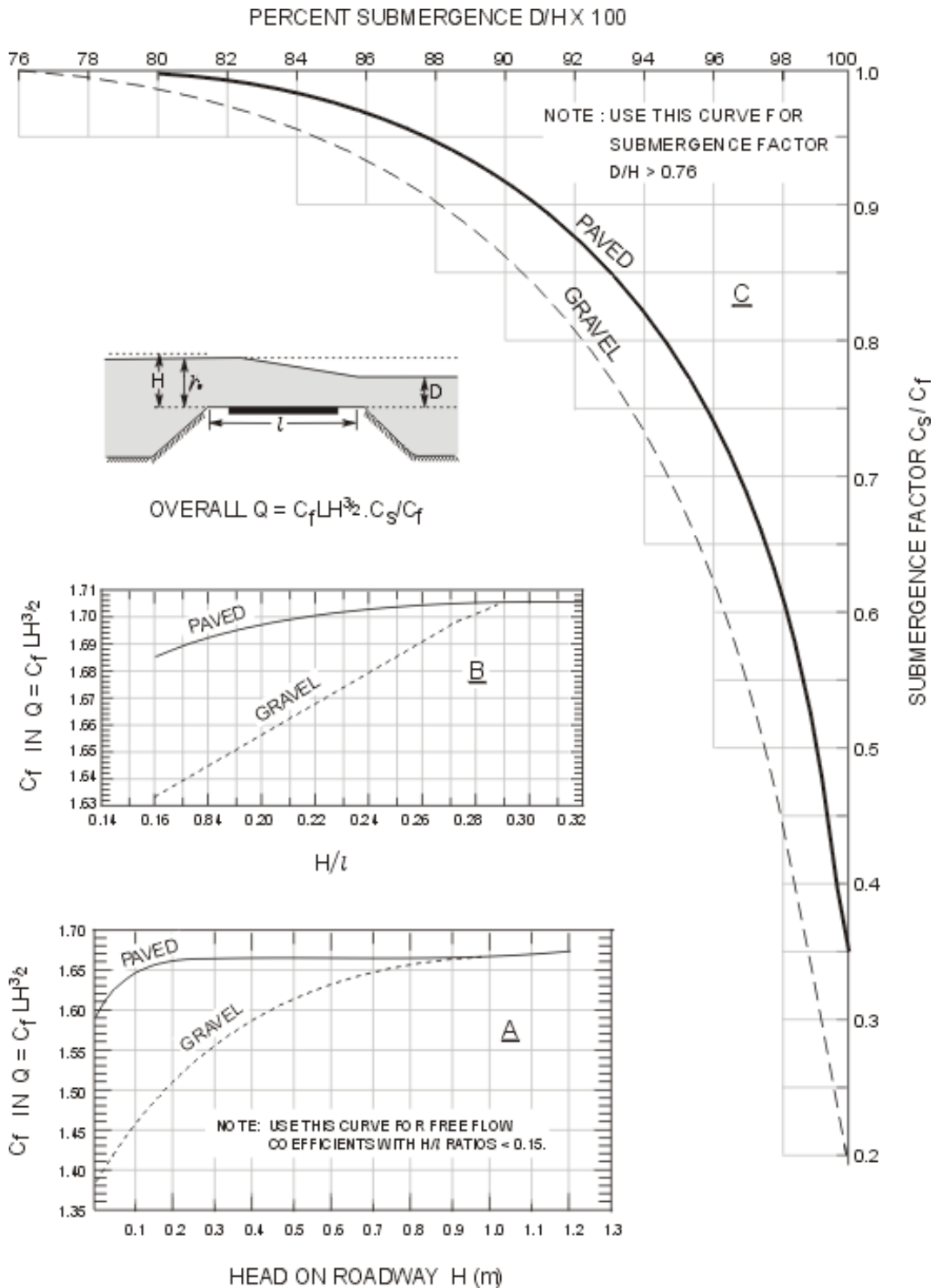


Figure 4.2 – Discharge Coefficients for Floodways

6. Calculate discharge (m^3/s) over floodway using the broad crested weir formula,

$$Q = C_f LH^{3/2} \frac{C_s}{C_f} \quad (m^3/s) \quad \text{-----Equation (2)}$$

7. If submergence is present check that the discharge over the floodway matches the design discharge. If it does not, adjust the depth of flow above floodway crest, h , and repeat procedure. Alternatively, the floodway crest level or length can be adjusted.

When a bridge or culvert operates in conjunction with a floodway, a backwater versus discharge curve needs to be determined for the bridge or culvert. Once the floodway is overtopped an iterative procedure, as indicated above, is required to estimate the combined flows of bridge or culvert and floodway. It should be noted that once the floodway comes into operation, backwater is generally decreased and the flow through the bridge or culvert is decreased.

When the culverts are small or take only a small proportion of the total flow, calculation is greatly simplified if the flow through the culvert is ignored. This will yield a slightly conservative design, but the accuracy is typically not greatly affected.

4.4.3 Simplified Method

For free outfall conditions (unsubmerged), the variation of C_f with H/l is a constant value of 1.69 can be assumed for C_f . This approach assumes that the approach velocity is low, and that the velocity head can be ignored. This assumption simplifies calculation and does not lead to any significant error. The simplified equation is shown below:

$$Q = 1.69H^{3/2}L \quad \text{-----Equation (3)}$$

This equation is typically referred to as the '*broad crested weir*' capacity equation.

The equation above is only valid if the flow is critical somewhere at some point over the floodway embankment cross section, that is, the flow is unsubmerged. This will occur if D/H is less than 0.76.

When D/H is greater than 0.76, the equation is no longer valid, as the flow remains sub-critical at all locations on the cross section. Under these conditions, the floodway is submerged and no longer acts as a hydraulic control. For the early stages of submerged flow, the detailed method described in Section 4.4.2 should be used. For fully submerged flow, the floodway can be analysed as an open channel using Manning's equation.

4.5 CLOSURE PERIOD ASSESSMENT

To calculate the time of closure for a specific flood, the following information is required:

- A hydrograph of the flood, which may be obtained from actual measurements at the stream crossing, or for ungauged streams by the use of a runoff routing method or synthetic unit hydrograph methods.
- The trafficable capacity of the stream crossing.

The time of closure may then be calculated by drawing a horizontal line on the hydrograph at the trafficable discharge level and measuring the time for which the flow is above this level. The procedure for estimating the average annual time of closure may be found in Section 9.2.2 of the Austroads Waterway Design Manual (1994).

Road closure for light vehicles is assumed when the total head (static plus velocity) on a carriageway with a two-way crossfall or across the highest edge of a carriageway with a one-way crossfall exceeds 300mm (refer Section 3.2).

Examining the hydrographs depicted in Figure 4.3, the adopted serviceability criteria will require that the road is passable for some specified ARI flood (with associated flow Q_1). The adopted maximum closure time criterion requires that the road closure time (Δt) is less than the maximum for some specified larger flood event with flow Q_2 .

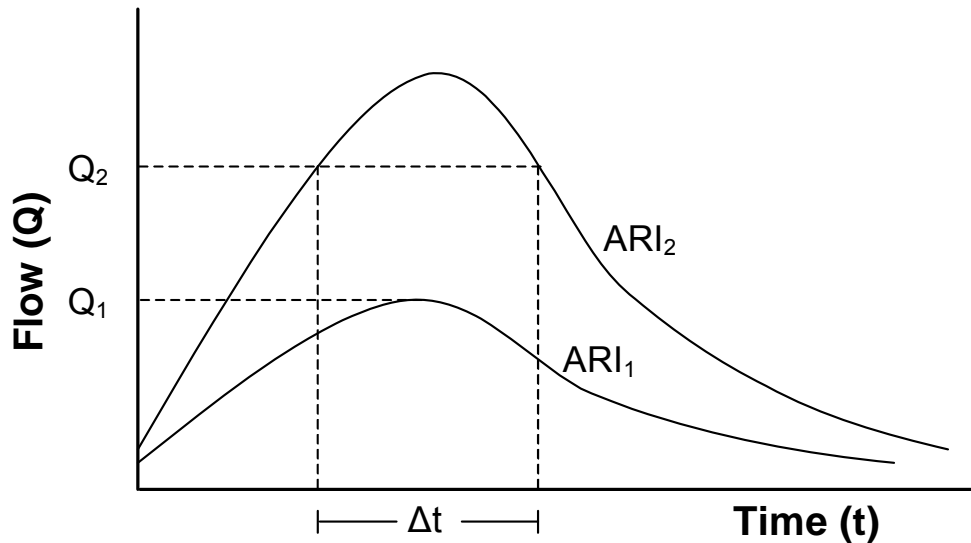


Figure 4.3: Hydrographs for Determining Serviceability Specifications

Thus the floodway must be designed such that when the flow is Q_1 , the critical depth over the floodway is less than 200 mm to meet the serviceability criteria. The floodway must also be designed so that the closure time Δt for the flow Q_2 is less than the maximum allowed, in order to meet the maximum closure time criteria.

The governing criteria will depend on the relative magnitudes of Q_1 and Q_2 , and the shape of the hydrograph.

4.6 FLOODWAY VELOCITY CALCULATION

4.6.1 Flow Regimes

The definition of submerged and unsubmerged (free) flow is discussed in Section 4.4. Free flow may be further subdivided into plunging flow and surface flow. The various flow regimes are shown in Figure 4.4.

Figure 4.4A shows free outfall flow with low tailwater, in which a high velocity jet passes down the downstream batter of the flood crossing, accelerating to a maximum velocity at the bottom of the batter. The sudden change of direction and bed friction decelerates the flow until the water level passes through the critical depth and a hydraulic jump occurs.

As the tailwater rises (Figure 4.4B) the hydraulic jump moves upstream until it reaches the downstream batter of the flood crossing. At this stage free plunging flow occurs with the high velocity jet plunging into a turbulent body of water. The velocity of the jet reaches a maximum at or just below the surface of the tailwater. Bed friction and eddy currents in the body of the tailwater decelerate this jet. In general, plunging flow is of the most interest due to its more severe erosive effects on the downstream batter slope.

With further rise in tailwater level, surface flow occurs (Figure 4.4C). Surface flow occurs when the flow separates from the surface of the floodway and overlays the downstream tailwater.

Submerged flow occurs when the tailwater level rises further, and the depth of flow over the floodway is everywhere greater than the critical depth.

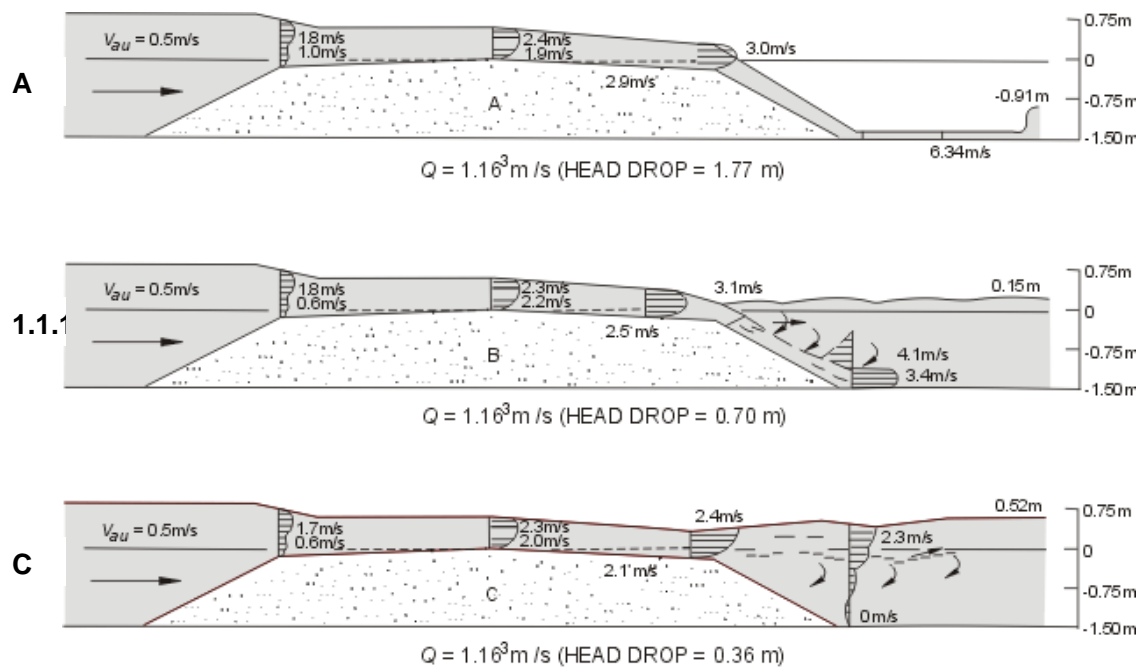


Figure 4.4 – Flow Regimes

4.6.2 Velocity Calculation

The design velocity at the upstream batter slope is equal to the approach velocity of the stream. This can be obtained from the AFFLUX analysis of the natural section carried out to determine the stage levels. In reality there is a zone of lower velocity just upstream of the line of maximum water level where drawdown commences, but for simplicity this is ignored.

