INVESTIGATION ON THE BEHAVIOUR OF FLOODWAY STRUCTURES UNDER EXTREME FLOOD LOADINGS

A Thesis submitted by

Isaac Greene

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ABSTRACT

Floodways are infrastructure used in road design to enable safe vehicular passage across waterways susceptible to flooding at relatively low average recurrence intervals. During the significant flood events of the past decade, repeat structural damage and consistent failure mechanisms have been observed, compromising the reliability of the rural road network and thus the resilience of communities. These failures are a result of the flood loadings encountered during extreme flood events, which are not currently considered in floodway design guidelines that typically rely upon hydraulic design principles.

The primary objective of this research was to evaluate the principal structural failure mechanisms of concrete floodways. To achieve this, an industry based survey and an experimental program were conducted, and a scaled finite element model created to compare findings. This enables the typical failure mechanisms for floodways to be determined. A numerical finite element model of a full scale standard engineering floodway type was then developed and parameters applied to simulate extreme flood conditions. Modelling was then expanded to cover five standard engineering floodway types enabling a refined structural design methodology to be deduced. The finite element models were then used to determine the worst case loading scenario, and to develop a floodway design guideline with an incorporated structural design methodology.

Limited research into the structural adequacy of concrete floodways had been undertaken; the key discovery of this research was the vulnerability of concrete floodway structures to impact loading during extreme flood events. The key research outcome was the conclusive design guideline which collated the research and subsequent investigations and provided a practical end-user outcome. Through the implementation of this design guideline more resilient floodway structures will result, thus improving the reliability of the rural road network as well as community resilience and safety.

CERTIFICATION OF THESIS

This Thesis is the work of Isaac Greene except where otherwise acknowledged, with the majority of the authorship of the papers presented as a Thesis by Publication undertaken by the Student. The work is original and has not previously been submitted for any other award, except where acknowledged.

Signed: [redacted] Date: 07/03/2022

Endorsed by:

Assoc. Prof. Weena Lokuge
Principal Supervisor

Prof. Karu Karunasena
Associate Supervisor

Student and supervisors' signatures of endorsement are held at the University.

STATEMENT OF CONTRIBUTION

Paper 1:

Greene, I., Lokuge, W., & Karunasena, W. 2020. “Floodways and Flood-Related Experiences: A Survey of Industry Experts and Asset Owners”, *Engineering for Public Works*, Vol. 19, pp. 69-75. ISSN 2652-6050. `https://issuu.com/ipweaqld/docs/3660___epw_september_2020_final`.

Student contributed 75% to this paper. Collectively W. Lokuge and W. Karunasena contributed the remainder.

Paper 2:

Greene, I., Gunasekara, C., Lokuge, W., & Karunasena, W. “Qualitative Investigations into Floodways Under Extreme Flood Loading”, *Journal of Flood Risk Management*. Submitted - February 2022, Currently Under Review (Q1).

Student contributed 60% to this paper. Collectively C. Gunasekara, W. Lokuge and W. Karunasena contributed the remainder.

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Student contributed 75% to this paper. Collectively W. Lokuge and W. Karunasena contributed the remainder.

Paper 5:
Greene, I., Lokuge, W., & Karunasena, W. "Floodway Design Guideline", *Bushfire Natural Hazard CRC and Institute of Public Works Engineering Australasia, Queensland Division (IPWEAQ)* - Submitted November 2020, Currently in Third Review Cycle.

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NOTATION

Symbol	Description	Units
A	Area	m^2
C	Cohesion	Pa
C_s/C_f	Submergence factor	Dimensionless
C_f	Coefficient of discharge free flow	Dimensionless
C_s	Coefficient of discharge flow with submergence	Dimensionless
C_l	Coefficient of lift	Dimensionless
C_d	Coefficient of drag	Dimensionless
D/H	Percent submergence	m/m
d	Stopping distance	m
E	Modulus of elasticity	Pa
F	Maximum frictional force	kN
g	Gravitational constant	9.81 m/s^2
H	Depth of water upstream of floodway	m
h	Depth of water above floodway	m
HW	Depth of floodway head water	m
K	Initial stiffness	kN/m
L	Length	m
l	Top width of road formation	m
M^*	Design bending moment	kN.m

m	Mass	kg
N	Normal force	kN
n	Mannings roughness coefficient	Dimensionless
P	Pressure	Pa
Q	Flow	m^3/s
R	Hydraulic radius	m
S	Stream hydraulic gradient	m/m
V	Volume	m^3
V^*	Design shear force	kN
v	Velocity	m/s
w	Channel width	m
Δ	Displacement	m
ϵ	Strain	Dimensionless
μ	Friction coefficient	Dimensionless
ρ_w	Density of water	1000 kg/m^3
ρ	Density	kg/m^3
σ	Stress	Pa
τ	Shear stress	Pa
θ	Angle of internal friction	Degrees

ABBREVIATIONS

ARI	Average Recurrence Interval
AS 1170	Structural Design Actions
AS 3600	Australian Concrete Design Code
AS 5100	Australian Bridge Design Code
FEA	Finite Element Analysis
FEM	Finite Element Model
LGA	Local Government Area
LVRC	Lockyer Valley Regional Council
REF	Review of Environmental Factors
2D	2-Dimensional
3D	3-Dimensional

TERMINOLOGY

Definitions of non-universal terms in the context of this study.

- Floodway: Infrastructure utilised in road design to facilitate the safe crossing of waterways during low rainfall events (depth of flow over the road is < 300 mm). Floodways are also referred to as fords and causeways in the international context.
- Boulder: The sedimentological definition of a boulder is a rock fragment greater than 256 mm (Shobe et al., 2021).
- Extreme flood event: A rainfall event with an average recurrence interval (ARI) of greater than 100.
- Structural adequacy: Has been designed adequately for durability, serviceability and strength.
- Resilience: “A community’s capacity to either absorb or adapt to a shock without changing its fundamental nature” (Mullett et al., 2015).

CHAPTER 1

INTRODUCTION

Flooding is a form of natural disaster that results from intense heavy rainfall associated with exceptional adverse weather patterns and tropical cyclone events (Sultana et al., 2016). Australia has a history of extreme weather fluctuations and variable climatic events such as flooding. It is reported that these extreme weather events have become more severe and frequent in the recent decade (Bromhead, 2021; BNHCRC, 2015; Nasr et al., 2021; Sassu et al., 2017). Furthermore, Sultana et al. (2016) summarises that in the past decade Queensland has experienced many unpredictable climatic events, including Cyclone Olga in 2010, Cyclone Yasi in 2011, the January 2011 flooding in South-East Queensland, Cyclone Oswald in 2013 and Cyclone Marcia in 2015. All of these events have caused widespread flooding and significant damage to the Queensland built environment.

Designing water crossings, particularly for flood immunity presents design challenges that engineers must successfully navigate to design flood resilient infrastructure. This research focuses on increasing the flood immunity of floodway road structures. Floodways are described as road infrastructure which water may flow over for short periods of time during flood, whilst remaining trafficable (Figure 1.1) (Department of Transport Main Roads, 2010). This is facilitated through improvements to the trafficable surface to enable safe and predictable vehicular passage. Floodways can be a favourable alternative to bridges and culverts, particularly in rural settings and on routes that do not service sufficient people to warrant a higher level of service (BNHCRC, 2015).

Figure 1.1: A typical rural concrete floodway.

Research into floodway design is currently limited and over the past decade, due to the increase in flood events, repeat structural damage and consistent failure mechanisms have resulted (GHD, 2012; BNHCRC, 2015; Wahalathantri et al., 2015). Inspections of failed floodway structures conclude that this is due to floodways existing in complex surroundings that often do not conform to the hydraulic design assumptions and simplifications stated in the current floodway design guidelines (BNHCRC, 2015; Wahalathantri et al., 2015). Furthermore, the current Australian design guidelines do not consider the load combinations acting on floodways whilst in a submerged state. These forces can be associated with the ultimate limit state (ULS) design forces defined in AS5100.2 (Standards Australia, 2017*b*) for bridges acting under external loading influences such as drag, debris, log impact, scour and buoyancy.

To address this problem, this research seeks to build upon the limited work previously undertaken to deduce a comprehensive structural design method for floodway structures while acting under extreme flood loadings. The final stage of this research is the development of a floodway design guideline that integrates the structural design method with the current adopted hydraulic design processes, thus providing a practical end-user outcome.

1.1 Objectives

The overall objective of this research is to evaluate the principal structural failure mechanisms of floodways and develop a structural design approach that will improve structural resilience post extreme flood event.

In-line with the overall objective of enhancing the resilience of floodways, the specific research objectives are as follows:

1. Determine the loading case or cases which result in the maximum displacement and stresses for five standard engineering floodway types that are currently implemented in the Lockyer Valley Regional Council using non-linear finite element methods (FEM), hydrostatic loading and loading combinations derived from AS5100.2 (Standards Australia, 2017*b*).

2. Investigate design features using FEM to improve floodway resilience against extreme flood loading.

3. Recommend a conclusive structural design method, or methods, which satisfy the structural requirements of the five standard engineering floodway types that are currently implemented in the Lockyer Valley Region.

4. Develop a design guideline which will provide designers with a complete design approach inclusive of design charts and comprehensive worked examples.

1.2 Limitations

Potential limitations identified within this research are listed as follows:

1. Case study area - The Lockyer Valley Region was used for problem statement development and standard engineering floodway types. Although outcomes have been applied in a generalised sense, outcomes may still reflect this and not be representative of other global scenarios.

2. Survey based research - Research was based upon the responses achieved from the target audience within the survey and the way they interacted with the survey instrument utilised.

3. Experimental data - The experimental program was limited to the resources and funding available. Laboratory facilities were sourced through RMIT University, with strain analysers, load cells and soil box availability dictating the available test specimen size, assumptions and constraints in the experimental work.

4. Numerical Analysis - Numerical analysis outcomes derived are dependent on the agreement achieved with the industry survey responses and experimental results.

1.3 Thesis Outline

This thesis is in the format of *'Thesis by Publication'* and consists of seven (7) main chapters. To enable structure coherence, the below outline provides the thematical links between chapters.

Chapter 1 - Introduction: Provides background information and the motivation for the research. It also explicitly sets out the research objectives and identifies possible constraints and limitations.

Chapter 2 - Literature Review: Presents relevant findings from the review of existing literature in the context of the research. Topics addressed include what a floodway is, existing research, research gap identification, research idea justification, theoretical frameworks, research methods and an overview of the case study area.

Chapter 3 - Industry Survey & Experimental Investigations: Presents a survey containing industry perspective and experiences in the field of research, as well as an experimental program.