In order to determine the level of protection required on the downstream batter or the type of pavement to use, the maximum flow velocity which can be expected at various locations on the embankment cross section needs to be calculated. Flowcharts showing a summary of the design and calculation processes are presented in Appendix C. Worked examples are provided in Appendix D.

Flow that is critical over the road may not cause high velocities on the batter, if the flow is of the surface type (Figure 4.4C). High velocities on the batter result from a plunging flow. The upper limit of D/H at which this type of flow occurs is dependent upon the ratio of H/l (Figure 4.5). In a plunging type flow, the velocity of the jet reaches a maximum at or just below the surface of the tailwater and maintains this velocity down the batters and along the streambed.

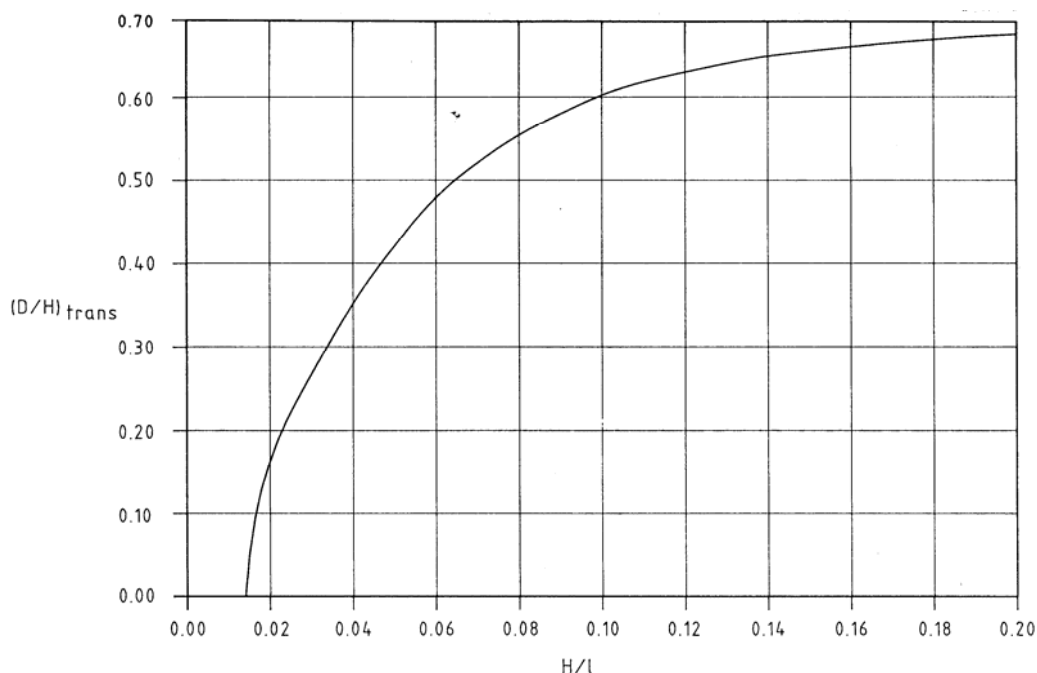


Figure 4.5 – Upper Limit of Transition Range

The peak velocity on the pavement (V_p) will always occur at the downstream edge just before submergence. It is under these conditions that discharge is a maximum in the supercritical regime and hence the greatest velocity occurs. The peak velocity on the batter (V_b) will occur somewhere in the plunging flow regime, but it may or may not be at the greatest discharge associated with plunging flow. This is explained further in the following procedure, which is used to calculate the maximum velocities that can be expected on the batter and pavement.

At a low tailwater condition the flow will accelerate down the batter until either one of three things happen.

- It reaches a steady state velocity as described by Manning's Equation. The flow will maintain this velocity down the batter until penetrating the tailwater surface. It is then decelerated by the turbulence in the hydraulic jump. Under these conditions the maximum velocity obtained by the flow occurs above the tailwater surface and equals the steady state velocity.
- It penetrates the tailwater surface while still accelerating. It is then decelerated by the turbulence of the hydraulic jump. Under these conditions the maximum velocity obtained by the flow occurs at the tailwater surface and will be less than that described by Manning's Equation.
- It reaches the natural surface and remains supercritical until a hydraulic jump occurs further downstream. Observations made in the field have shown that this does not usually occur and hence this condition will not be considered further.

The steady state velocity of flow on a slope (V_s) may be calculated from Manning's Equation. For ease of construction and safety, batter slopes (S_b) are typically 1:3 (or 1:6 for a low embankment). For roads with low traffic volumes where cost savings are required, the batter slope may be set to 1:2, however this situation is undesirable and should be avoided wherever possible. The pavement crossfall is typically 3%.

Manning's 'n' will vary from 0.012 for a batter protected with concrete slab and up to 0.06 for dumped rock. Table 4.1 gives values of Manning's 'n' for various classes of dumped rock protection and other forms of protection. The rougher the batter, the lower the velocity and hence the designer should aim at a compromise between roughness and the safety of motorists should they run off the road.

Pavement/Batter Protection	Manning's 'n'
Bitumen Seal	0.013-0.016
Concrete	0.012
Grass	0.030
Rock Mattress	0.050
Dumped Rock: Facing Class	0.055
Light Class	0.055
¼ tonne	0.060
½ tonne	0.060
1 tonne	0.060

Table 4.1 – Values of Manning's 'n'

Manning's equation:

$$V_s = \left(\frac{1}{n} q^{2/3} S^{1/2} \right)^{3/5} \text{-----Equation (4)}$$

As the flow accelerates down the batter, it will achieve a steady state velocity only when sufficient potential head has been converted to enable this velocity to be attained. Thus a flow with an upstream head H above the crown and which is known to have a steady state velocity (V_s) on the batter will achieve this steady state at a vertical distance Δp below the crown, such that:

$$\frac{V_s^2}{2g} + \frac{q \cdot \cos \theta}{V_s} = H + \Delta p = E_s \text{-----Equation (5)}$$

The maximum velocity on the batter (V_b) will occur when either the flow just reaches steady state at the tailwater level, that is, when Δp in Equation 5 becomes equal to p -TWL, or at the transition from surface to plunging flow. The discharge at which this occurs is best solved for by graphical means. To simplify the analysis, the $\cos \theta$ term in Equation 5 can be taken as 1, causing less than about 3% error.

Thus the energy of a steady state flow on the batter is given by the simplified energy equation:

$$E_s = \frac{V_s^2}{2g} + \frac{q}{V_s} \text{-----Equation (6)}$$

For a range of q , V_s can be determined by Equation 4 and E_s by Equation 6. By plotting E_s vs. q on the stage discharge curve using the TWL (stage) curve as the datum for E_s (that is, the energy level is equal to the stage plus E_s from Equation 6), an energy grade line for flow on the batter, which is assumed to reach steady state at the tailwater surface, is generated. This plot is best achieved using a spreadsheet. This energy line is only valid for tailwater levels below the crown (since we are considering the flow plunging into the tailwater surface) and consequently the analysis must be divided into two flow regimes, $TWL \leq p$ and $TWL > p$. The maximum batter velocity in each regime must be checked independently.

If the E_s line intersects the USWL curve at a tailwater level below the crown level, the maximum batter velocity in this regime (V_{bu}) equals V_s for the discharge at which the intersection occurs. If the curves do not intersect, then V_b will equal the maximum batter velocity in the $TWL > p$ regime (V_{bo}).

The maximum batter velocity in the $TWL > p$ regime will occur at the transition discharge (i.e. the discharge at which the flow changes from plunging flow to surface flow). V_{bo} will be the lesser of V_s , as

given by Equation 4 using q at transition, and the velocity, V_m , as given by Equation 7. Equation 7 gives the maximum velocity of flow attainable (assuming no losses) given a head of $H + \Delta p$. In this regime, we are assuming that the plunging flow will be decelerated below the shoulder level because of the high tailwater and hence use $\Delta p = p -$ downstream shoulder level.

$$V_m = K\sqrt{H} \quad \text{-----Equation (7)}$$

Where: V_m = the velocity of flow

K = a proportionality constant dependent upon the ratio $\Delta p/H$ and is given in Figure 4.6.

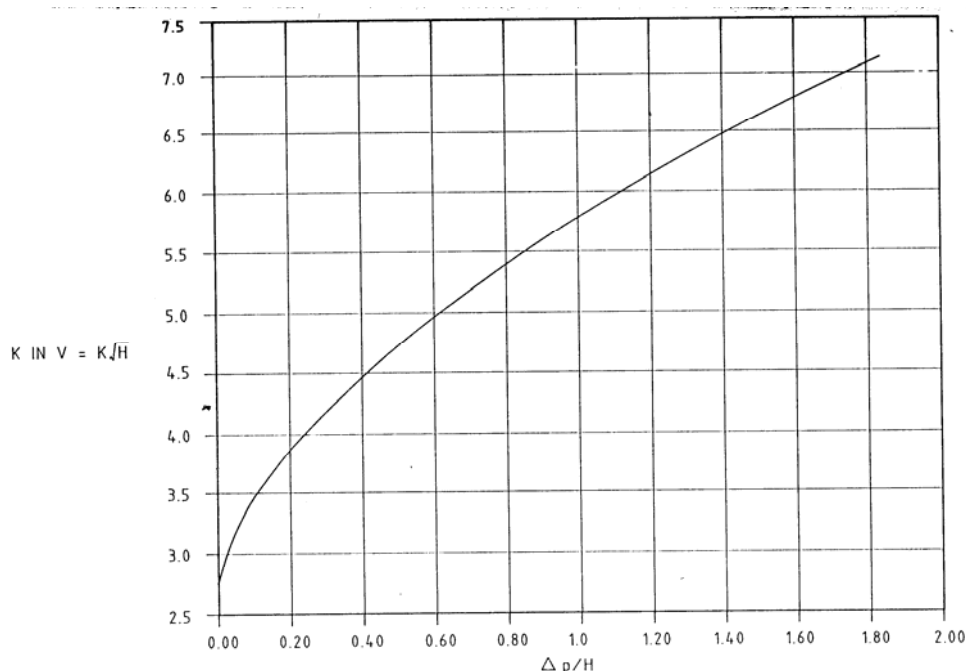


Figure 4.6 – K Values for Calculation of V_m

The maximum batter velocity then is the greater of the batter velocities as calculated in each regime, i.e. the greater of V_{bu} and V_{bo} .

On the pavement the maximum velocity (V_p) occurs at the downstream shoulder at submergence. V_p is the lesser of V_s (using Equation 4 with q at submergence) and V_m (using Equation 7 with $\Delta p = p -$ downstream shoulder level).

The velocity of flow for any other discharge may be calculated using a similar procedure. On the pavement use the lesser of V_s and V_m with $\Delta p = p -$ downstream shoulder level and q as required. On the batter, use the lesser of V_s and V_m with $\Delta p = p -$ downstream shoulder level for a TWL above the crown or $\Delta p = p -$ TWL otherwise.

For a discharge greater than the submergence discharge, V_p may be approximated as q/D . However, this is usually not a concern since the flow is subcritical. Under these conditions the backwater is also likely to be small, since the embankment is no longer a control.

Other flow characteristics which may be desirable to know are the critical velocity and critical depth at the crown of the road. These values are given by the following formulae:

$$V_c = \left(\frac{2}{3} gH\right)^{1/2} \quad \text{-----Equation (8)}$$

$$y_c = \frac{2}{3} H \quad \text{-----Equation (9)}$$

Note that these formulae are only valid for a free outfall type of flow i.e. $D/H < 0.76$.

5 SCOUR PROTECTION

5.1 GENERAL

When designing floodways, the protection works against scour damage is a prime consideration. Scour damage can occur at the batter slope and the shoulders of the floodway, but is also present on the road surface. Protection techniques against scour include:

- Appropriately designed rock protection
- Pump-up concrete revetment mattresses
- Cut-off walls (end walls)
- Rock fill below embankment
- Cement stabilised batter slope / embankment fill
- Cement stabilised subgrade / basecourse
- Two-coat bituminous seal.

When a floodway is founded on a creek bed subject to bed load sediment transport movements, appropriate investigative works must be carried out in order to ensure the stability of the base of the floodway.

The level to which scour protection is included on the ramps may be chosen to be higher than the design flood level. This is in order to prevent damage to the floodway for larger than design flows. It may not be practical or cost-effective to protect a floodway for extreme flood events, so a cost-benefit analysis should be carried out to compare the cost of additional scour protection to the cost of on-going maintenance and repair works, to determine the appropriate extent of protection (that is, to compare the cost of regular minor damage and ongoing maintenance works, to the cost of catastrophic failure, road closure and total reconstruction).