- **Paper I:** Floodways and Flood-Related Experiences: A Survey of Industry Experts and Asset Owners, *Engineering for Public Works.* `https://issuu.com/ipweaqld/docs/3660___epw_september_2020_final`. Published - September 2018.
- **Paper II:** Qualitative investigations into floodways under extreme flood loading", *Journal of Flood Risk Management.* Submitted - February 2022, Under Review.

Chapter 4 - Structural Design Methodology for Floodways: Presents an investigation into floodway vulnerabilities through numerical modeling of a single floodway type and provides a preliminary integrated design methodology.

- **Paper III:** Structural Design of Floodways Under Extreme Flood Loading, *International Journal of Disaster Resilience in the Built Environment*, Vol. 11 No. 4, pp. 535-555.

.https://doi.org/10.1108/IJDRBE-10-2019-0072. Published - 9 May 2020.

Chapter 5 - Applying Design Methodology to Several Floodway Types: Builds upon the research outcomes in Chapter 4, through the numerical simulation of four additional floodway types to deduce a conclusive numerical analysis methodology for floodways.

- **Paper IV:** A Numerical Approach to Improving the Resilience of Floodway Structures Under Extreme Flood Loading, *Sustainable and Resilient Infrastructure*. Submitted - September 2021, Under Review.

Chapter 6 - Floodway Design Guideline: - Provides a design guideline for floodways based on research outcomes, thus providing a practical end-user outcome.

- **Joint Publication BNHCRC & IPWEAQ:** Floodway Design Guideline, *Bushfire & Natural Hazard CRC and Institute of Public Works Engineering Australasia, Queensland Division (IPWEAQ)* - Submitted November 2020, Currently in Third Review Cycle.

Chapter 7 - Conclusion: Summarises the research outcomes in terms of the objectives set out in the introduction and provides future recommendations.

In addition, significant milestones relating to this research were presented at the *Queensland Disaster Management Research Forum 2021* and at the *Australasian Fire Authorities Council* (AFAC19). Refer Appendix A for further details.

CHAPTER 2

LITERATURE REVIEW

2.1 Floodways

Floodways are road infrastructure often used within ephemeral (short lasting) water courses, water courses with shallow continual flow and in large flow scenarios with the addition of culverts (Gautam and Bhattarai, 2018). The floodway structure is designed to permit vehicular crossing during low flow events through improvements to the stability and predictability of the trafficable surface. The application of floodways is suited to relatively consistent creek bed profiles and not deeply incised creek beds (Clarkin et al., 2006).

Floodways are a solution often implemented when a reduced or interrupted level of service during rainfall events can be accepted (Figure 2.1). This often includes lower order roads such as minor roads, formed tracks and access roads with a relatively low vehicle count per day (less than 50). Floodways, due to their simple anatomy provide capital savings compared to other road asset classes such as bridges and culverts. However, due to frequently being in a submerged state floodways often require greater maintenance.

Figure 2.1: Submerged floodway crossing precluding vehicular access.

Floodways (Figure 2.2) can also be of an unsealed, sealed or concrete construction depending on the level of service required. Many rural roads in outback and regional Australia are typically of an unsealed (gravel) or sealed construction, as opposed to concrete floodways which are frequently implemented in semi-rural areas. Due to the regular upkeep required of unsealed floodways, preference has been to replace these structures with concrete floodways to reduce the need for significant reoccurring operational expenditure for councils (The Department of Infrastructure, Transport, Regional Development and Communications, 2020). Floodways can also be situated level with the channel bed or raised. They can also be unvented or vented through the incorporation of culverts. Flow over the floodway structure is generally dispersed more widely than that of culvert applications, therefore, aiding in the reduction of flow concentrations and erosion downstream of the structure (Department of Transport Main Roads, 2010).

Figure 2.2: Typical floodway types (Lockyer Valley Regional Council, 2013).

2.1.1 Floodway Components

Floodways are generally constructed from reinforced concrete, with members subjected to constant wetting and drying from being in contact with water. Floodways are typically constructed from several major members, with variation depending on the specific location and if the floodway is raised or level with the stream bed (Figure 2.3 and Figure 2.4, respectively). These members and there functions are listed as follows:

- Upstream and downstream rock protection - provides bed protection against scour and erosion in the immediate upstream and downstream zones.
- Upstream and downstream cut-off wall - prevents undermining and

overturning of the structure, as well as the migration of groundwater through the underlying granular material.

- Upstream and downstream batter - accounts for the difference in upstream and downstream elevation between the road surface level and bed level. It also defines the weir type and hydraulic properties of the floodway. The batters are often armoured, or of a concrete construction to prevent scour. Furthermore, weep holes are often incorporated within concrete downstream batters to relieve hydrostatic pressure.

- Pavement (base and sub-base) - provides a stable, homogeneous and predictable foundation for the floodway structure.

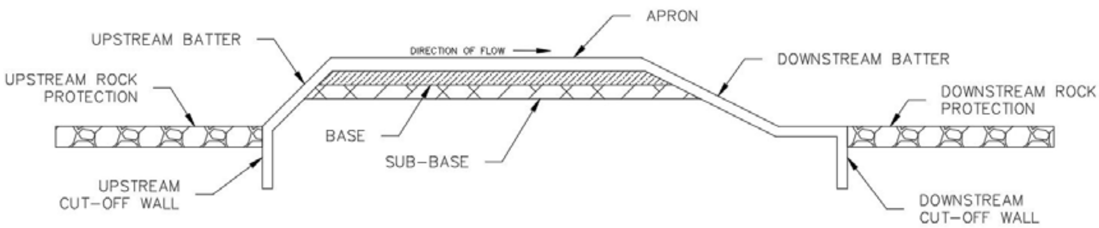

Figure 2.3: Schematic outlining major members of raised floodways.

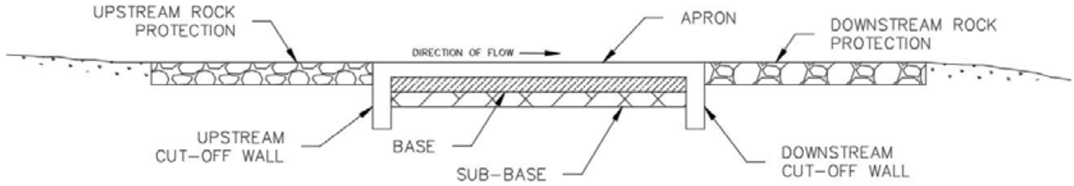

Figure 2.4: Schematic outlining major members of level floodways.

2.1.2 Failure Mechanisms

GHD (2012) undertook a case study into twenty-one (21) floodway sites within South Australia to categorise floodway failure mechanisms into the mode, and the area of failure. From this study three (3) primary modes of failure were derived as well as four (4) primary areas.

Failure modes:

1. Erosion - Erosion occurs when the velocity of flow is high enough to overcome the scour resistance of rock protection and the natural channel strata. Scouring, if not rectified, undermines the floodway structure causing failure (Figure 2.5) .

Figure 2.5: Undermined structure due to erosion.

2. Deposition (Figure 2.6) - Deposition is a result of localised watercourse widening or from reducing the original watercourse grade to make allowance for the floodway structure. This subsequently slows the velocity of flow and allows sediments and silts to settle on the structure. This mechanism rarely results in failure, however creates on-going maintenance issues.

3. Failure of the structure (Figure 2.7) - Floodways are exposed to significant loading as a result of being in a frequently submerged state, including; drag, debris, impact, hydrostatic pressure, uplift and vehicular loading. If these loadings exceed the design strength then failure will occur, sometimes catastrophically.

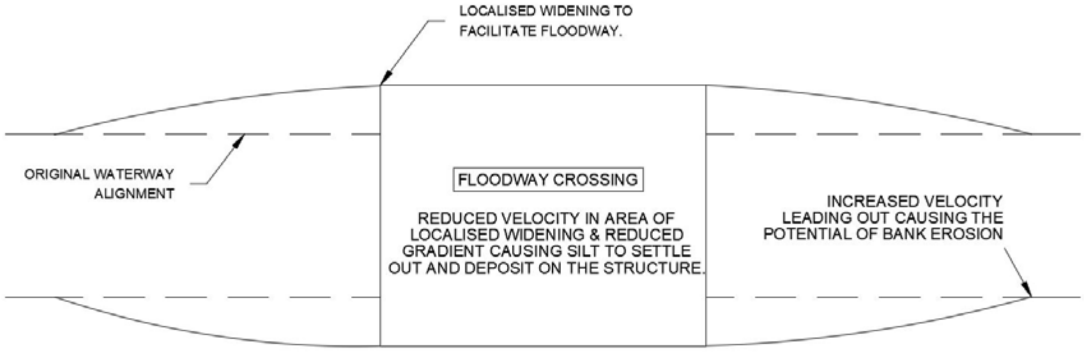

Figure 2.6: Sketch illustration deposition phenomenon.

Figure 2.7: Failure of the structure (Toowoomba Region, 2022).

Failure areas:

1. The upstream zone - The zone immediately upstream of the road shoulder, to a point where erosion will no longer compromise the floodway structure.

2. The roadway zone - The trafficable zone of the floodway including approaches and shoulders.

3. The downstream zone - The zone immediately downstream of the road shoulder, to a point where erosion will no longer compromise the floodway structure.

4. The peripheral zone - The zone immediately surrounding the floodway, yet

not apart of the floodway structure.

A study by Wahalathantri et al. (2015) also outlined the analysis of 64 failed floodways within the Lockyer Valley region after the 2013 Queensland flood event. This research determined that the high flow velocities associated with flooding were a significant contributing factor and documented common failure modes as washout, undermining, scour, damage to the concrete floodway structure, damage to rock protection and obstructions due to debris. The research by Wahalathantri et al. (2015) also documented the prevalence of local road authorities to undertake ‘patch’ repairs immediately after flood events to reopen roads expeditiously. These ‘patch’ repairs were discovered to compound damage if a subsequent flood event occurred in close succession to the first. An additional study by Wahalathantri et al. (2018) undertook a hydrological assessment to determine the relationship between flood discharge and damaged floodways in a case study area. It was discovered that floodways situated further downstream in the catchment, thus exposed to higher flows were more likely to fail. Further, all floodways but two, that experienced flows greater than 300 m^3/s completely failed. As a result of the two which remained Wahalathantri et al. (2018) concluded that other failure mechnisms must exist and shall be explored as further work.

2.2 Design Guidelines

Regional municipalities typically utilise generalised engineering floodway drawings to initiate design. They then adapt the design to the channel of concern, subject to local conditions and constraints derived from site-specific investigations. These standard drawings are often adopted from standard floodway structures included within floodway design guidelines.