A hydraulic analysis to examine the failure mode of the structure is an integral part of the floodway design.

5.2 TYPES OF SCOUR

When designing floodways, scour protection works are a prime consideration. The locations where scour hazard can occur, in order of severity are (see Figure 5.1):

- (a) Toe of the downstream batter slope
- (b) Surface of batter slope
- (c) At the edge of downstream shoulders
- (d) On the road surface
- (e) On the upstream batter slope

The causes of scour at these positions are:

- (a) Due to impinging super-critical velocity at the toe of the batter slope
- (b) Due to the drag/shear resistance on the batter slope
- (c) Due to an uplift force caused by the embankment geometry
- (d) Due to shear/drag resistance on the running surface
- (e) Due to approach velocity.

The methods used to protect these locations are detailed in Sections 5.4 and 5.5.

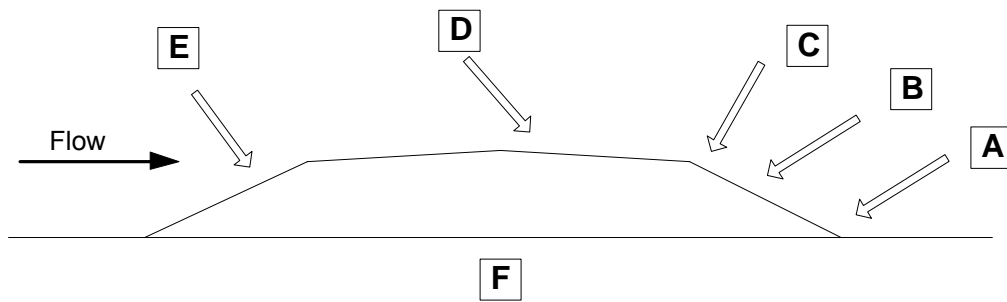


Figure 5.1 – Scour Locations on a Typical Floodway

Additionally, scour below the floodway can cause failure (F). This scour is caused by either piping or riverbed instability due to sediment transport phenomena.

5.3 LIMITING FLOODWAY SCOUR POTENTIAL

Scour protection requirements can be limited by intelligent hydraulic design of the floodway. Any attempt to reduce the scour protection requirements must be weighed against serviceability requirements, site conditions and construction costs, in order to achieve the most cost-effective solution.

It is evident that an efficient flood crossing is one designed to become submerged at a low flow (yet remain trafficable to meet the serviceability criteria) and will thereby have lower flow velocities and require less scour protection. A crossing with these characteristics is one that allows the tailwater to rise such that $D/H \geq 0.76$ at a low flow. This can be achieved by one or both of the following:

- Passing sufficient flow under the road via a culvert or bridge and allowing the tailwater to rise to a high level before the floodway becomes operative, or
- Keeping the floodway low and causing the floodway to cease to act as a hydraulic control at a relatively low upstream water level.

It should be noted that during the early stages of overtopping relatively high velocities may be present and thus slope stability should be a design consideration. The designer should also note that the maximum design flow rarely corresponds to the peak floodway velocity. The peak velocity typically occurs for some discharge less than the maximum design flow. Consideration of the volume of water involved at these early stages is also required to ensure that the design is not overly conservative (very small flows typically do not cause significant damage, even at high velocities).

Also, where flood crossings are constructed in isolation, upstream drainage channels (table drains) and culverts under the road should be provided to prevent ponding on the upstream side of the flood crossing and thereby preventing water entering the road pavement and causing subsequent failure.

The risk of damage to the downstream shoulder can be reduced by rounding the shoulder as much as possible, to avoid the generation of negative pressures at the change of flow direction. A radius of approximately 3.3m is recommended.

Flow through the embankment can lead to high uplift pressures under impervious types batter slope protection such as concrete slabs and pump-up revetment mattresses. Relief holes are required to allow drainage through the protection system and avoid pressure build-up. Dumped graded rock and gabion mattresses are not impervious and pressure build-up is unlikely to be a problem.

Leakage at the upstream side of a concrete cut-off wall can lead to significant pressures acting on the upstream face of the wall. Destabilising negative pressures can also result at the downstream shoulder due to abrupt changes in grade. If these pressures exceed the passive resistance of the soil wedge at the shoulder, failure of this wedge may occur.

Significant forces can act on upstands near the downstream shoulder (such as kerbs and guardrail posts). These high forces promote localised scour damage, which can act as a starting point for progressive scour damage by other means. Upstands should be avoided wherever possible.

It is also necessary for the designer to identify and predict the failure mode(s) for flows in excess of the adopted design flows so that temporary repair work can be carried out expeditiously.

5.4 PAVEMENT DESIGN

Two types of pavement are generally used in floodways:

- **Stabilised base course:** is typically used for floodways in areas where periods of inundation are relatively short (eg. Less than 30 hours per year), which are not subjected to heavy traffic in submerged conditions.

The base course is generally stabilised with cement and has a two coat seal. The stabilisation aids the bond between the seal and the base course and prevents the seal being stripped off, as a result of flow drag, while the road is inundated.

- **Concrete pavement:** is typically used where periods of inundation are long and the road is subjected to heavy traffic in wet conditions.

Concrete pavements and pavements with stabilised base courses should be designed in accordance with the Austroads Pavement Design Guide (1992).

All floodways with a flexible pavement should be constructed with a cement-stabilised basecourse and a two-coat seal.

The choice of pavement type is based on consideration of the traffic volume during wet and dry conditions, the frequency and duration of overtopping, the risk of scour damage, and the cost of construction and maintenance. Concrete floodways are typically used only in very high risk situations or in urban locations where traffic volumes are consistently high.

5.5 EMBANKMENT BATTER PROTECTION

The need for upstream protection will depend upon the velocity of flow approaching the floodway, the time the floodway is submerged, and the skew of the floodway to the direction of flow. When upstream protection is provided to protect against high approach velocities, it is not generally necessary to protect the full height of the batter, but only the road shoulder and the top of the batter. However, for floodways that are submerged for long periods, it is usual to provide similar protection on the upstream batter to that provided on the down stream batter.

Downstream protection of floodway embankment batter slopes may be either flexible or rigid. All protection should sit flush with the road pavement at the shoulder to avoid high pressures resulting at any sharp steps or grade changes. Examples of flexible and rigid protection are listed below.

It should be noted that MRWA generally prefer the use of dumped graded rock. Other systems should only be considered where availability is low, or where a significant cost saving can be achieved. The advice of the MRWA Waterways Section should be sought whenever a proprietary scour protection product is proposed for use.

Flexible Protection

- **Dumped graded rock (riprap),** defined as graded stone dumped upon a prepared slope. In most areas dumped rock is the least costly type of protection. A suitable length toe (typically 1.0-1.5 times the embankment height) should be provided at the base of the rock to protect the embankment against the high velocities at the change of grade.
- **Hand placed graded rock,** which is inferior to dumped rock, is seldom used today.
- **Rock mattresses (gabion mattresses)** are rock placed in wire baskets or in wire covered mats. Wire enclosed rock is generally used in locations where suitable materials for dumped rock are not readily available or are uneconomic. The size of rock should be larger than the openings in the wire enclosure, and a suitable length toe is required as above. The wire used should be PVC coated to avoid corrosion.
- **Flexible mats** comprise individual small high-density concrete blocks cast onto a geotextile loop matting. Each mat is generally about 5m by 2.5m and protection is provided by laying the mats

side by side with an overlap. These are proprietary products and the designer should refer to the manufacturer's technical literature for advice on their application and installation.

- **Flexible pump-up revetment mattresses** are concrete filled nylon mattresses where the concrete flows into discrete segments that are largely independent once the concrete has set, providing a degree of flexibility. These are proprietary products and the designer should refer to the manufacturer's technical literature for advice on their application and installation.
- **Vegetative cover** can form an effective scour protection system for floodways where the embankment is low and the approach velocity is low (less than 1.0 m/s). This type of protection is only appropriate in regions where adequate vegetation cover exists all year round (typically humid northern region). The peak floodway velocities should also be kept low. This is not recommended for use as the primary scour protection system, but is useful as an additional protective measure.

Rigid Protection

- **Grouted rock** is dumped or hand placed rock with the voids filled with mass concrete. The concrete should be sufficiently fluid to fill all voids over the full depth of the rock layer. It is generally used in locations where stone of a size suitable for other forms of protection is not economically available. It is also useful where only a small depth is available for construction of rock protection (such as over culvert pipes) or where access to construct larger rock is difficult.
- **Rigid pump-up revetment mattresses** are nylon mattresses into which a small aggregate concrete is pumped. These are proprietary products and the designer should refer to the manufacturer's technical literature for advice on their application and installation.
- **Concrete slab** protection is plain or reinforced concrete slabs poured or placed on the surface to be protected. This type is not often used due to its high cost, but may be warranted at crossings subject to extended periods of inundation. It may also be warranted in exceptionally high velocity situations, where other types of protection are inadequate.

Rigid protection is susceptible to undermining by scour, especially at the toes of batters, and should not be used unless the design engineer is confident that scour will not occur. Combinations of flexible and rigid systems may also be considered.

The use of a concrete cut-off wall at the downstream shoulder is recommended when high velocities are expected at the shoulder. The purpose of this wall is to prevent scour damage at the shoulder from progressing into the road pavement. These walls are typically 0.50-0.75m deep and 0.20-0.30m wide and are generally constructed of low strength mass concrete.

Where necessary a permeable geotextile filter should be placed between the embankment fill and the flexible scour protection. A graded sand/gravel filter may also be used. A filter layer is required if the rock and embankment fill do not meet the criteria below. If the criteria are not met, a geotextile filter, or sand/gravel filter layer that does meet the criteria is required (typically 0.15-0.20m thick).

$$5 \leq \frac{d_{15}(\text{upper})}{d_{15}(\text{lower})} \leq 40 \qquad \frac{d_{15}(\text{upper})}{d_{85}(\text{lower})} \leq 5$$

Where: d_n = The size of rock in a sample of which n% is smaller.

Design tables for dumped graded rock and gabion mattresses are provided in Tables 5.1, 5.2 and 5.3 below.

Velocity (m/s)	Class of Rock Protection, W_c (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 5.1 – Design of Rock Slope Protection

Rock Class	Rock Size (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
	0.20	10	90
¼ tonne	0.75	500	0
	0.55	250	50
	0.30	35	90
½ tonne	0.90	1000	0
	0.70	450	50
	0.40	100	90
1 tonne	1.15	2000	0
	0.90	1000	50
	0.55	250	90
2 tonne	1.45	4000	0
	1.15	2000	50
	0.75	500	90
4 tonne	1.80	8000	0
	1.45	4000	50
	0.90	1000	90

Table 5.2 – Standard Classes of Rock Slope Protection

Thickness (m)	Rock Fill Size		Critical Velocity (m/s)	Limit Velocity (m/s)
	(mm)	D ₅₀ (mm)		
0.15-0.17	70-100	85	3.5	4.2
	70-150	110	4.2	4.5
0.23-0.25	70-100	85	3.6	5.5
	70-150	120	4.5	6.1
0.30	70-120	100	4.2	5.5
	100-150	125	5.0	6.4

Table 5.3 – Indicative Thickness of Rock Mattresses

5.6 PROTECTION OF ASSOCIATED STRUCTURES

Waterways structures associated with floodways, such as bridges and culverts are often subject to high velocities and must be adequately protected against scour damage. This section deals with the protection of drainage culverts at floodways. The protection of bridges and major culverts are not described here.

Drainage culverts represent a scour hazard to the overall crossing, as the headwalls and wingwalls often involve sharp angle changes and drop-offs. High flow pressures can result at such locations, causing localised scour damage that can lead progressively to major scour damage and embankment failure. Drainage culverts are also susceptible to undermining, which can lead to failure of the culvert end treatments and large-scale movement of the pipes.

Immediately adjacent of the wingwalls and headwalls, a layer of nominal 0.20-0.30m diameter grouted rock is recommended to prevent scouring behind the end treatments. This is applicable at the downstream end treatments, and also upstream when the approach velocities are high. This rock should be tied into the concrete cut-off wall (if any) and should be overlapped with the two-coat bituminous seal.

At the culvert inlet and outlet, concrete downturn walls should be constructed at the ends of the culvert apron slabs. These walls are typically 0.50-1.0m deep and are required to prevent the erosion of the culvert bedding material from under the apron and base slabs.

At the end of the downstream apron slab, a 3.0-5.0m long layer of rock protection might need to be placed over the full width of the apron slab, depending on the outlet velocity, the size of culvert and the apron length. The rock should be sized using the culvert outlet velocity, including consideration of the reduction in velocity over the length of the apron.

5.7 TYPICAL FLOODWAY DESIGNS

Typical floodway scour protection designs are presented below. These floodway types are taken from a MRWA rural road-upgrading project.

These typical sections are intended as a guide only. The type, extent and thickness of rock protection; the use of geofabrics; the depth of cut off wall; the use of concrete slab batter protection or rock mattresses; and other issues should be considered on a site-specific basis on the advice of the MRWA Waterways Section and the guidance of relevant literature.

Type 1 Floodway (Figure 5.2): Floodway Type 1 consists of a cement-stabilised pavement with a two-coat seal, and rock protection to the downstream batter slope with a geofabric underlay.

The geofabric provides some resistance to scouring of the pavement due to the high pressure at the road shoulder, but is suitable for low velocities only. The Facing rock protection does not have a toe and is also suitable for low velocities only.

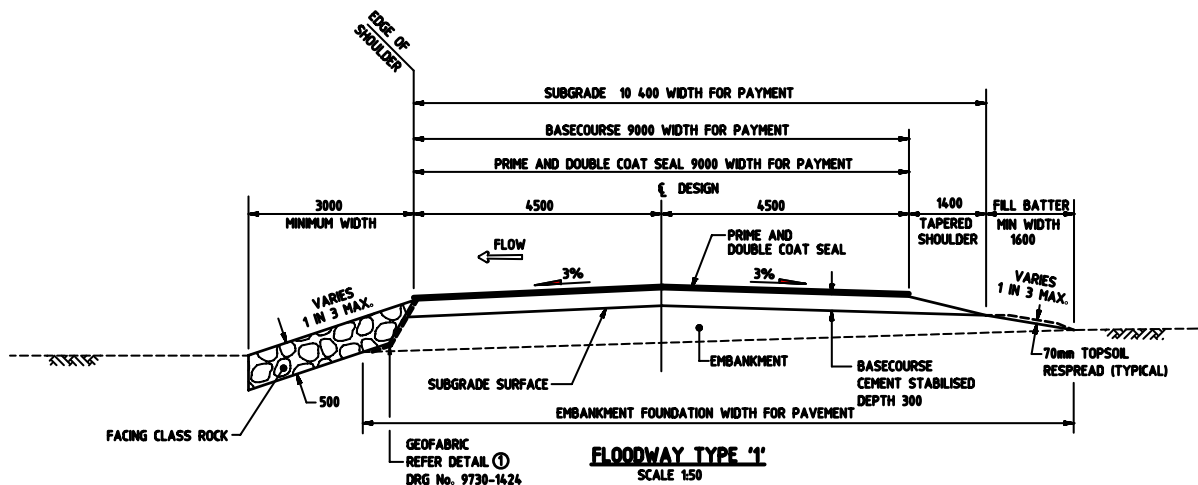


Figure 5.2 - Floodway Type 1

Type 2 Floodway (Figure 5.3): Floodway Type 2 is similar to Type 1, but provides increased scour protection at the shoulder. The concrete cut-off wall provides improved resistance to scour damage at the shoulder, and the toe of the rock protection improves the stability of the batter slope rock protection and decreases the risk of scour downstream of the floodway. The two-coat seal should overlap the concrete cut off wall as shown.

Floodway Type 2 is suitable for medium velocities.

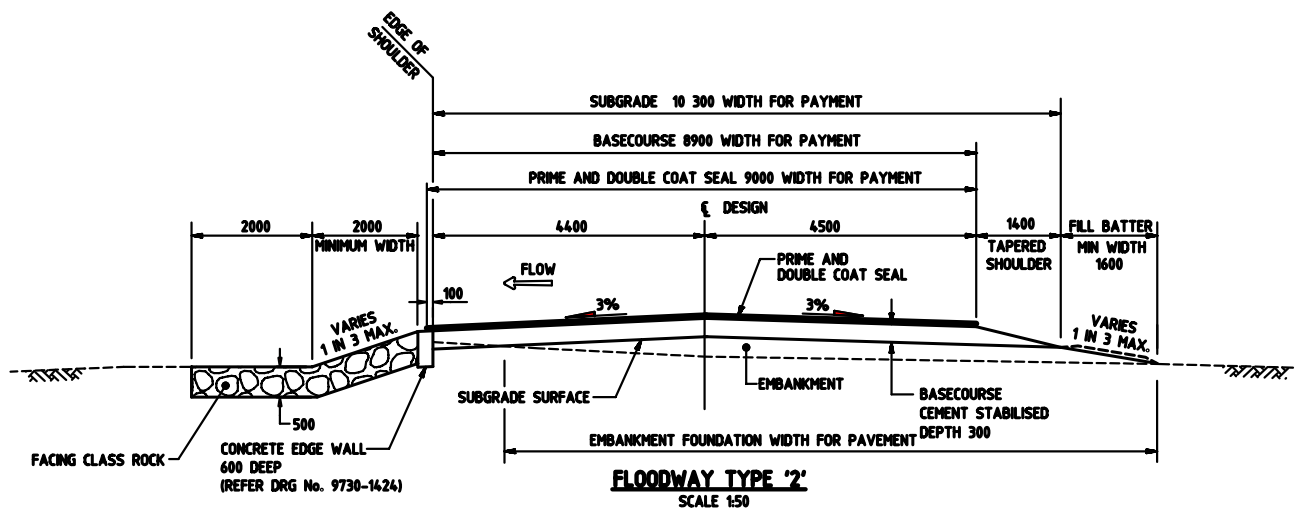


Figure 5.3 - Floodway Type 2

Type 3 Floodway (Figure 5.4): Floodway Type 3 is similar to Type 2, but involves heavier and thicker rock protection, and a deeper concrete cut-off wall. The two-coat seal should overlap the concrete cut off wall as shown.

Type 3 is suitable for higher velocities.

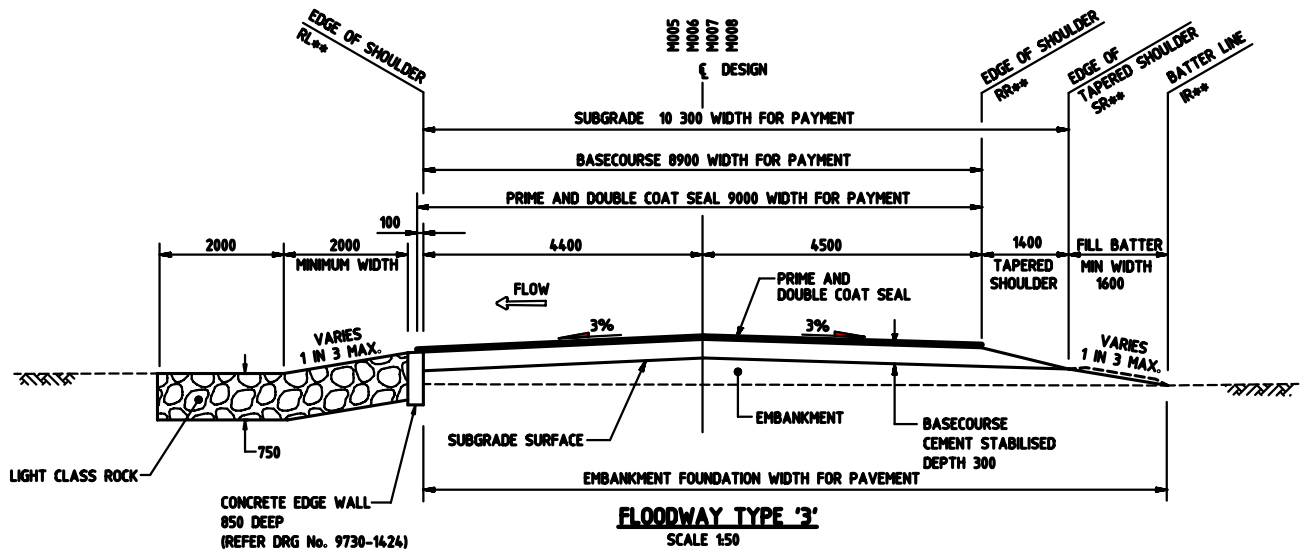


Figure 5.4 - Floodway Type 3

6 REFERENCES

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APPENDIX A
Nomenclature & Terminology

NOMENCLATURE

The symbols used throughout this document for calculations taken from Major are defined below:

C_f	-	Broad crested weir coefficient (free flow coefficient of discharge)
D	m	Depth of flow above embankment downstream
d_n	mm	Size of rock in a sample of which n% is smaller
$(D/H)_{trans}$	-	The value of D/H at which transition from plunging flow to surface flow takes place
E	m	Specific energy of flow (general)
E_2	m	Specific energy of flow at flood crossing crest
g	m/s^2	Acceleration due to gravity = $9.81 m/s^2$
H	m	Upstream head relative to flood crossing
h	m	Depth of flow above embankment upstream
h_1^*	m	Backwater = $H + p - TWL$
K	-	Coefficient of proportionality for calculating V_m in Equation 7
L	m	Floodway length
l	m	Width of flood crossing crest
n	-	Manning's roughness coefficient
p	m AHD	Flood crossing crest elevation above datum
Q	m^3/s	Total discharge
θ	$1/n$	Angle of batter from horizontal
q	m^2/s	Discharge per unit width of flow
Q_u	m^3/s	Flow under the embankment (i.e. through bridge or culvert)
S	m/m	Generalised slope
S_b	1:n	Slope of the batter
S_p	1:n	Crossfall of the pavement
TWL	m AHD	Tailwater level
$USWL$	m AHD	Upstream water level
V_1	m/s	Average approach velocity of the flow
V_b	m/s	Peak velocity on the batter
V_{bo}	m/s	Maximum batter velocity in the regime of $TWL > p$
V_{bu}	m/s	Maximum batter velocity in the regime of $TWL \leq p$
V_c	m/s	Critical velocity
V_m	m/s	Maximum velocity obtainable for a given discharge and head drop using the Bernoulli Equation
V_p	m/s	Peak velocity on the pavement
y_c	m	Critical depth
Δp	m	Elevation difference between crest and either road shoulder level or tailwater level as appropriate.

TERMINOLOGY

The following definitions are used in this report:

Critical depth – The flow depth for which the specific energy of the flow is minimum. For broad crested weirs, the critical depth typically occurs at or near the road crown, on roads with a two-way crossfall.

Critical flow – Floodway flow where the flow depth is less than the critical depth at some location on the floodway embankment cross-section.

Culvert invert level – the lowest point at the internal face of a pipe culvert, or at the top surface of the base slab for a box culvert.

Culvert obvert level – the highest point at the internal face of a pipe or box culvert.

Floodway invert level – The lowest point on the road centreline at the floodway.

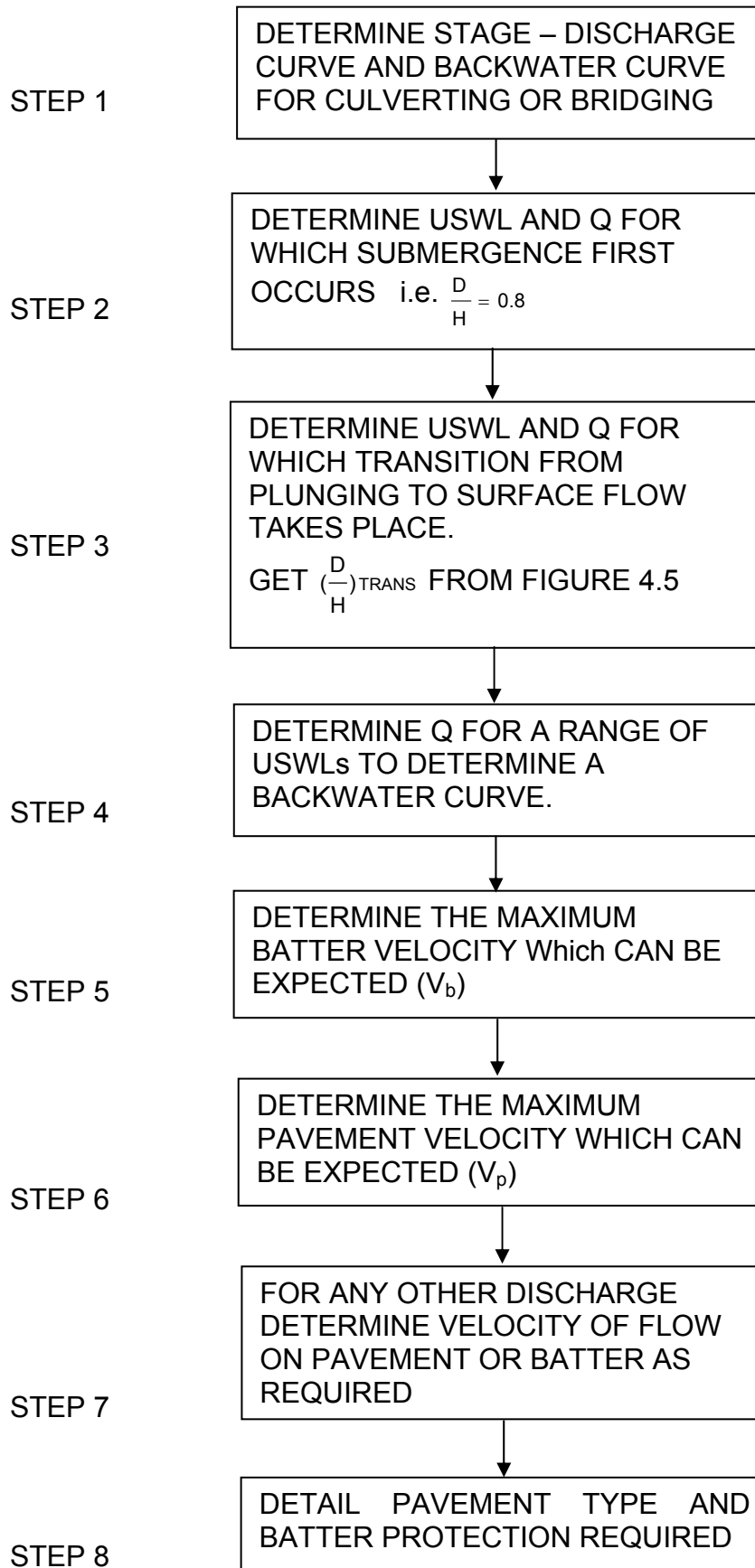
Submerged flow – Floodway flow where the flow remains sub-critical over the entire floodway cross-section, and the floodway ceases to act as a hydraulic control. That is, the flow depth is greater than the critical depth at all locations on the floodway cross-section.