Three different nationally recognised Australian design guidelines exist covering the best practises for floodway construction within Australia. These guidelines are the Austroads *'Guide to Road Design – Part 5B: Drainage – Open Channels, Culverts and Floodways'* (Austroads, 2013), Main Roads Western Australia *'Floodway Design Guide'* (Main Roads Western Australia, 2006), and Department of Transport and Main Roads *'Road Drainage Manual – Chapter 10 Floodway Design'* (Department of Transport Main Roads, 2010). In an international context the Iowa State University, Institute for Transportation has developed a design guideline called *'Low Water Stream Crossings: Design and Construction Recommendations'* (Lohnes et al., 2001), and the US Army Corps of Engineers Afghanistan Engineer District have developed a guideline called *'AED Design Requirements: Culverts and Causeways'* (US Army Corps of Engineers Afghanistan Engineer District, 2009). The typical format of these design guidelines include geometric, environmental, hydrology, hydraulic, time of submergence, closure and protection design considerations.

Austroads (2013); Department of Transport Main Roads (2010); Lohnes et al. (2001); US Army Corps of Engineers Afghanistan Engineer District (2009) specify standard floodway types based on wearing course and downstream protection type which include concrete, granular with bitumen wearing course and rock mattress or dumped riprap. Main Roads Western Australia (2006) alternatively specifies just three (3) floodway types based on low, medium and high flow velocities. Within all of the specifications, structural considerations for concrete floodways are generic and typically include 150 mm thick concrete with centrally placed reinforcement or a 300 mm thick granular pavement with a wearing course. Downstream protection ranges from concrete cut-off walls, 225 mm to 300 mm thick rock protection which includes dumped rock riprap or gabions. The Lockyer Valley Regional Council have implemented five (5) standard engineering floodway types in accordance with these design guidelines, however, with slight variations to cut-off walls based on observed failures (refer sub-section 2.3). BNHCRC (2015); GHD (2012); Wahalathantri et al. (2015) have all reported the presence of structural damage to floodways, which have either been built in accordance with the design guidelines or as modified by the Lockyer Valley Regional Council as a result of flood loadings.

Both Australian and international design guidelines are based predominately upon hydrological and hydraulic design principles and neglect the forces that floodways are exposed to, yet expected to withstand during extreme flood events. The hydraulic design principles specified within the design guidelines, both nationally and internationally, utilise the Empirical Broad-Crested Weir formula and Mannings formula exclusively for raised and flush floodway structures, respectively. These formulas contain a number of assumptions such as the water course is of an open prismatic and uniform channel. In addition, through the application of a mean velocity in Manning's equation, inaccuracies are introduced as often the velocity is lower at the deeper sections within a creek and subsequently higher at the edges and mid-section, which results in higher stresses (BNHCRC, 2015). The studies conducted by Wahalathantri et al. (2015) also concluded that floodways often exist in complex surroundings that include horizontal and vertical bends. Due to these complexities, assumptions used by the current hydraulic focused design guidelines often create outcomes which are less than optimal and investigation into improving design practices is warranted.

2.3 Previous Research

Minimal research is available in the field of floodways, in particular covering structural analysis. As a means of increasing design accuracy and creating more resilient floodway structures Setunge et al. (2015) discusses analysing floodway superstructures with the loadings defined for bridges in AS5100.2, *'Bridge Design, Design Loads'* (Standards Australia, 2017*b*). This includes flood loadings such as drag, debris, log impact and buoyancy and can be related to floodways due to the similarity in exposure when both structures are acting in a submerged state. Applying this concept, there has been very few research attempts into its implementation, that is, analysing the structural adequacy of floodways designed as per the current Australian design guidelines using the limit state loadings defined in AS5100.2 (Standards Australia, 2017*b*) and hydrostatic fluid force theory.

Significant research and investigation have been undertaken into the principal failure mechanisms of floodways during peak flow events in recent years (BNHCRC, 2015; Wahalathantri et al., 2015; Furniss et al., 2002; GHD, 2012). The research conducted by BNHCRC (2015); Wahalathantri et al. (2015); GHD (2012) specifically investigated the factors contributing to the many floodway failures reported after the significant and widespread flooding within Australia. These studies have recorded consistent failure mechanisms and have attributed the primary cause of floodway failure to increased sediment loads, organic debris (logs) and boulders impacting the floodway causing the superstructure to experience significant loading while in a submerged state. Erosion was also present at many of the damaged floodway sites indicated by scouring in the immediate upstream and downstream rock protection zones. Furniss et al. (2002) explains that this type of failure is often complex and is a factor of the location and frequency of landslides, watercourse bank erosion, treefall and other processes occurring upstream of the concrete floodway structure. In some severe instances undermining of the floodway superstructure was also present. GHD (2012) reported that the presence of downstream erosion at a floodway site often had the potential to form an erosion head cut which eventually, if left unrepaired resulted in catastrophic failure of the superstructure.

This existing research provides an excellent insight into the failures present within floodway structures post extreme flood event, but does not provide correlation, nor answers as to the cause of the failure and if the current design practices are adequate to withstand the effects of flooding.

Due to the limited research in the field of structural analysis of floodways, previous research relating to bridges under impact and flood related loadings was also investigated. Guo et al. (2022) explains that scour in the case of bridges reduces horizontal and vertical capacity of bridge foundations, leaving them susceptible to horizontal loads such as collision. Previous literature has documented hydraulic and hydrologic analysis in terms of both experimental and numerical approaches for bridges (Greco et al., 2021). Similarly, current design codes, both Australian and International (AS5100 and AASHTO, respectively) utilise equivalent static methods for the estimation of impact force and related strength-based bridge design.

In terms of impact related research, Gholipour et al. (2021) discovered that the geometry of the structure being impacted had a significant influence on the impact force. Gholipour et al. (2021) determined that in a bridge application, flat-faced piers resulted in greater impact forces than round piers. This is also supported within the equivalent static methods utilised in AS5100.2 and within the manuscript by Hung and Yau (2014), which assigns a factor based on pile geometry, with the factor been greater for square end pier nosing (1.4) then round end pier nosing (0.7). The research by Gholipour et al. (2021) documented the typical failure modes of reinforced concrete bridge piers under different impact loads based on observed historical failure events. These failure modes due to high collision impact are all related to shear failure and include punching shear failure, shear hinge and a diagonal shear failure which originates from the point of impact (Figure 2.8). Furthermore, under these loading conditions the columns would suffer from brittle failures, increasing the probability of complete structure failure.

From a numerical evaluation perspective, the nonlinearity assigned to the concrete material and material of the object impacting the column significantly affected the impact response (Yunleia et al., 2018). Further, Yunleia et al. (2018) states that accurate prediction of the response of bridge columns to impact loading is paramount and is required to properly design the structure to resist collapse due to impact load. Due to large-scale experimental work being both expensive and time consuming, numerical simulation using finite element methods is extensively used in the study of vessel impact on bridge columns.

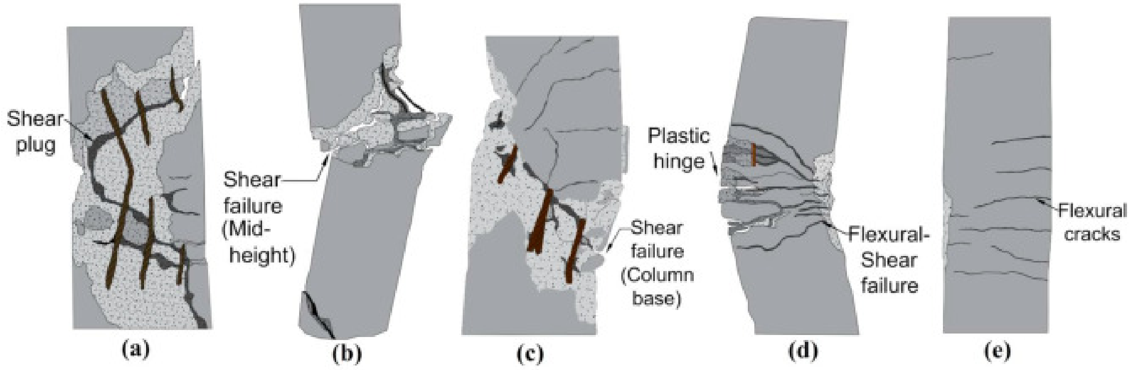

Figure 2.8: Typical failure modes of reinforced concrete bridge piers due to vessel impact (Gholipour et al., 2021).

The above research provides insight into the failure modes of reinforce concrete structures due to impact. It is suggested by the writer, and in the absence of specific guidelines for floodways, that a correlation can be made between bridges and floodways, due to a similarity in operational environment and based on shared flood related loadings, including impact loading. Utilising methods and analysis techniques deduced through literature relating to bridges, an understanding and prediction of the response of floodways to impact related loadings can be had. Further, the validity of the use of numerical modelling through finite element methods for bridge related loading analysis is not only economical but is considered an accurate approach within literature.

2.4 Previous Numerical Investigations

Cummings (2015) conducted a 2-dimensional linear elastic analysis into a single raised floodway type. This research was a case study and based upon a specific floodway application in the Mt Sylvia region. Cummings (2015) undertook this research to highlight areas of floodway vulnerability within this setting using three different loading combinations. From this research, it was concluded that impact loading resulted in the maximum displacement and stress results. All cases analysed were within acceptable limits except for one. This case utilised a flow velocity of 10 m/s and was concluded by the author as not realistic of flow velocities experienced in the Mt Sylvia area. By adopting a 2-dimensional linear elastic model, several assumptions were inevitably applied, such as plane strain conditions and materials that abide by linear elastic material criterion.

Greene (2018) expanded on Cummings (2015) research and utilised non-linear 3-dimensional finite element methods to analyse a single engineering floodway type in a universal setting. The objective of this research was to quantify the non-linear response of a standard engineering floodway type under extreme flood loadings. Based on these findings Greene (2018) proposed a simplified structural design method which utilised design charts with the absolute maximum bending moment and shear force, however this was only applied to one floodway type. Further feedback obtained from the LVRC has also identified several limitations within Greene (2018) research which included; the model depth being too shallow causing the fixed restraints to over stiffen the model. The cut-off wall in the model was omitted from the two sides opposing traffic (the cut-off wall should be applied to the entire floodway perimeter), soil materials were assumed drained with the fluid level set to zero, hydrostatic uplift force was duplicated and in-situ stresses were not assigned in the model causing results to not converge as model depth was increased.

Both of these studies adopted simplified model validation techniques reliant on linear analytical solutions such as Hooke's Law and the Static Friction formula, as well as visual observations of deformation behaviour and magnitude. This method is often limited to simple numerical models and is used to validate individual linear theorems. Floodways are of a complex geometry and are exposed to non-linear material behaviours and properties, limiting the degree of confidence that exists within these earlier studies.

Furthermore, no current experimental data exists to validate the non-linear behaviour of material properties used in floodway applications. These non-linear behaviours include soil, which is governed by cohesion and internal friction angle

(Jiang, 2018) and the failure of concrete when stress exceeds yield strength in either a tension or compression scenario (Feng et al., 2019). If agreement between experimental and numerical solutions can be achieved, this will provide a preliminary data set for the validation of future studies utilising numerical modelling, thus providing greater confidence limits.

2.5 Improvements Implemented by Asset Owners

Following the widespread damage sustained by floodways in Queensland, several asset owners, including the Lockyer Valley Regional Council have experimented with amendments to geometric features specified within their standard engineering floodway designs. These amendments are based on practical experiences and were verbally mentioned to the authors at the commencement of research. These amendments are summarised as follows (Greene, 2018);

- encompassing a cut-off wall to the entire perimeter of the floodway - Design and construction practices prior to this in the Lockyer Valley Regional Council only included cut-off walls to the upstream and downstream extents only. By including the cut-off wall to the entire perimeter they have managed to prevent the migration of ground water flowing through the underlying granular road pavement, thus providing further protection against the structure being undermined due to afflux at the perimeter ends.
- Varying cut-off wall depths - The Lockyer Valley Regional Council standard drawings currently specify either a 900 or 1100 mm cut-off wall as shown in Figure 2.9. The selection of cut-off wall depth is currently a function of the proposed sites average stream velocity. The increased cut-off wall depth is anticipated to provide increased scour protection and also increase the lateral

resistance against movement of the structure due to the greater surface area present.

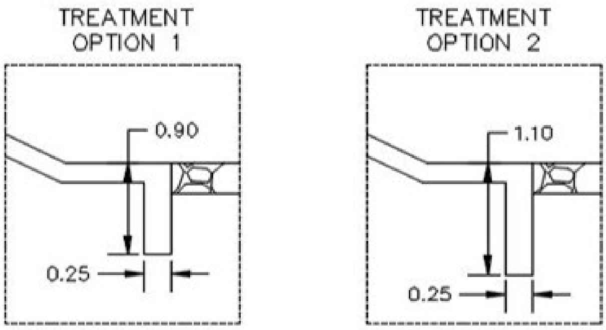

Figure 2.9: Schematics of the treatment options (in m) applied to floodways in the LVRC (Greene, 2018).

- There is also an emerging trend of asset owners vertically realigning damaged floodway structures level with the creek bed after observing significantly less exposure to damage during extreme flood events for this construction type.

Although these improvements have been trialled in practice there is no tangible data to quantify the success of these trials or if the designs adopted are optimal. Through further investigation and by adopting a parametric analysis procedure, the benefit and optimisation of these improvements can be derived.

2.6 Analysis of Grant Applications

Analysis of historical floodway infrastructure maintenance and capital expenditure that exists within literature enables the issues and considerations identified in current floodway design to be identified, along with the methods being adopted to address these issues. This ensures that any new research outcomes take into account the lessons of past infrastructure applications and have consideration of current industry practice.

The Australian Government offers various grant programs to Local and State government organisations to enhance road amenity and safety (The Department of Infrastructure, Transport, Regional Development and Communications, 2020). These programs include Roads to Recovery Program and Black Spot Projects. This section provides an analysis and discussion of floodway related projects between 2013 and 2020. The information and direct quotations presented within this sub-section have been sourced directly from the project summaries on The Department of Infrastructure, Transport, Regional Development and Communications (2020) website and further information can be found through conducting a search based on the referenced project ID.

During the period of analysis, a total of 193 floodway projects where awarded funding for activities including general maintenance, flood damage rectification, improved safety and increased user amenity (betterment projects). These projects were implemented in most states and territories within Australia except for the Australian Capital Territory (ACT) and Tasmania (TAS). An analysis of these projects (sample, n = 193) aims to provide a better understanding of specific industry problems faced with existing floodway design, what improvements are being made and the requirement for capital infrastructure spend for this asset class.

2.6.1 Original Floodway Construction Type

Of the 193 funded projects, and prior to any project works, 96 floodways were unsealed, 38 were sealed and 59 were constructed from concrete (Figure 2.10). Unsealed floodways were the most common construction type as a result of floodways typically having a rural locality and associated low traffic volumes. It must be noted that even though unsealed floodways were the most common original construction type, many of these projects were related to the removal of existing gravel floodways and replacement with new concrete floodways. This was the result of the increased maintenance and wash-out probability associated with unsealed floodway constructions.

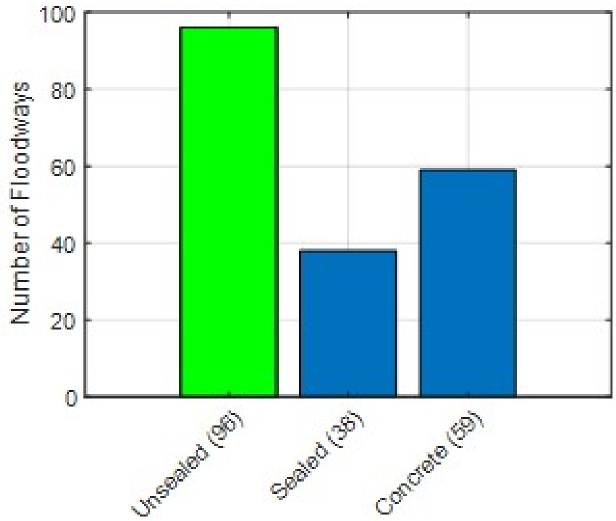

Figure 2.10: Original floodway construction.

2.6.2 Project Type

Assessing project type (Figure 2.11) it can be noted that flood damage accounts for the highest number of approved projects (58). This suggests that existing floodway structures often prematurely fail due to exposure to flood. Projects relating to increased amenity, general maintenance and improved safety rank respectively, thereafter.

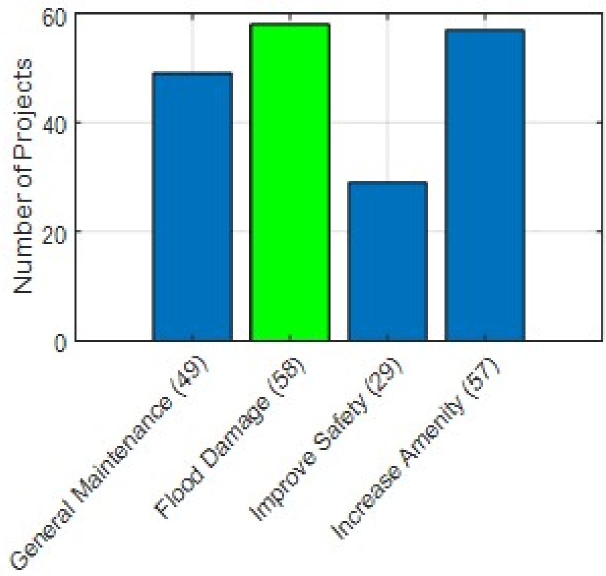

Figure 2.11: Government funded floodway project types.

2.6.3 Flood Damage

Common failure mechanisms attributed to flood damage can be classified as scour and wash-out, undermining and debris build-up (Figure 2.12). Out of the common failure mechanisms scour and wash-out accounted for 44 (76%) of flood related failures. In many instances this can be attributed to the significant use of unsealed floodway constructions within rural roads, which were noted as being regularly and significantly compromised even after the smallest rainfall events. Many of the betterment projects listed were to replace unsealed floodways with concrete floodway alternatives, which would significantly reduce operational expenditure and maintenance requirements. Undermining and debris build-up from flood waters were associated with sealed and concrete floodway construction.

Comments relating to project applications on flood damage have been directly quoted as follows:

"The downstream of the concrete structure has been undermined due to flood and has caused damage on the foundation. It is proposed to remove the existing floodway

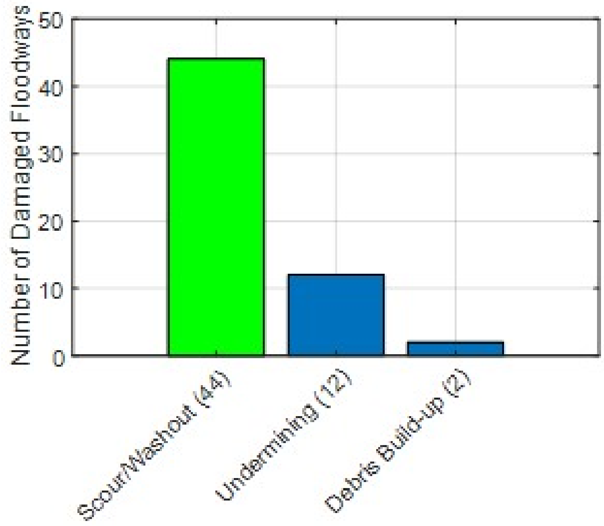

Figure 2.12: Flood damage projects break-down.

and construct a new floodway in current engineering standards." Project ID: 067642-16VIC-RTR

"Damaged (undermined) and dangerous concrete floodway. Replacement of floodway with concrete floodway with appropriate cut-off footing." Project ID: 097526-18QLD-RTR

"The batter on the downstream side of the floodway crossing Chippendale Creek has failed and water has eroded under the batter. It is proposed to backfill under the slab and batter, remove the old concrete and build a replacement concrete batter." Project ID: 091600-16QLD-RTR

"Gravel road approaches to concrete floodway wash away during floods creating a dangerous situation when concrete floodway is still under water." Project ID: 052851-14QLD-RTR

"Gravel floodway washes out after all but minor rain events resulting in a hazardous surface. Construct a new reinforced concrete floodway." Project ID: 104966-19QLD-RTR

"Existing floodway unsealed and regularly is washed away causing immediate dangerous situation and then road closure. Construct a concrete floodway to proper engineering standards." Project ID: 097527-18QLD-RTR

2.6.4 Improved Road Safety

Twenty-seven (93%) improved road safety projects related to rectifying horizontal and vertical floodway alignments to improve sight distance and the predictability of the floodway road surface for motorists (Figure 2.13). Another two (7%) projects where related to improving road safety through the addition of signage assisting with increasing motorist awareness, judgment and decision making. These types of projects suggest that there are a considerable number of floodways in service within Australia that do not meet the current best practices relating to both geometrical alignment and signage.