Unsubmerged flow (free flow) – Floodway flow where the flow is critical at some location on the floodway cross-section.

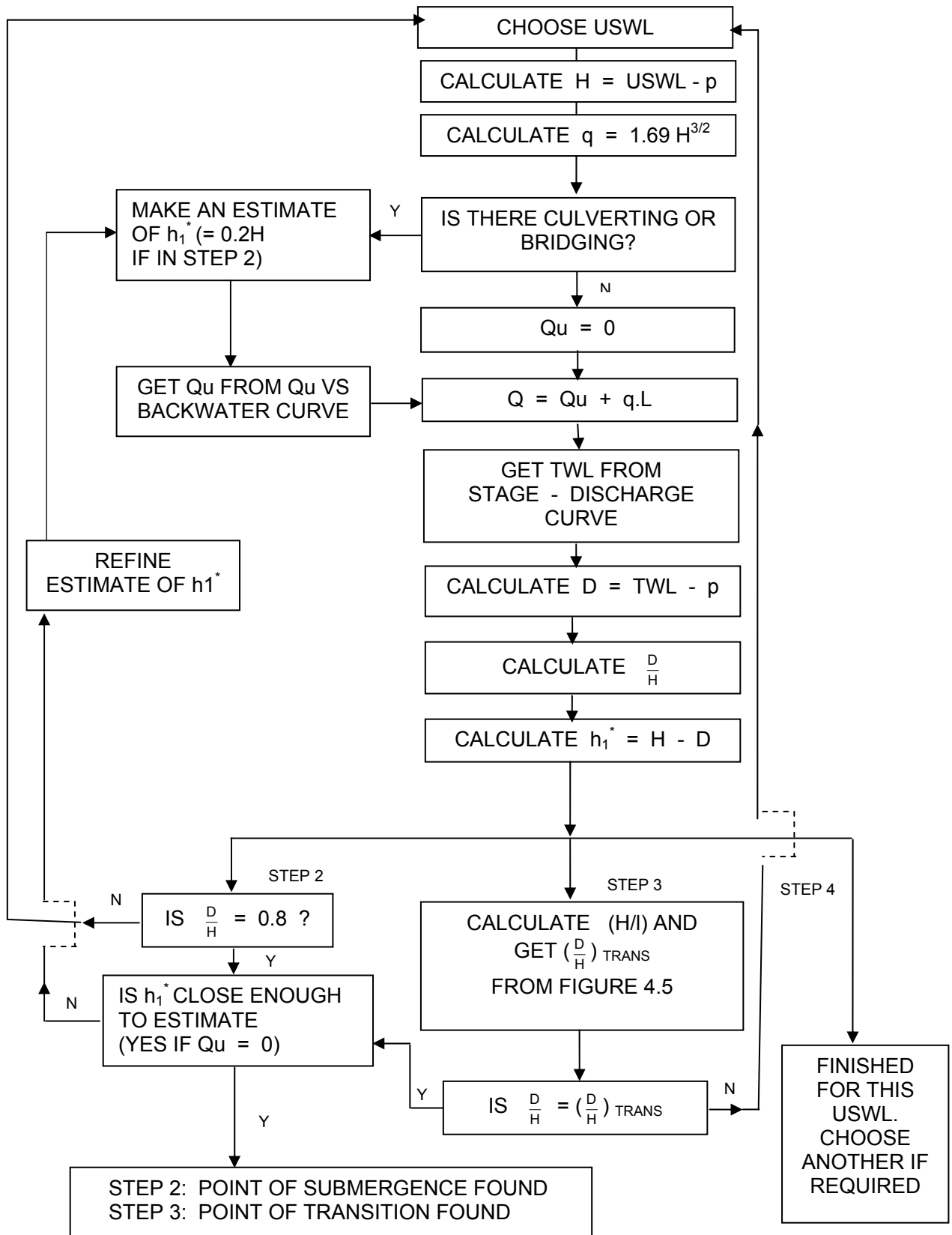
APPENDIX B
Manning's 'n' Roughness Coefficients

Values of Manning's 'n' for natural Streams		
Reproduced from the Austroads Waterways Design Manual (1994) Table 4.1		
Minor Streams (Surface width at flood stage less than 30m)		
(a)	Fairly regular section: <ul style="list-style-type: none"> - Some grass or weeds, little or no brush - Dense growth of weeds, depth of flow materially greater than weed height - Some weeds, light brush on banks - Some weeds, heavy brush on banks Note: For trees within channel, with branches submerged at high stage, increase all values above by 0.010 – 0.020.	0.030 – 0.035 0.035 – 0.050 0.035 – 0.050 0.050 – 0.070
(b)	Irregular sections, with pools or slight channel meander, increase values in (a) above by 0.010 – 0.020.	
(c)	Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage: <ul style="list-style-type: none"> - Bottom of gravel, cobbles and few boulders - Bottom of cobbles with large boulders 	0.040 – 0.050 0.050 – 0.070
6.1.1.1.1 Flood Plains		
(a)	Pasture, no brush: <ul style="list-style-type: none"> - Short grass - High grass 	0.030 – 0.035 0.035 – 0.050
(b)	Cultivated areas: <ul style="list-style-type: none"> - No crop - Mature row crops - Mature field crops 	0.030 – 0.040 0.035 – 0.045 0.040 – 0.050
(c)	Brush: <ul style="list-style-type: none"> - Scattered brush, heavy weeds - Light brush and trees - Medium to dense brush 	0.050 – 0.070 0.060 – 0.080 0.100 – 0.160
(d)	Trees: <ul style="list-style-type: none"> - Clear land with tree stumps, no sprouts - Same as above, but with growth of sprouts - Heavy stand of timber, a few fallen trees, little undergrowth and flood stage below branches - Same as above with flood stage reaching branches 	0.040 – 0.050 0.060 – 0.080 0.100 – 0.120 0.120 – 0.160
Major Streams (Surface width at flood stage greater than 30m)		
(a)	The 'n' value is less than for minor streams of similar description, because the banks offer less effective resistance. <ul style="list-style-type: none"> - Regular section with no boulders or brush - Irregular and rough section 	0.025 – 0.060 0.035 – 0.100

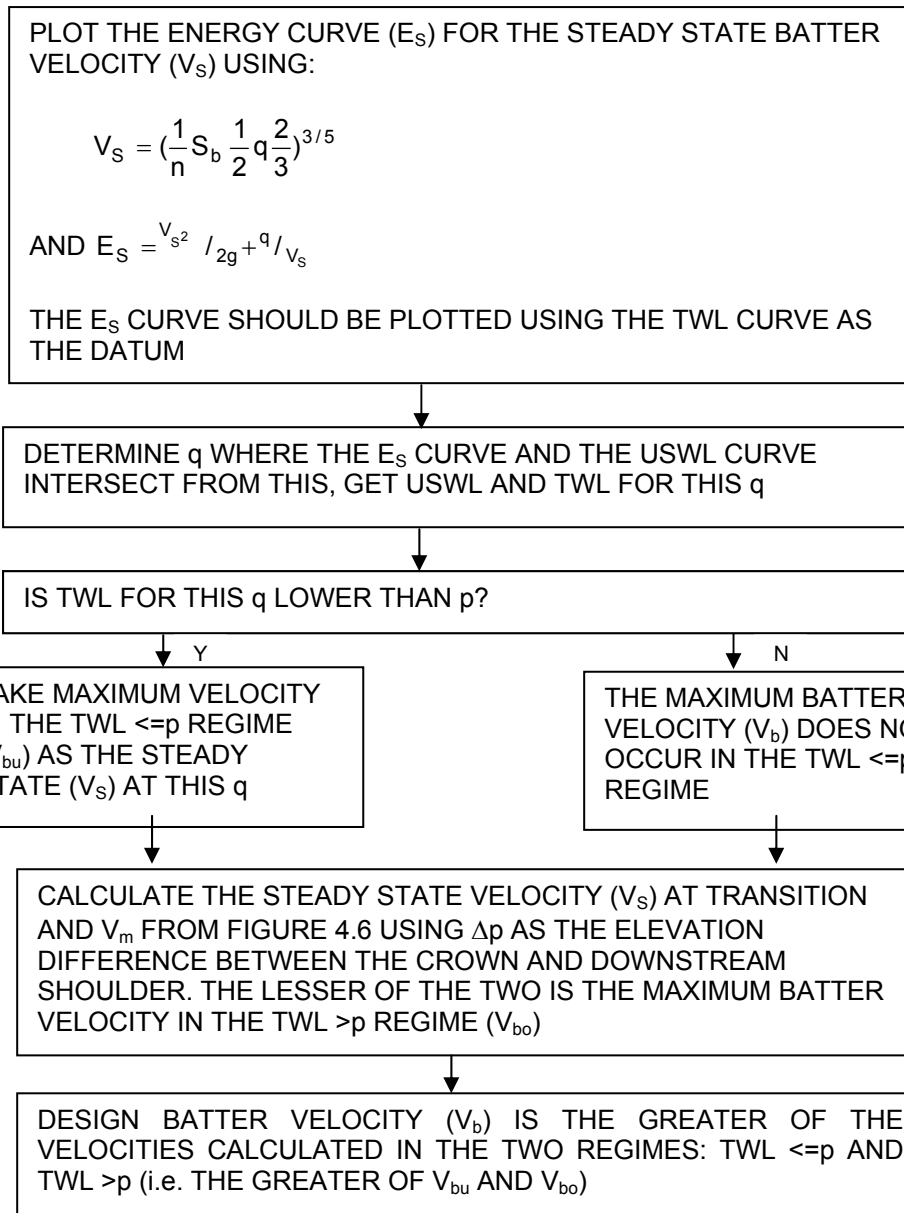
APPENDIX C
Calculation Flowcharts



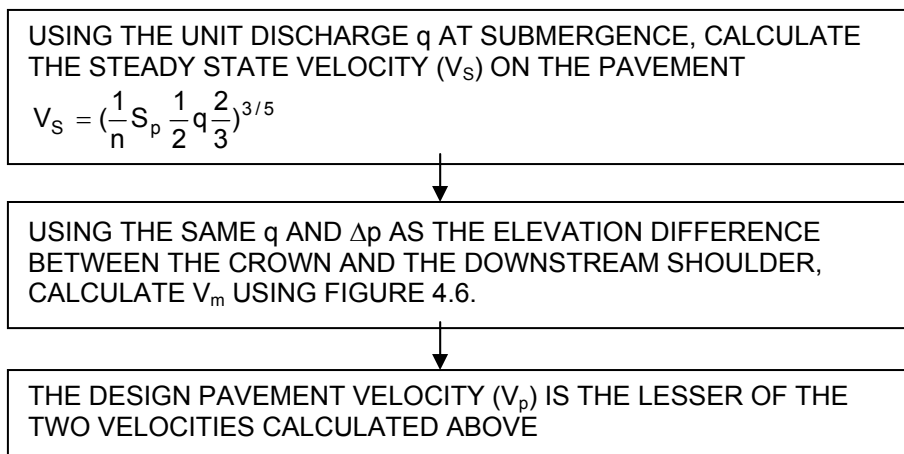
FLOWCHART 1 - Summary of Design Procedure