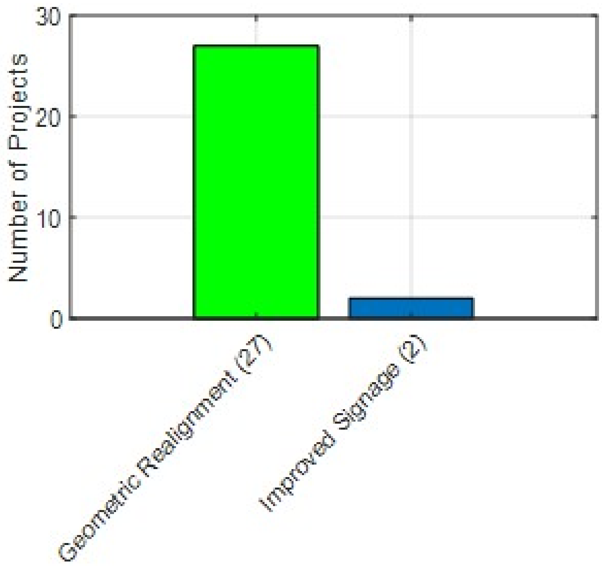

Figure 2.13: Improved safety projects break-down.

Comments relating to project applications to improved road safety have been directly quoted as follows:

"Low level floodway with poor horizontal and vertical alignment over regular flowing

creek." Project ID: 051038-13QLD-RTR

"Improve vertical grade on floodway approach, install shoulder to >1 m sealed, construct pavement shoulders, seal bitumen on curves & approaches, culvert widening, install warning signs, reflector guideposts, centre & edge lines & guardrail." Project ID: 100093-19QLD-BS

"This section of the Road includes a large floodway with tight curve, a crest and poor line of site for all vehicles, lack of drainage due to culverts broken and others are blocked. The Floodway will be upgraded, and drainage to the sides of the road addressed. Delineation and signage graded and replaced where required." Project ID: 102143-19WA-RTR

2.6.5 Increased Amenity (Betterment Projects)

Forty-four (77%) betterment projects related to upgrading existing floodways (Figure 2.14). This includes installation of concrete floodways in-lieu of unsealed floodways, floodway widenings and floodway extensions. A minority of the projects (five) within this classification were to upgrade existing floodways to alternative asset classes such as bridges or culverts (< 9%). Several projects did, however, seek to implement floodways as a solution for dilapidated bridges (12% or seven projects). This is considered implementing a solution with a reduced level of service, however, is ideal in scenarios where decreased amenity requirements exist (lower order roads) or if the limited funding available is not enough to maintain the higher asset class ongoing.

Comments relating to project applications to increase amenity/betterment have been directly quoted as follows:

"Widen floodway and approaches to increase the seal width to 7.0 m, improve signage, line marking and delineation." Project ID: 100901-19WA-BS

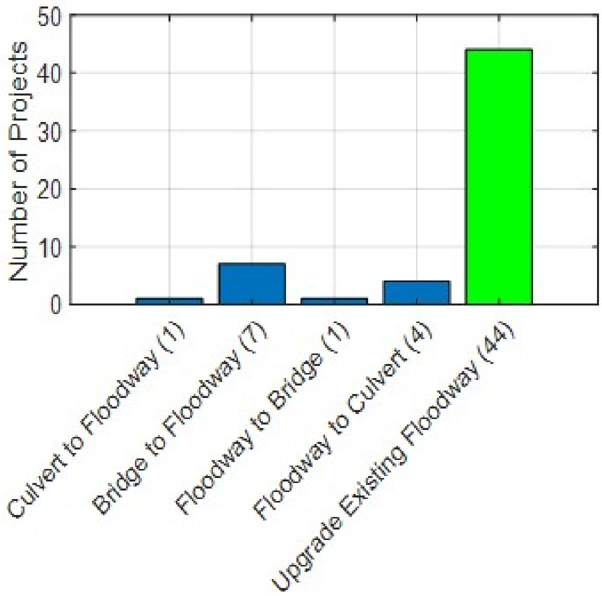

Figure 2.14: Improved amenity projects break-down.

"The existing gravel floodway suffer continuous damage during the wet season and becomes boggy and slippery. This causes safety problems and hazards for motorists driving on a gravel floodway in wet weather. Road closures are also more frequent due to unsafe condition of the road. Construct a 30 metres long concrete invert at Chainage 19.485 kilometres." Project ID: 101529-19NT-RTR

"Existing timber bridge has significantly deteriorated resulting in significant structural deficiency. Replacement of bridge with 2 cells of 1500 mm reinforced concrete pipe. Concrete floodway to be constructed over pipe." Project ID: 050162-13QLD-RTR

"Safety Issues with increased traffic use. Minor rain events cause Road to be closed for short periods of time. Replace floodway with improved drainage by installing reinforced concrete box culverts and raising the road level." Project ID: 055778-15QLD-RTR

2.6.6 General Maintenance

Thirty-two (65%) of maintenance projects were undertaken to replace end-of-life floodways in a like for like manner and/or with minor improvements to site drainage. The remaining 17 (35%) maintenance projects were to conduct gravel re-sheets and/or floodway surface re-seals (Figure 2.15).

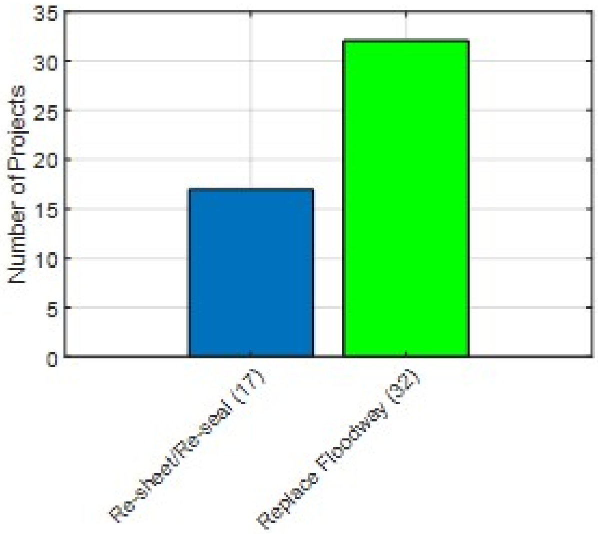

Figure 2.15: General maintenance projects break-down.

Comments relating to project applications for general floodway maintenance have been directly quoted as follows:

"Failing floodway due to poor historic design. Cement stabilising of floodway and upgraded headwalls to design levels." Project ID: 100069-18WA-RTR

"The floodway has reached its design life and is beginning to show signs of failure. Substantial structural issues have been identified which poses a risk to the floodway being washed away in a storm event, resulting in the isolation of communities. Reconstruct the concrete causeway and approaches." Project ID: 062091-15QLD-RTR

"Drainage Structure failure/metal helicor pipes collapsing and concrete deck and

batter deterioration/cracking. Remove Current floodway and replace with new concrete pipes, deck and batters to standard." Project ID: 055112-14QLD-RTR

"Floodway at the end of its service life and requires replacing. Install three cells of new culverts 600 mm x *1200 mm with a new 150 mm slab over the top of the culverts with bitumen approaches."* Project ID: 106081-19QLD-RTR

"Floodway subgrade has failed, pavement and seal has broken up and is now a safety hazard to the road users. Existing 200 mm of soaked pavement and subgrade will be removed. Road base will imported for subgrade and cement stabilized. New pavement will be imported and cement stabilized, final trimmed and sealed." Project ID: 106365-19WA-RTR

2.7 Research Frameworks and Methods

Successful research is dependent on applying the appropriate theoretical frameworks and having the appropriate controls placed on the research methods. Outlined within this section are the theoretical frameworks and research methods which are proposed to be applied within the research.

2.7.1 Numerical Modelling

Floodways, like bridge structures are physically very large, limiting the ability for full-scale experimental studies (Timilsina et al., 2021). Li et al. (2020) explains that non-linear finite element models are often used for virtual testing as an effective supplement to experimental investigation for very large structures due to the relatively long duration, cost and complexity of establishing an experimental program. Numerical analysis therefore provides an advantageous platform to analyse these structures under structural loading, with the approach being adaptable to suit a wide range of applications. Numerical analysis can also often provide a more detailed understanding of the failure, particularly that within the test structure, which may not be visible (Mohammed et al., 2020). Cui et al. (2021) states that in the application of bridges, the numerical model is developed based on engineering drawings, with a number of assumptions and approximations applied to establish boundary and material behaviours. Often, these assumptions limit the accuracy of results within numerical analysis and therefore validation of the model's behaviour and accuracy needs to be undertaken. Validation is typically undertaken by comparing finite element results with experiment results and agreement assessed accordingly. This is usually undertaken through the development of an appropriate experimental program of scaled models, or augmented components (Zare et al., 2021). To solve the defined problem, numerical analysis subdivides a large system into smaller parts referred to as finite elements and solves an associated set of equations in an iterative manner based on the assigned initial conditions (Arora et al., 2021).

Surveys and questionnaires can also provide a qualitative means to validate numerical analysis outputs. Questionnaires capture the observations, experiences, and opinions of a defined sample group. This sample group often consists of subject matter experts that can relate their in-depth understanding and experiences, and provide an individual perspective on topics, often identifying alternative behaviours, limitations, and opportunities. This information captured can often lead to several hypotheses that can be tested against numerical solutions to determine whether qualitative agreement exists. If agreement exists, then it can be concluded that the finite element methods being applied qualitatively describe the loading-failure interaction of the structure.

Figure 2.16 provides an outline of the modelling techniques summarised from literature to be applied within the numerical modelling of floodways, including validation inputs.

The different theoretical frameworks that are required to support the development of a numerical model and to enable analysis through a structural adequacy

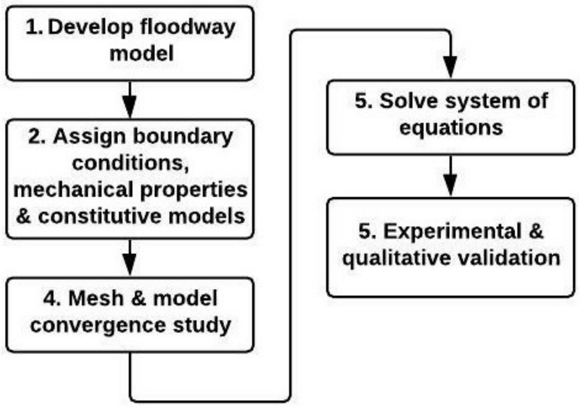

Figure 2.16: Numerical simulation techniques to be applied in the numerical simulation of floodways.

approach can be explained as boundary conditions, constitutive models, mechanical properties and load assignment.

Boundary conditions

Boundary conditions are assigned to nodes to provide restraint and load conditions. In the instance of floodways, boundary conditions need to be assigned accurately to represent the in-situ support conditions from the adjoining soil material. Arora et al. (2021) explains that boundary conditions need to be carefully assigned to not over or under restrain the model. A mesh and model convergence analysis is required to provide confidence that boundary conditions have been assigned appropriately.

Constitutive models

Constitutive models are assigned within numerical analysis to provide a theoretical prediction of the onset and evolution of failure mechanisms within a material (Maimi et al., 2007). Floodways are concrete structures that are constructed in-situ and therefore have two primary material behaviours associated with concrete and soil materials. These materials behave non-linearly and therefore require non-linear continuum models assigned to define the behaviour.

Mohr-Coulomb Yield Criterion is documented as a commonly assigned continuum model for soil materials and describes soil as being governed by cohesion and internal friction angle (Jiang, 2018).

Max Stress Yield Criterion is documented as a continuum model that defines failure within concrete when stress components exceed the assigned yield strength (as defined via a stress versus strain curve (Figure 2.17)) in either tension or compression (Feng et al., 2019; Ferdous et al., 2018).

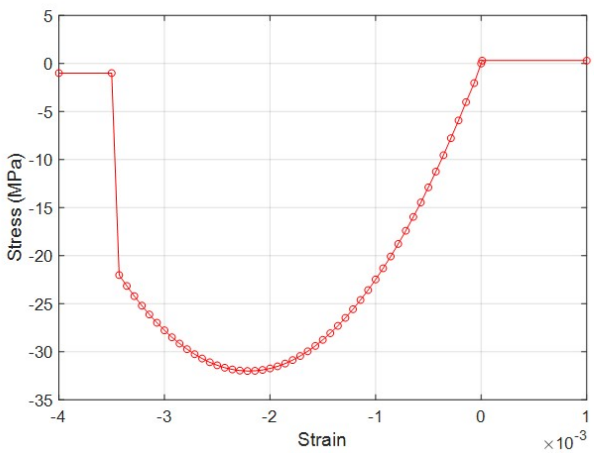

Figure 2.17: Typical 32 MPa concrete stress-strain curve.

Furthermore, many studies assume a perfect bond at the interface of different materials if no bond failure is observed in practice, or the limiting load being applied is well below the friction force limit (Ferdous et al., 2018).

Mechanical properties

Mechanical properties are assigned to elements within numerical analysis to define the individual strength properties of elements.

Load assignment

Since current floodway design processes are governed by hydraulic design aspects and do not consider the various design loads and loading combinations floodways are subjected to as part of their serviceable life, applicable load assignment assumptions need to be developed for floodways. Research undertaken by Bushfire & Natural Hazard CRC (2019) details that the force characteristics and design loading combinations can be related to bridges. These loadings include drag, debris, impact, lift and vehicular. Bushfire & Natural Hazard CRC (2019) concludes with reasonable assumption that in the absence of specific details, the loadings specified in AS 5100.2, "Bridge Design, Design Loads" (Standards Australia, 2017*b*) can be considered as a worst case loading scenario for floodways with several assumptions applied to derive coefficients. Preliminary research work conducted by Bushfire & Natural Hazard CRC (2019) and Cummings (2015) states that drag and lift forces are negligible for floodway structures without culverts, and that impact loading is the largest accidental loading considered in AS 5100.2, which aligns to the observations of damaged floodways as a result of boulders within the Lockyer Valley Regional Council.

The movement of boulders occurs once drag force exceeds the frictional force exerted by the channel bed. During the arrival of floodwaters, large fluctuating accelerations in flow occur. These accelerations exert a very large impulsive force on stationary objects, such as boulders because of loss of fluid momentum. It is these velocity fluctuations in conjunction with drag force from the floodwater that contribute to the movement of boulders during flooding (Alexandar and Cooker, 2016). Furthermore, this is exacerbated by the considerably high bulk density of sediment laden floodwaters that increase buoyancy, decreasing the force required to move an object. Boulders move slower than the velocity of the water column due to their intermediate contact with the channel, shear mass and their spherical geometry (Fondriest Environmental, 2014). As the head of the floodwaters advances past the boulder a lower net contribution from impulsive force results as flow velocity is generally more consistent (Alexandar and Cooker, 2016).

AS 5100.2, "Bridge Design, Design Loads" (Standards Australia, 2017*b*) provides a design force calculation for a floating log impacting a bridge superstructure. This formula is an equation of work with force equal to the kinetic energy of the object impacting the structure. This equation assumes that an object such as a log is buoyant and therefore moving at the same velocity as flow. That is the velocity of flow is equal to the velocity of the object and no net acceleration is present. The movement of boulders due to floodwaters is a much more complex phenomenon as they are submerged and roll, slide and saltate along the waterway channel in both steady and unsteady state flow conditions. Therefore, the application of an appropriate factor to the impact loading formula in AS 5100.2 (Standards Australia, 2017*b*) is required to reflect this difference in behaviour.

2.7.2 Survey Instrument

Survey instruments are widely used to gather large data sets on a subject within an expeditious time frame. Oosterveld et al. (2019) summarises surveys as structured instruments that utilise statements, questions and stimulus words that require a judgment or response from the target survey audience. Elangovan and Sundaravel (2021) explains that the quality of the survey instrument adopted has a significant effect on outcomes, and if incorrectly structured poor data quality may result due to incorrectly phrased questions and poor development processes. Gillham (2007) explains that the rate of response can be used to gauge the success of a survey instrument. It is suggested that a response rate of over 50% can be considered a good response and suggests that the respondents were engaged with the survey instrument. Elangovan and Sundaravel (2021) states that despite the various frameworks available to develop a quality survey instrument content validation for appropriateness is an essential aspect for an survey instrument.

The use of a survey instrument will be utilised in this research to expeditiously gain the practical experiences in relation to floodways from subject matter experts. The questionnaire will form a structured approach, and target a return completion rate of greater than 50% as per the metric defined by Gillham (2007). Furthermore, the survey will undertake a number of review iterations to ensure that the survey content is validated for appropriateness.

2.7.3 Experimental Program

As a result of simplifications and assumptions applied during the development of finite element models, validation against experimental observations is important to provide confidence in the numerical results obtained (Bola et al., 2021). Due to the absence of documented experiments concerning floodways in literature, an experimental program is essential to support finite element modelling approaches. Validation can be undertaken based on displacements, strains and correlation of deformed shapes (Bola et al., 2021). The novelty of undertaking an experimental program for a floodway will provide initial documented literature, which enables other authors to validate future models (Colaço et al., 2021).

2.8 Case Study Area

The Lockyer Valley Region, the case study area chosen for this research is located approximately 60 km West of Brisbane at the base of the escarpments of the Great Dividing Range (Figure 2.18). The Lockyer Valley Region has a combined population of approximately 42,267 (Australian Bureau of Statistics, 2021). The Lockyer Creek, the main watercourse which traverses the Lockyer Valley Region is fed by a significant number of upper catchments before meandering its way East across open agricultural rich farmland until it meets the Brisbane River just South of Wivenhoe Dam (Queensland Floods Commission of Inquiry, 2011). The Lockyer Creek is the largest tributary system of the Brisbane River.

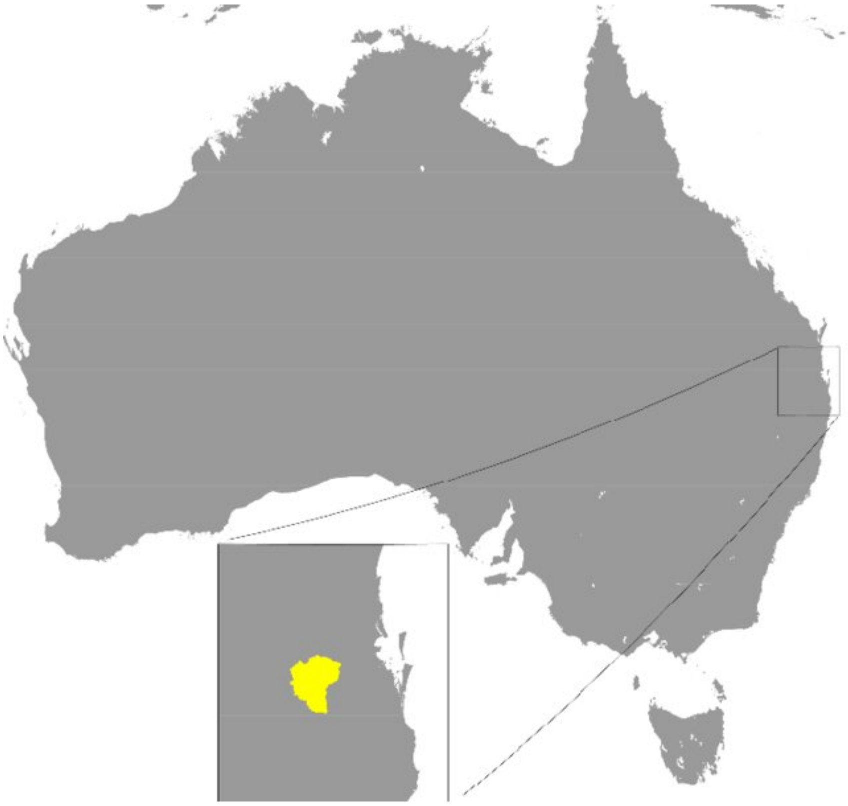

Figure 2.18: Location of Lockyer Valley Catchment in Queensland, Australia (Roberts et al., 2019).

The Lockyer Valley Region has an established history of flooding with the trend becoming more prevalent in recent decades (Queensland Floods Commission of Inquiry, 2011). A recent event which had devastating consequences on the built environment causing loss of life was the Lockyer Valley floods of January 2011. Fitzgerald et al. (2019) explains that the La Nina pattern occurring between 2010 to 2011 was the strongest observed since 1974 and brought widespread rainfall to regions within Queensland. As a result of this event, the Lockyer Valley Region experienced a rare and damaging flood event. This flood event was described by McPherson (2011) as an inland tsunami and resulted in 59 flooded rivers, broke 12 flood records and devastated key road infrastructure. The infrastructure restoration expense post natural disaster was estimated at 5 billion dollars (Sultana et al., 2016). The floods also impacted the Australian economy more broadly with the IBISWorld downgrading the countries GDP by 0.3% (Wisetjindawat et al., 2017).

Other extreme weather events similar to the floods of 2010-2011 where the floods of 2013 and again more recently with the extreme weather event occurring in January 2019 as a result of a quasi-stationary monsoon depression within North-East Australia Cowan et al. (2019). These flood events, similar to the floods of 2010-2011 caused significant infrastructure damage, isolated communities and inundated thousands of homes.