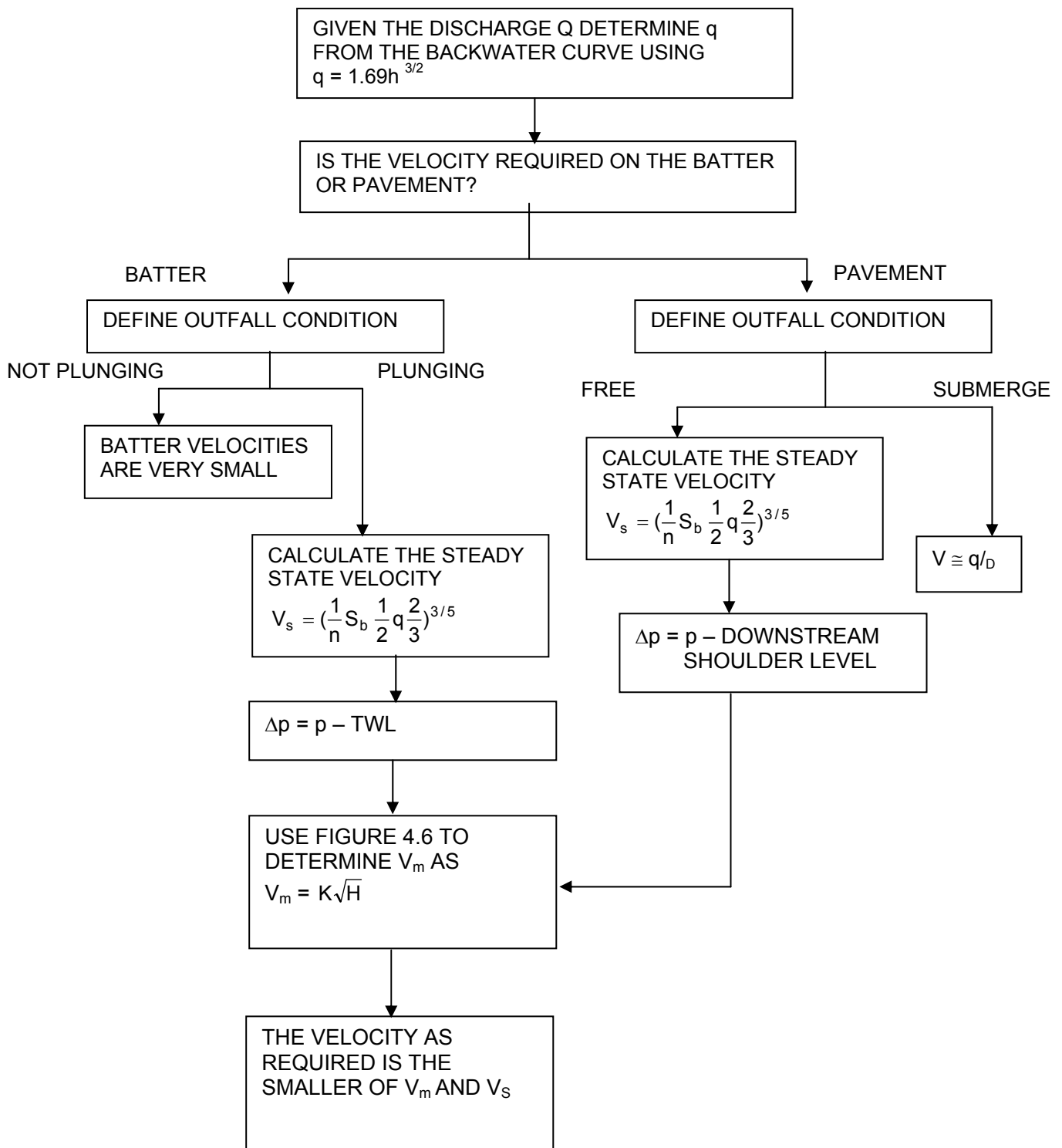
FLOWCHART 2 - Summary of Steps 2, 3 and 4 from Flowchart 1



FLOWCHART 3 - Summary of Step 5 from Flowchart 1



FLOWCHART 4 - Summary of Step 6 from Flowchart 1



FLOWCHART 5 - Summary of Step 7 from Flowchart 1

APPENDIX D

Worked Examples

The 'steps' mentioned in these examples refer to the design steps shown on the flowcharts presented in Appendix C.

Example 1: Seven Mile Creek Floodway

Figure D1 shows a long section of the Seven Mile Creek Floodway. A proposed replacement structure was a three barrel 1200 mm x 750 mm RCB culvert and floodway. Figure D2 shows the stage – discharge curve for the natural channel and floodplain.

One could expect a maximum discharge of only about 8m³/s through the culverts and hence, this can be ignored in the analysis, because it is such a small proportion compared to the flow which will go over the floodway.

The floodway characteristics are:

- Length = 300 m (i.e. distance between IP's)
- I.L. = 380.50 m
- Width = 9.0 m

The worksheet for this example is given in Figure D3, with each step in the procedure bracketed appropriately. It is advisable to follow the calculation of each quantity closely so that a thorough appreciation of the methodology is gained.

Initially the point of submergence was found (i.e. D/H = 0.8) and then the upper limit of the transition range was found. These are indicated as steps 2 and 3. The backwater for each of these two discharges has been plotted on Figure D2, the stage – discharge curve. A third point, the backwater for the theoretical zero discharge is already known, since at this discharge, the USWL is equal to the embankment (N.B. this is because the discharge under the road is being ignored). From these three points, a fairly good approximation of the backwater curve can be drawn in. If required, more points can be found by proceeding with step 4.

Once the backwater curve has been established, V_b and V_p can be determined.

Determination of V_b is made by plotting the E_s curve on figure D2 using the TWL as the datum.

Characteristics of the batter are:

- Slope = $S_b = 3:1$
- Manning's 'n' = 0.04

Using Equations 4 and 6, values of V_s and E_s have been calculated. These are shown below and are plotted in Figure D2. It should be noted that because flow under the road is ignored that $Q = q.L$.

q (m ² /s)	$Q = q.L$ (m ³ /s)	V_s (Eq. 4) (m/s)	E_s (Eq. 6) (m)
0.0	0	0.00	0.00
0.1	30	1.98	0.25
0.2	60	2.61	0.42
0.3	90	3.07	0.57
0.4	120	3.44	0.71
0.5	150	3.76	0.85

Table D1 - Values for E_s and V_s in Example 1

FIGURE 7
SEVEN MILE CREEK

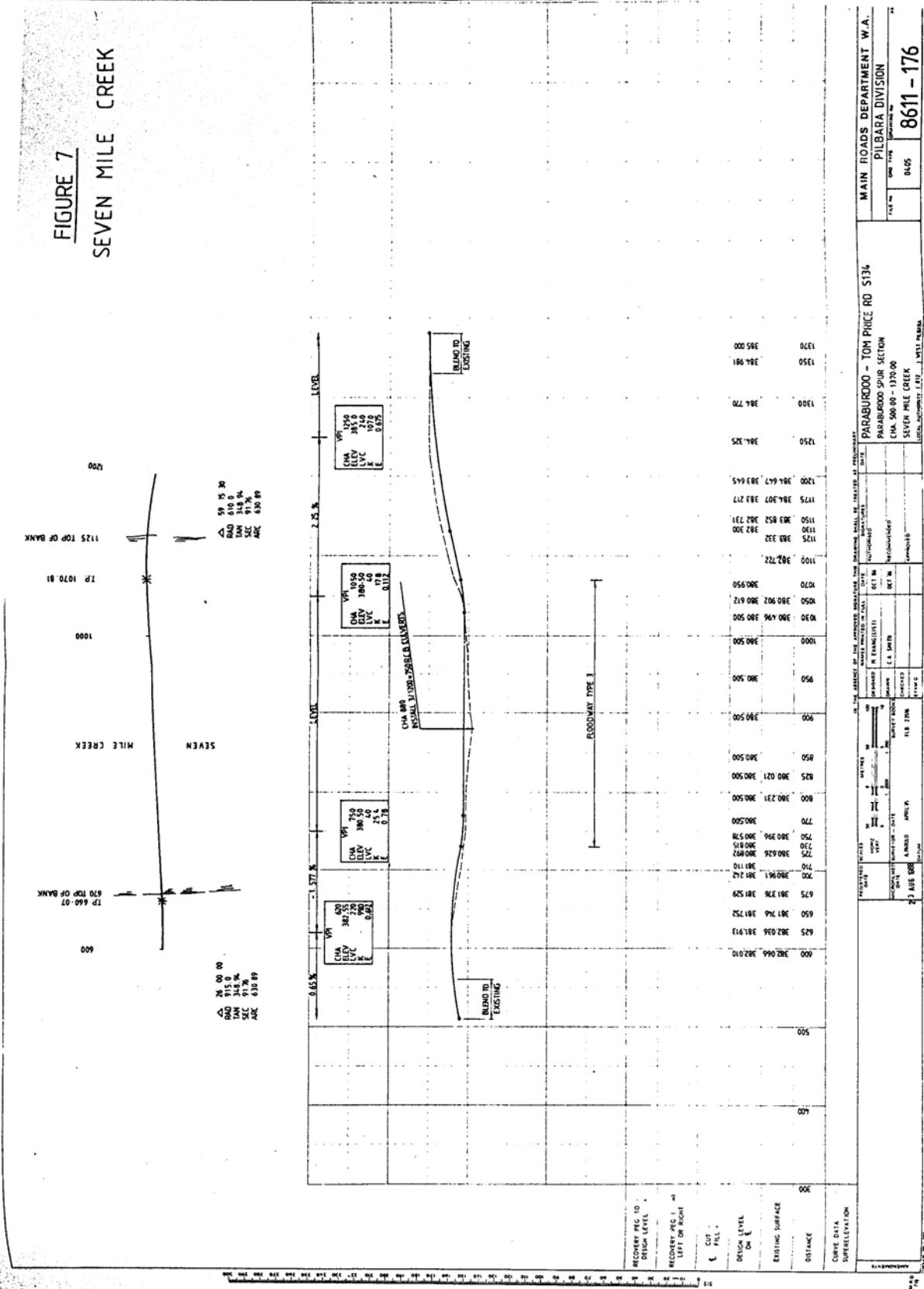


Figure D1 - Seven Mile Creek

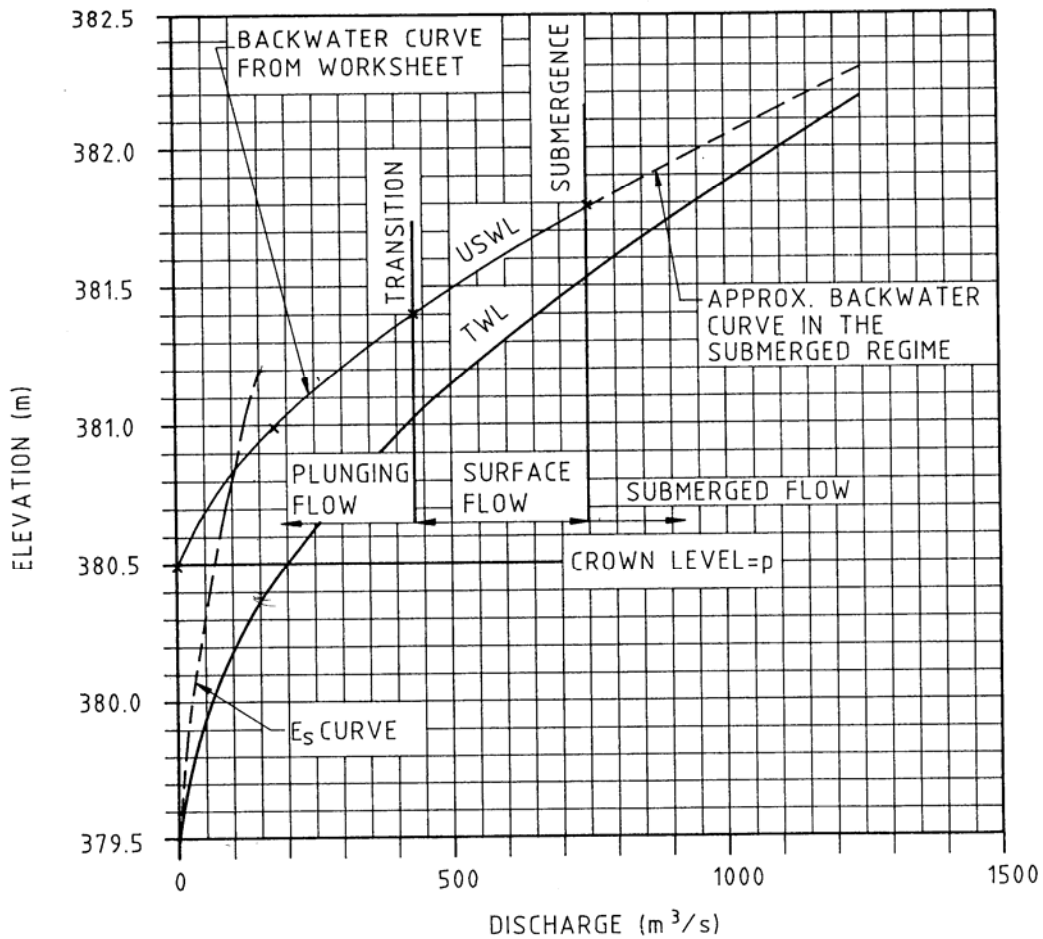


Figure D2 - Stage-Discharge Curve and Backwater Curve: Example 1, Seven Mile Creek

WORKSHEET FOR DETERMINING FLOODWAY BACKWATER

TITLE: SEVEN MILE CREEK

FIGURE 9

FLOODWAY LENGTH (L): 300m FLOODWAY I.L.(p): 380.50m FLOODWAY WIDTH (I): 2.0m

	USWL (m)	H =USWL-p (m)	q =1.49.H ^{1.48} (m³/s/m)	h _r * estimate (m)	Q _u (m³/s)	Q =q.L + Q _u (m³/s)	TWL (m)	D =TWL-p (m)	h _r * =H-D (m)	D/H	(D/H) trans	COMMENTS
STEP 2	381.60	1.10	1.950			585	381.28	0.78	0.32	0.71		Need D/H = 0.8 (Submergence) Increase H Need even higher upstream water level Close enough, Submergence at q = 2.505
	381.70	1.20	2.222			666	381.41	0.91	0.29	0.76		
	381.80	1.30	2.505			751	381.55	1.05	0.25	0.81		
STEP 3	381.30	0.80	1.209			363	380.89	0.39	0.41	0.49	0.58	Finding Transition. Need to Increase P/H Still need higher D/H; Try USWL = 381.40m Close enough. Transition at q = 1.443
	381.35	0.85	1.324			397	380.96	0.46	0.39	0.54	0.59	
	381.40	0.90	1.443			433	381.03	0.53	0.37	0.59	0.60	
STEP 4	381.00	0.50	0.594			178	380.45	-0.05	0.55			Finding h _r * given USWL = 381.00m

Figure D3 – Worksheet for Example 1

The intersection of the backwater curve and the E_s curve is at $Q = 105 \text{ m}^3/\text{s}$ and $q = 0.35 \text{ m}^2/\text{s}$. This occurs at a TWL below the crown height and thus V_{bu} is calculated from Equation 4 as 3.26 m/s. This must be compared with V_{bo} .

At transition, $q = 1.443 \text{ m}^2/\text{s}$
 $H = 0.90 \text{ m}$

V_s here is given by Equation 4 and is equal to 5.75 m/s.

V_m , given by Equation 7 using

$$\Delta p = p - \text{downstream shoulder level} = 0.135 \text{ m}$$

$$\text{and } \frac{\Delta p}{H} = \frac{0.135}{0.90} = 0.150$$

$$\text{is } V_m = K\sqrt{H}$$

where $K = 3.70$ from Figure 4.6

$$\text{i.e. } V_m = 3.70 \times \sqrt{0.90} = 3.51 \text{ m/s}$$

At transition the velocity, V_{bo} , is the lesser of V_s and V_m , i.e. $V_{bo} = 3.51 \text{ m/s}$.

This is greater than $V_{bu} = 3.26 \text{ m/s}$ for the $\text{TWL} \leq p$ regime. Therefore the design batter velocity, V_b is 3.51 m/s. This requires $\frac{1}{4}$ Tonne Class rock with a 1.00 m section thickness on the batter to provide adequate protection (Table 5.1).

On the pavement the velocity, V_p , will be the lesser of V_s and V_m when the flow is at the lower limit of submergence.

The pavement characteristics are:

$$\text{Manning's 'n'} = 0.015$$

$$\text{Slope} = S_p = 3\%$$

The steady state velocity is calculated from Equation 4

$$\begin{aligned} V_s &= \left(\frac{1}{n} S_p^{1/2} q^{2/3} \right)^{3/5} \\ &= \left(\frac{1}{0.015} 0.03^{1/2} 2.505^{2/3} \right)^{3/5} \\ &= 6.27 \text{ m/s} \end{aligned}$$

Once again using $\Delta p = 0.135$

$$\text{And } \frac{\Delta p}{H} = \frac{0.135}{1.30} = 0.104$$

$$V_m = K\sqrt{H}$$

where $K = 3.50$ from Figure 4.6

$$V_m = 3.50 \times \sqrt{1.30} = 3.99 \text{ m/s}$$

The maximum pavement velocity experience, V_p , is the lesser of V_s and V_m i.e. $V_p = 3.99 \text{ m/s}$.

Batter and pavement velocity calculation may also be made at any other desired discharge.

e.g. The velocities are required for a flow of 150 m³/s. Since the flow under the road is being neglected, q may be calculated as

$$q = \frac{Q}{L} = \frac{150}{300} = 0.50 \text{ m}^2 / \text{s}$$

On the pavement, the steady state velocity is

$$\begin{aligned} V_s &= \left(\frac{1}{n} S_p^{1/2} q^{2/3} \right)^{3/5} \\ &= \left(\frac{1}{0.015} 0.03^{1/2} 0.50^{2/3} \right)^{3/5} \\ &= 3.29 \text{ m/s} \end{aligned}$$

From the backwater curve, the USWL is read as 380.94 m and hence H = USWL – p = 0.44 m.

$$\frac{\Delta p}{H} = \frac{0.135}{0.44} = 0.307$$

and from Figure 4.6, K = 4.20

$$\text{i.e. } V_m = 4.2 \times \sqrt{0.44} = 2.79 \text{ m/s}$$

V_p is the lesser of V_m and V_s i.e. V_p = 2.79 m/s.

In calculating the velocity of flow on the batter, the tailwater level is below the crown level for Q = 150 m³/s (Figure D2). From the backwater curve, it is evident that the E_s line is above the USWL at Q = 150 m³/s, hence the flow is governed by the USWL and thus V_{bu} = V_m.

Figure 4.6 should be used with H = 0.44 m and

$$\begin{aligned} \Delta p &= p - \text{TWL} \\ &= 380.50 - 380.36 \\ &= 0.14 \text{ m} \end{aligned}$$

$$\Delta p/H = 0.318$$

thus K = 4.25 from Figure 4.6.

$$\text{i.e. } V_m = 4.25 \times \sqrt{0.44} = 2.82 \text{ m/s}$$

Thus the batter velocity, V_b = 2.82 m/s, for a flow of 150 m³/s.

This required Light Class rock with a 0.75 m section thickness on the batter to provide adequate protection (Table 5.1).

Example 2: Majors Creek Floodway

A proposed structure and Majors Creek Floodway was 3/2.7 m x 2.7 m RCBs with 215 m of floodway (Figure D4). The culverts are at an invert level of 72.00 m and the floodway invert level is 75.30 m. The backwater curve for the culverts was established using the AFLUX software and is plotted on the stage – discharge curve (Figure D5) up to the floodway invert level.

The point of submergence is not able to be found in this example (Figure D6). This is due to the limit of the stage discharge curve. Finding a point of submergence in this incidence is not important, since it is clear that it is only submerged for flows in excess of the 1000 year flow of about 500 m³/s. The pavement design would be subject to a lower level of serviceability than this and thus estimating V_p at submergence is not relevant.

The point of transition is found (step 2) as for Example 1, except that an extra parameter, h_1^* , must be iterated in addition to D/H.

Step 4 involves the computation of 3 points on the backwater curve; enough to enable the full curve to be drawn with sufficient accuracy.

The E_s curve has been drawn from the values in Table D2 using the following batter characteristics:

$$\text{Slope} = S_b = 3:1$$

$$\text{Manning's 'n'} = 0.03$$

In Table D2, H has been calculated from Equation 3 and the USWL read off Figure D5.

q (m ² /s)	H (Eq. 3) (m)	USWL (m)	V _s (Eq. 4) (m/s)	E _s (Eq. 6) (m)
0.0	0.00	75.3	0.00	0.00
0.1	0.15	75.45	2.35	0.33
0.2	0.24	75.54	3.10	0.55
0.3	0.32	75.64	3.64	0.76
0.4	0.38	75.68	4.09	0.95
0.5	0.44	75.74	4.47	1.13

Table D2 - V_s and E_s Values for Example 2

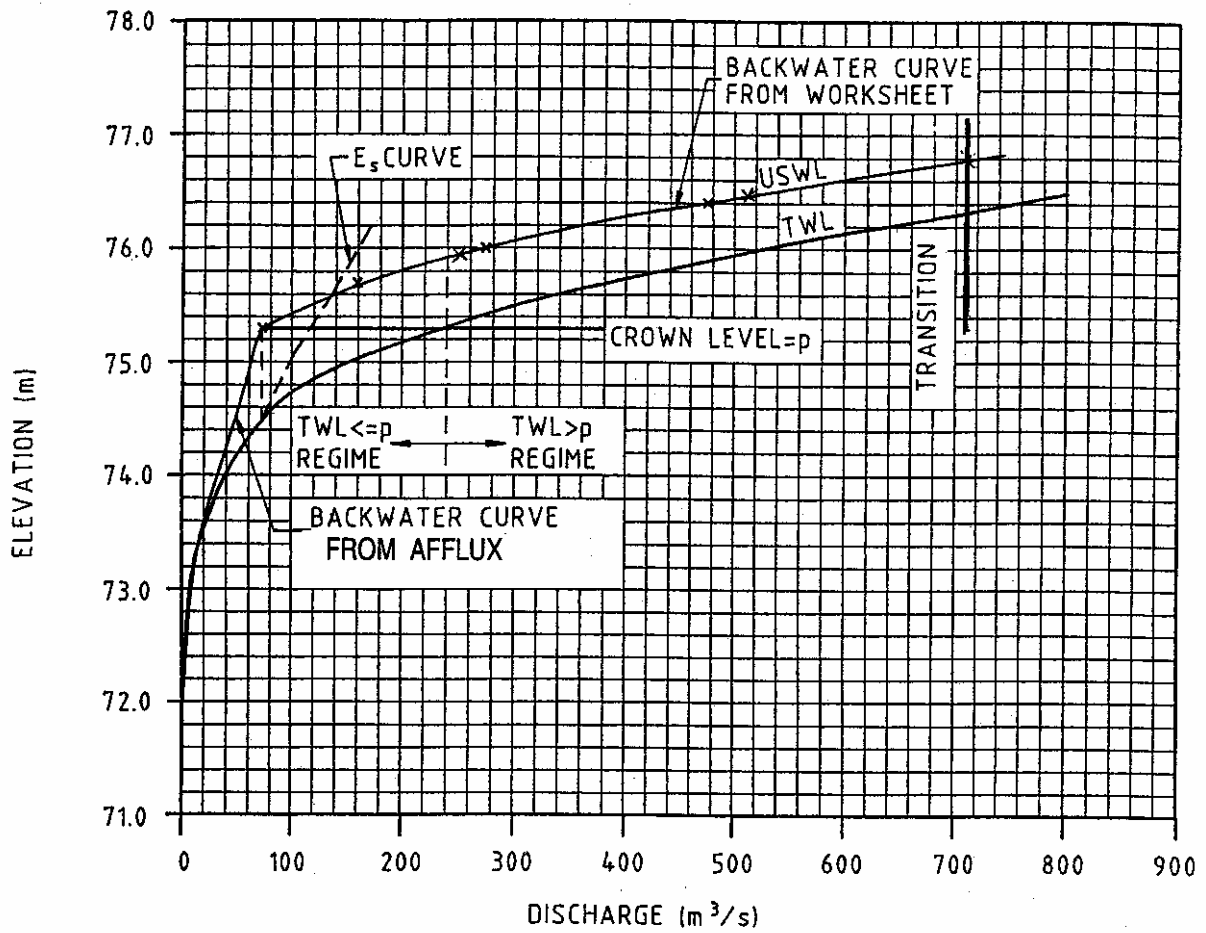


Figure D5 - Stage Discharge Curve and Backwater Curve - Example 2, Majors Creek

WORKSHEET FOR DETERMINING FLOODWAY BACKWATER

Majors Creek Floodway

Floodway length L (m): 215
 Floodway invert level (p) (m)
 AHD): 75.3
 Floodway Width (l) (m): 9.0

Step	USWL	H=USWL - p	$q=1.69H^{3/2}$	h_1^*	Qu	Q=qL + Qu	TWL	D=TWL -p	$h_1^*=H-D$	D/H	H/l	$(D/H)_{Trans}$	Comments
	m	m	m ³ /s/m	m	m ³ /s	m ³ /s	m AHD	m	m				
2	76.00	0.70	0.99	0.14	30.00	242.80	75.33	0.03	0.67	0.04			Need D/H=0.8, submergence, increase H
	76.50	1.20	2.22	0.24	39.00	516.64	75.95	0.65	0.55	0.54			Need even higher upstream water level
	77.00	1.70	3.75	0.34	46.00	851.38	76.58	1.28	0.42	0.75			This is well above the 1000yr flow. Cease calculation. Floodway will not be submerged.
3	76.80	1.50	3.10	0.55	60.00	727.52	76.35	1.05	0.45	0.70	0.17	0.68	h_1^* estimate not bad. Neet to increase D/H.
	76.78	1.48	3.04	0.48	56.00	710.21	76.30	1.00	0.48	0.68	0.16	0.67	Close enough. q at transition = 3.043 m ² /s
4	75.70	0.40	0.43	0.45	53.00	144.92	74.96	-0.34	0.74				Need to increase h_1^* estimate
	75.70	0.40	0.43	0.70	67.00	158.92	75.01	-0.29	0.69				O.K.
	76.00	0.70	0.99	0.60	62.00	274.80	75.45	0.15	0.55				Need to lower h_1^* estimate
	76.00	0.70	0.99	0.56	60.00	272.80	75.44	0.14	0.56				O.K.
	76.40	1.10	1.95	0.50	56.00	475.19	75.90	0.60	0.50				O.K.