During the recovery effort after these flood events it was observed that the most prevailing damage was to small road structures such as culverts and floodways. Investigation suggested this was as a result of the loading on these structures being proportionate to the magnitude of the floodwaters, which increases exponentially (Setunge et al., 2015). During the floods of 2010-2011, the total number of damaged road infrastructure within the Lockyer Valley Region alone was surveyed as 48 bridges, 256 culverts and 192 floodways, requiring $180 million of Natural Disaster Relief and Recovery Arrangements (McPherson, 2011). As a result of this damage, delays in the safe reopening of roads were experienced, prolonging the adverse effects of the flood and preventing humanitarian efforts from commencing expeditiously. This illustrates the need for resilient road networks to maintain safety and expedite the recovery effort for flood prone communities (Wisetjindawat et al., 2017; Pregnolato et al., 2017).

2.9 Research Significance

Flooding presents a reoccurring threat to the built environment and human life with studies reporting an increasing change in magnitude, frequency and geomorphic state over the past century (Thompson and Croke, 2013; Sultana et al., 2016). The extremities of these events, as caused by heavy precipitation, tropical cyclones and storms are expected to cause the most challenging conditions for infrastructure resilience, putting immense pressure on government and economic sectors (Beniston et al., 2011; Mullett et al., 2015). Climate change is at the forefront of many debates regarding the current observed increase in extreme rainfall events and temperature, for which the reason remains unsolved (Bromhead, 2021). Agreement does however exist that further increases in extreme weather related events at a global level is expected to continue into the foreseeable future (He et al., 2021; Zhang et al., 2021).

In the transportation sector, flooding is the largest cause of weather related disruption (He et al., 2021). Road structures located within waterways, such as floodways are assets that are particularly vulnerable and frequently sustain damage or catastrophically fail as a direct result of increased flood waters (BNHCRC, 2015). This causes significant disruption to communities either as a result of flood depth precluding traffic passage or velocities and associated loading causing road infrastructure to be destroyed (Mullett et al., 2015). As a result of this disruption economic loss is observed along with significant financial impact to isolated communities (Du et al., 2019). These periods of disruption also delay early recovery efforts, exacerbating disaster effects post extreme flood events.

2.9.1 Bushfire and Natural Hazard CRC (BNHCRC)

Following several natural disasters the Federal Government of Australia committed funding towards creating more resilient communities. This commitment has been matched in many instances by State and Territories and has lead to a concentration of research efforts into natural disaster resilience. The Bushfire and Natural Hazard Co-operative Research Center (BNHCRC) was formed as a result of this funding for which improvement of small road structures, including floodways has been a focus.

2.10 Research Gaps

The investigation and development of methods and frameworks to improve the built environment and its resilience against extreme flood events is a significant focus area of current research. It is also an essential aspect in managing economic viability within the built environment (He et al., 2021). To achieve more flood-resilient floodway infrastructure, time of exposure to extreme weather events and minimising vulnerabilities through incorporating proper structural mitigation measures is essential (Chowdhooree and Islam, 2018). As a result of the frequent reoccurring damage to floodways, an investigation into the current floodway design process is warranted to improve design methods and framework.

Both Australian and international design guidelines are based predominately on hydrological and hydraulic design principles, and neglect the significant forces present during extreme flood events, such as hydrostatic, debris, vehicular, drag, impact and lift which act against the floodway structure. Discussions with staff at the Lockyer Valley Regional Council stated that impact from boulders was the primary failure mechanism within their catchment (Furniss et al., 2002). This loading case also had qualitative agreement with various sources of literature, which summarised the critical conditions for the movement of boulders based on field observations (Rijn, 2021; Inbar and Schick, 1979; Turowski et al., 2009; Fahnestock, 1963). However, the lack of empeircal evidence within literature on the effect of these forces on floodway structures restricts the formulation of design methods and framework to reduce reoccurring damaged post extreme flood events.

Similarly, several industry members have modified several aspects of floodway design, based on intuition to increase floodway resilience (as detailed in Section 2.5), as well as recorded structural failure mechanisms of floodways (Wahalathantri et al., 2015). These factors are interlinked, however depart from current design guidelines which focus on hydraulic aspects and failure resulting from erosion, scour and raised tail water levels. Furthermore, academic studies conducted to date don't specifically focus on these factors.

These research gaps highlight several short comings in the current knowledge pertaining to floodways, and suggest that once suitable investigation has been undertaken revision to current design guidelines and framework is required to incorporate structural design methods that will improve structural resilience and assist in creating more resilient communities.

CHAPTER 3

INDUSTRY SURVEY & EXPERIMENTAL INVESTIGATIONS

3.1 Industry Survey

Publication I: Floodways and Flood-Related Experiences: A Survey of Industry Experts and Asset Owners

Greene I, Lokuge W & Karunasena W 2020, "Floodways and Flood-Related Experiences: A Survey of Industry Experts and Asset Owners", *Engineering for Public Works*, Vol. 19, pp. 69-75. ISSN 2652-6050. https://issuu.com/ipweaqld/docs/3660___epw_september_2020_final. Published - September 2020.

This chapter contains an extract of the above published manuscript. Please refer to Appendix B for copyright permission.

Authorship Contribution Statement

The contribution of Isaac Greene (Candidate) was 75%. Isaac undertook question development, USQ internal ethics approval process (H20REA145, refer Appendix C), online survey instrument design, participant invitation, data anlysis and interpretation, drafting, revising and finalisation of the manuscript. Assoc. Prof. Weena Lokuge (Principal Supervisor) and Prof. Warna Karunasena (Supervisor) contributed to question development, review of survey instrument, participant invitation, technical input and editing/co-authoring of the manuscript. These contributions were 15 and 10% respectively.

Linking Manuscript to Research Outcomes

A survey was prepared and used as an instrument to document the opinions and experiences regarding floodways from industry based subject matter experts. The survey was designed to support and provide further context to the findings derived from the literature survey as detailed in Chapter 2.

The survey instrument (Appendix C) consisted of both closed and open ended questions and was distributed to engineers, asset owners and individuals with experience in floodway construction and maintenance. The survey focused on floodway structures during extreme flood events and defined the vulnerability, the failure mechanisms and the possible design improvements.

The outcomes of this survey provided qualitative alignment with research and further defined the issues being experienced in practice enabling efforts to improve structural resilience of floodways against extreme flood events to be concentrated. The areas of vulnerability discovered were, downstream floodway components (rock protection, cut-off wall and apron), raised floodway structures, sandy and dispersive soil types and extreme loading from debris impact as a result of debris laden flood waters.

The results from this survey provide a significant knowledge base pertaining to the current experiences and observations of asset owners, along with the challenges they face.

FLOODWAYS AND FLOOD-RELATED EXPERIENCES:

A SURVEY OF INDUSTRY EXPERTS AND ASSET OWNERS

I. Greene, W. Lokuge and W. Karunasena*
Centre for Future Materials, School of Civil Engineering and Surveying, University of Southern Queensland
**Corresponding author*

Following several extreme flood events within Australia, the investigation into floodway construction and maintenance techniques has been undertaken by many government organisations to enhance the resilience of rural communities. To collate the intuition and experiences of engineers, asset owners and individuals undertaking these investigations, a survey containing open-ended and closed-ended questions was prepared and distributed. This survey specifically focused on the vulnerability of floodway structures during extreme flood events, the failure mechanisms observed and feedback relating to design improvements and amendments. The responses received form a significant repository of information pertaining to the design, construction and maintenance techniques being undertaken in practice. The responses also highlight several areas of vulnerability such as the downstream rock protection, raised floodway structures, sandy soil types and impact loading from debris conveyed by floodwaters. Several improvements to the current design practices were also identified.

Introduction

Floodway road structures (Figure 1) are often utilised in lower order roads to facilitate the safe crossing of ephemeral, low flow and large flow water courses through improvements to the stability and predictability of the trafficable surface. During large flow rainfall events, such as that experienced during flooding, traffic movements are often precluded. Floodway construction is highly variable and dependent upon the roads required level of service and the specific site conditions. Variations of floodway structures include vented and unvented, raised or level (relative to the creek bed) and unsealed, sealed and concrete. A typical concrete floodway consists of several major components as outlined in Figure 2. The Austroads (2013) publication, *Guide to Road Design Part 5B: Drainage - Open Channels, Culverts and Floodways* is the nationally accepted design guideline for floodway construction in Australia. In addition, the Queensland Department of Transport and Main Roads (2010) and Main Roads Western Australia (2006) have developed best practice design guidelines. These three design guidelines primarily detail floodway design based upon hydraulic design principles.

Over the past decade, flood-related natural disasters have caused significant and widespread damage to the built environment. Examples of recent flood events include the floods in 2011 and 2013 which brought widespread heavy rainfall to Queensland, the 2016 extreme weather event in South Australia and the monsoonal and cyclonic rainfall events experienced yearly in the northern parts of Australia. As a result of these events and the associated increased floodwaters, floodways sustain increased direct loading and hydraulic influences that often cause significant structural damage. These structures also play a vital role in post-disaster recovery and need to be designed in a manner that allows them to re-open and be in a serviceable state immediately after an extreme flood event.

As a repercussion of the repeat damage being experienced, councils have undertaken numerous betterment projects to increase the structural resilience of floodways against flooding and to reduce ongoing operational expenditure. Significant experience and improved construction techniques have been acquired through the undertaking of these betterment projects relating to the design, maintenance and construction of floodways. To collate this knowledge, a survey focusing on the vulnerability and associated failure mechanisms of floodway structures during extreme flood events was distributed to engineers, asset owners and individuals. Sixty-four completed responses were received and analysed, forming a significant repository of knowledge pertaining to the design, construction and maintenance practices of floodways within Australia.

Figure 1. Floodway structure implemented in a low flow watercourse (Lockyer Valley Regional Council, personal photograph, 16 June 2011).

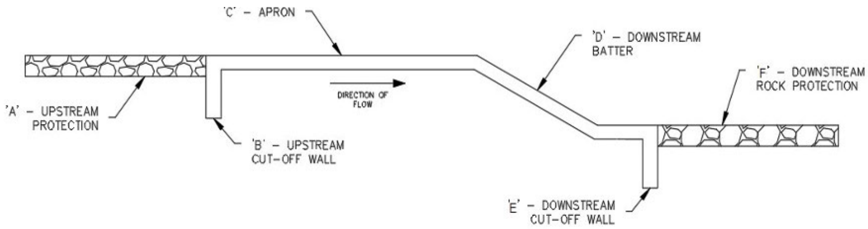

Figure 2. Major components of a typical floodway structure.

International Surveys

Floodways are not unique to Australia and are also utilised in many other countries. Several international authors, Lohnes et al. (2001) and Gautam and Bhattarai (2018), have also conducted research surveys into the suitability of floodway structures with many of the outcomes transferrable to floodway design, construction and maintenance in Australia. Provided below are summaries of these surveys. Although not directly aligned with this Australian survey, they provide an interesting insight which highlights additional items for consideration.