Figure D6 – Worksheet for Example

The intersection of the E_s curve and the USWL curve is at:

$$Q = 136 \text{ m}^3/\text{s}$$

$$\text{USWL} = 75.60 \text{ m} \quad \text{From Figure D5}$$

$$\text{Thus } H = 0.30 \text{ m}$$

$$\text{And } q = 0.28 \text{ m}^2/\text{s} \quad \text{From Equation 3}$$

$$V_s = 3.53 \text{ m/s} \quad \text{From Equation 4}$$

Hence $V_{bu} = V_s = 3.53 \text{ m/s}$

At transition,

$$q = 3.043 \text{ m}^2/\text{s} \quad (\text{step 3})$$

$$H = 1.48 \text{ m} \quad (\text{step 3})$$

$$\text{And thus } V_s = 9.17 \text{ m/s} \quad (\text{Equation 4})$$

From Figure 4.6, using $\Delta p = 0.15 \text{ m}$,

$$\frac{\Delta p}{H} = \frac{0.15}{1.48} = 0.10$$

$K = 3.50$ from Figure 4.6

$$V_m = 3.50 \times \sqrt{1.48} = 4.26 \text{ m/s}$$

Thus V_{bo} is equal to the lesser of V_m and V_s , which is 4.26 m/s and V_b equals the greater of V_{bu} and V_{bo} , which is 4.26 m/s. It should be noted that this occurs for about the 1000 year flow. It would be practical to design for, in this instance, a 50-year flow, since the maximum velocities at transition and submergence are not likely to be obtained in the life of the structure.

$Q_{50} = 195 \text{ m}^3/\text{s}$ and at this discharge the TWL is below the crown, thus we are analysing in the $\text{TWL} < p$ regime.

At this flow E_s is greater than the USWL and consequently the USWL controls the velocity, i.e. $V_{bu} = V_m$. V_m is calculated using Figure 4.6 with,

$$\begin{aligned} \Delta p &= p - \text{TWL} \\ &= 75.30 - 75.14 \\ &= 0.16 \text{ m} \end{aligned}$$

$$\begin{aligned} H &= \text{USWL} - p \\ &= 75.80 - 75.30 \\ &= 0.50 \text{ m (i.e. } q = 0.6 \text{ m}^2/\text{s)} \end{aligned}$$

$$\frac{\Delta p}{H} = \frac{0.16}{0.50} = 0.32$$

$K = 4.25$ from Figure 4.6

$$V_m = 4.25 \times \sqrt{0.50} = 3.01 \text{ m/s}$$

Therefore the Q_{50} design batter velocity, $V_b = 3.01 \text{ m/s}$. The protection required on the batter is 1.00 m thick section of $\frac{1}{4}$ Tonne Class rock (Table 5.1).

The pavement velocity is the lesser of the V_s and V_m . Pavement characteristics are:

$$\text{Slope} = S_p = 3\%$$

$$\text{Manning's 'n'} = 0.015$$

$$\Delta p = 0.15 \text{ m}$$

$$V_s = \left(\frac{1}{n} S_p^{1/2} q^{2/3} \right)^{3/5}$$

$$= \left(\frac{1}{0.015} 0.03^{1/2} 0.44^{2/3} \right)^{3/5}$$

$$= 3.12 \text{ m/s}$$

$$\frac{\Delta p}{H} = 0.30$$

From Figure 4.6, $K = 4.20$

$$V_m = 4.20 \times \sqrt{0.50} = 2.97 \text{ m/s}$$

The design Q_{50} pavement velocity, V_p , is 2.97 m/s.

Example 3: Hydrology – Bridge 4179 – Chester Road over Penny Brook

The example shows the steps involved in minor hydrological investigations for small catchments. The example is taken from MRWA Waterway Report No. 411 (October 2001), '*Culvert Replacement Br 4179 Chester Rd over Penny Brook*'. The example is from a bridge waterways investigation, but is equally applicable to floodway design.

1. Catchment Properties

The catchment area should be identified from topological maps of the area. For small catchments, 1:100,000 scale maps are generally adequate. The catchment area, main stream length, average stream gradient (over the main stream length), and the percentage of land cleared are required. The age of the map should be considered when estimating the clearing percentage, as the amount of clearing will have increased in many cases.

For Bridge 4179, the catchment was identified from the 'Northam' 1:100,000 scale topographic map. An area of 13.2 km² and a main stream length of 5.5 km were determined. Using the contours on the map, the stream was found to fall 90m vertically over the main stream length, giving an average stream gradient of 16.4 m/km. By measuring the approximate forested area on the map, a clearing percentage of 90% was adopted.

2. IFD Information

Information for calculating the Intensity-Frequency-Duration (IFD) data for the catchment is sourced from Australian Rainfall and Runoff (AR&R) Volume 2. The basic intensities, geographical and skew factors are estimated from a series of maps and charts. The IFD figures are determined in accordance with AR&R Volume 1, however software programs are available to perform the calculations (such as IFD Design Rainfall Program by WP Software).

The IFD information for Bridge 4179 is summarised below:

$I_{2,1}$	= 14.70	mm / hr	F_2	= 4.80
$I_{2,12}$	= 3.10	mm / hr	F_{50}	= 16.75
$I_{2,72}$	= 0.83	mm / hr	Skew	= 0.64
$I_{50,1}$	= 28.90	mm / hr	Lat.	= 31.89°S
$I_{50,12}$	= 5.50	mm / hr	Long.	= 116.97°E
$I_{50,72}$	= 1.55	mm / hr		

The average annual rainfall (P) can also be sourced from AR&R Volume 2. For Bridge 4179, P was estimated to be 750 mm / year.

3. Rational & Index Flood Methods

Design flows for small catchments are generally estimated using the Rational and Index Flood Methods in accordance with AR&R Volume 2. Different empirical formulae are applied depending on the catchment location and soil type. Simple spreadsheets can be prepared to make calculation easier.

After calculating the time of concentration (t_c , from AR&R formulae for each region) for the catchment, the corresponding rainfall intensities must be interpolated from the IFD data. The AR&R methods allow estimation of flows for 2, 5, 10, 20, and 50 year ARI. The 100 year ARI can be estimated using log-log extrapolation if required.

The results of the Rational and Index Flood Method calculations for bridge 4179 are summarised below:

Flow Estimation for Bridge 4179 (m ³ /s)				
ARI	Rational Method		Index Flood Method	
	Soil Type 1	Soil Type 2	Soil Type 1	Soil Type 2
2	2	2	2	2
5	5	4	4	4
10	9	7	8	7
20	16	13	14	13
50	28	22	27	24

Soil Type 1 – Loamy Soils
Soil Type 2 – Lateritic / Loamy Sandy Soils

In this instance, a good correlation can be seen between the results from the Rational and Index Flood Methods for each soil type.

4. Design Flows

Design flows for the catchment are estimated based on the various hydrological methods employed (Rational Method, Index Flood Methods, run-off routing etc). A degree of conservatism should always be adopted in the estimation of design flows.

Where there is a good correlation between the methods employed, the choice of design flows is often obvious, as was the case for Bridge 4179. In such cases, the design flows can be estimated from the results without additional investigation. The design flows adopted for Bridge 4179 are summarised below. The 100-year design flow was estimated from the other design flows using log-log extrapolation.

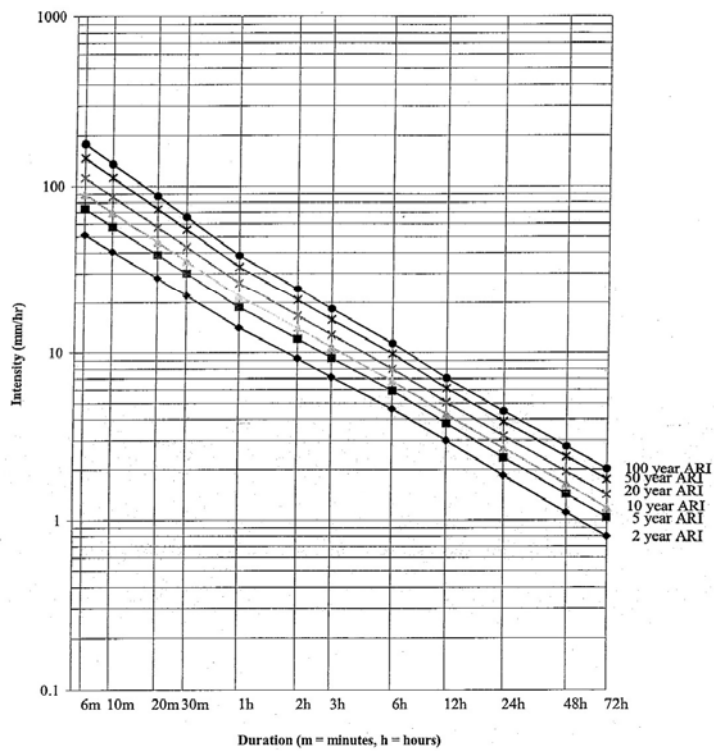
Bridge 4179 - Summary of Design Flows						
ARI (years):	2	5	10	20	50	100
Design Flow (m ³ /s):	2	5	9	16	28	43

Where there is poor correlation between the hydrological methods, it is useful to plot the results for each method on a log-log chart of ARI (x-axis) vs discharge (y-axis). From this chart, a line of best fit can be plotted to aid in estimating the design flows. The MRWA Waterways Section can be contacted for information on the accuracy of the various methods in particular regions, and for various catchment types and sizes.

5. Hydrographs

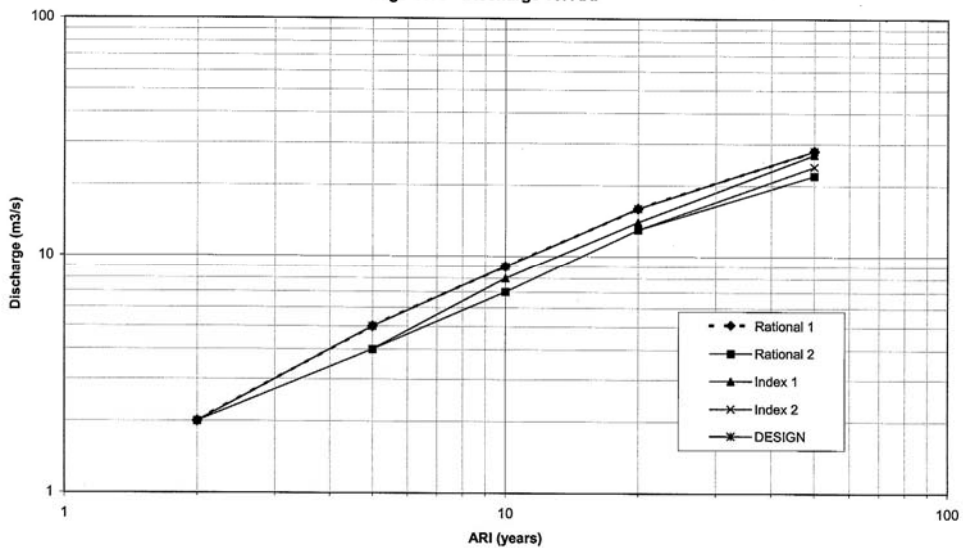
The Rational and Index Flood Methods do not provide hydrograph data. For cases where flow duration and time of closure information is critical, such as highways or main roads, runoff routing analysis should be carried out to calculate the flood hydrographs, or the MRWA Waterways Section should be contacted for advice on likely flow durations.

IFD Curve - Bridge 4179



SAMPLE IFD GRAPH

Bridge 4179 - Discharge vs. ARI



SAMPLE DISCHARGE vs ARI CHART