Iowa Survey, United States

A survey conducted by Lohnes et al. (2001) in Iowa, United States was aimed at collecting information relating to floodway crossings, along with suggestions on their practical use. This survey received a total of 70 responses, and the key findings are summarised as follows:

- There was a heavy reliance on in-house design standards and construction (64 per cent), whereas only 32 per cent referenced existing design guides. Four per cent stated that they utilise both.
- Majority of respondents preferred vented floodway constructions (i.e. culverts), as opposed to unvented floodways.
- Respondents suggested that floodways were not suitable in the following applications:
  1. When used with small pipe diameters for vents as they tended to become obstructed with debris
  2. In deep inscribed channels where velocity is often high
  3. In roads crossing watercourses at angles above 10 - 15 degrees
  4. In sedimentary silt soils due to floodway instability, and
  5. Where it is the only access to a local residence.
- Respondents suggested that floodways were suitable in the following applications:
  1. When an oversized drainage structure is incorporated
  2. When situated flush with the creek bed (for unvented floodway applications)
  3. In lower order roads where traffic volumes are low
  4. In ephemeral streams, and
  5. When floodway design permits gradual entry and exit points.
- It was suggested by 70 per cent of respondents that floodways are an excellent substitute for bridges and culverts when appropriate and applicable conditions exist. This asset class can also offer significant capital reduction and satisfactory maintenance periods when constructed correctly.

Illinois Survey, United States

Similarly, a survey was conducted by Gautam and Bhattarai (2018) to summarise the consensus of floodway use. Respondents were from 55 geographical regions within Illinois, United States and the key findings are summarised as follows:

- Floodways were often used within rural settings and on roads with low average daily traffic counts (less than 25 vehicles per day).
- Floodways were often implemented based on economic savings.
- The allowable overtopping duration of floodways was based on the utilisation category and asset importance level; however, the consensus stated that it should be restricted to less than five per centduration per year.
- Floodways should only be

implemented to access residential properties when an alternative route exists.

- A lack of warning signs was found to increase the likelihood of accidents significantly.

To further contribute to this survey knowledge, this research paper presents the results of a survey undertaken in Australia, which focuses specifically on the vulnerability of floodways during extreme flood events.

Survey Methodology

An online survey instrument consisting of 12 questions was prepared using Lime Survey (2020) and received ethics approval from the University of Southern Queensland. Prior to dissemination, the online survey instrument was thoroughly beta-tested to ensure functionality. The target audience consisted of individuals and asset owners with expertise in floodway design, construction and maintenance and included councils, road authorities, technical consultants and IPWEA members. The survey questions were mainly objective, however also incorporated questions that allowed the respondents to detail their experiences and knowledge further. On average, the survey took approximately nine minutes to complete and was open for three weeks.

A range of different open- and closed-ended survey questions were utilised. These survey questions can be further categorised into the following question types:

1. Likert scaled questions – these questions were used initially to measure the respondent's opinion and attitude on the topic
2. Multiple-choice questions – these questions were used throughout the survey to allow the respondents to define answers from a list of pre-defined answers or select 'Other' and explain an alternative answer, and
3. Array open-ended style questions – these questions enabled respondents to formulate their own responses or provide additional comments regarding their experiences.

Responses

The survey was accessed a total of 96 times, for which 64 completed responses (66.7 per cent) were received. The remaining 32 responses (33.3 per cent) were partially completed and were not considered in the analysis (Figure 3). The survey received participation from Qld, NSW, Vic and SA, providing a good cross-section of floodway experiences within Australia.

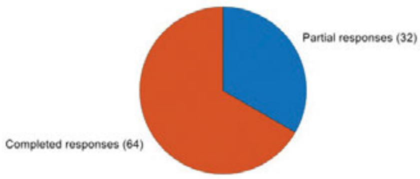

Figure 3. Survey responses received.

Results & Discussion

In your experience, what is the likelihood that a floodway, inclusive of protection, will sustain damage during extreme flood events?

The survey results strongly suggest that floodway structures were "highly likely" (42.2 per cent) or "likely" (40.6 per cent) to sustain damage during extreme flood events (Figure 4). The options "neither likely nor unlikely", "unlikely" and "very unlikely" received 10.9 per cent, 4.7 per cent and 1.6 per cent respectively.

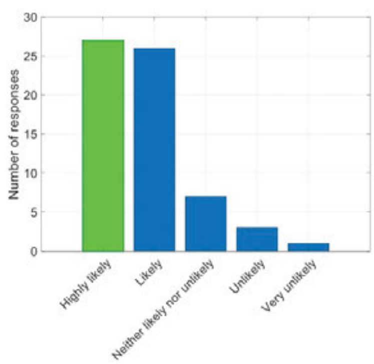

Figure 4. Likelihood of floodways to sustain damage during extreme flood events.

In your experience, which floodway component is most susceptible to damage during an extreme flood event? What is the likely cause of this damage?

It was suggested by respondents that downstream floodway structural components are most likely to sustain damage during extreme flood events. Downstream rock protection ranked highest (65.6 per cent), followed by the downstream batter (12.5 per cent) and downstream cut-off wall (7.8 per cent). Few respondents stated that the upstream rock protection (7.8 per cent), apron (4.7 per cent) and upstream cut-off wall (1.6 per cent) were the most susceptible component to be damaged during extreme flood events (Figure 5).

The respondent's reasoning behind selecting the downstream rock protection was relatively aligned. It was explained that as water flows over the concrete apron and down the batter, velocity increases. This water is then discharged into the downstream tailwater, which is of a much lower velocity causing an abrupt rise and associated turbulent conditions as energy levels dissipate (hydraulic jump). This phenomenon is further exacerbated in flood conditions where significant supercritical flow velocities exist, and the downstream channel is in a submerged/flooded state.

This hydraulic jump and associated turbulence were suggested to be the primary cause of scouring, erosion and "popping" of the downstream dumped rock rip rap protection. Several respondents stated that the use of rock mattresses (gabion baskets), geotextile fabrics, rock pitching with concrete/mortar grouting and anchoring of rock protection to the structure enhanced protection functionality during extreme flood events. One respondent mentioned that by creating a rock pool at the downstream toe of the floodway the locality of where the hydraulic jump occurs can be promoted and rock protection in this area designed accordingly.

Once the downstream rock protection had failed, the downstream cut-off wall was the next component observed to fail.

Other areas of floodway failure reported by respondents were as follows:

- Apron – no allowance for subsoil drainage or weep holes in the concrete floodway structure cause the pavement to remain saturated for prolonged periods and the apron to fail under vehicular loading, and
- Approaches – the approaches were found to be susceptible to wash-out if the floodway structure and subsequent rock protection didn't extend far enough past the high-water level.

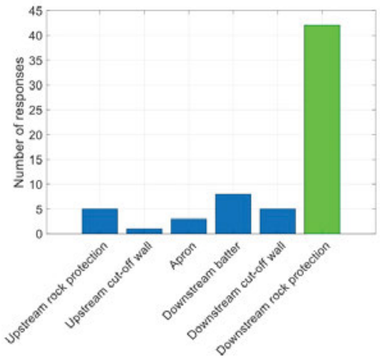

Figure 5. Floodway components most susceptible to damage during extreme flood events.

In your experience, is floodway failure more common in raised floodway structures or floodway structures situated level with the creek bed?

Fifty-two respondents (87.5 per cent) stated that floodway failure was more common in raised floodway structures as opposed to level floodway structures (12.5 per cent).

This response aligned well with the emerging trend of floodway asset owners vertically re-aligning raised floodway structures level with the creek bed after observing significantly less damage post-flood event.

Floodway structures situated level with the creek bed are extremely effective structures since they do not impart a hydraulic control on the watercourse. This eliminates lateral loadings which increase proportionally with velocity, such as drag, debris and impact. The potential of scouring and undermining of the downstream cut-off wall is also significantly reduced as no hydraulic jump or increase in flow velocity exists within the immediate downstream zone (headwater level equals tailwater level). They are also usually a simpler structure to construct and therefore are very cost-effective to implement. However, depth of flow is uncontrolled in level floodway crossings precluding the movement of traffic during anything other than minor rainfall events, thus reducing the level of service that can be achieved (increased time of closure).

Raised floodway structures are critical in applications where control of the flow depth and velocity over the structure is required. These structures reduce the time of closure and facilitate the incorporation of culvert structures allowing the road surface to remain unaffected during minor flooding. Raised floodway structures can also present issues for fish migration and suitable fish passages need to be considered during design.

Is floodway failure more common in certain soil types, if so which type?

78.1 per cent of respondents stated that floodway failure is more noticeable in certain soil types. Out of the multiple-choice list of soil types, a Sandy Soil type received the most responses (56 per cent), followed by Clay Soils (12 per cent), Silty Soils (10 per cent) and Gravel Soils (8 per cent). The option to select 'Other' also existed, which received seven responses (14 per cent) as illustrated in Figure 6. These responses detailed that sodic and highly dispersive soils were also problematic to floodway construction.

Permissible velocity based on the strata characteristics of the creek bed is a significant factor in determining if a non-erodible creek bed exists. Achieving a non-erodible creek bed during design ensures the best probability in minimising the occurrence of creek bed erosion and scour during flooding. Based on the responses from this survey, it can be concluded that soils which lack cohesion (sandy soils) or are dispersive have a higher tendency to be dissolved and erode during extreme flood events. From a lateral loading perspective, soils that lack cohesion also have a reduced ability to resist loading compared to well compacted cohesive soils.

To obtain a non-erodible creek bed, armouring of the creek bed is usually required. This involves selecting a material which has a limiting velocity greater than the calculated flow velocity of the watercourse. The adoption of geotextile fabric materials underneath the rock armouring also assists in mitigating creek bed erosion.

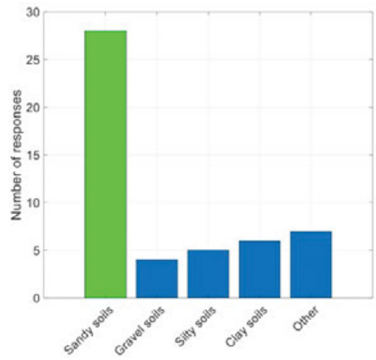

Figure 6. Likelihood of Floodway failure based upon creek bed soil type.

Floodway susceptibility to failure due to impact and debris loading.

Forty respondents (62.5 per cent) stated that increased sediment loading from articles such as organic debris (logs) and boulders have contributed to floodway failure as a result of being conveyed by floodwaters and impacting the floodway structure. Of these 40 respondents, 15 (37.5 per cent) stated that impact from boulders (large rocks) specifically contributed to the failures experienced. The other

25 respondents (62.5 per cent) indicated that the impact of boulders did not contribute to the failures.

Several authors have undertaken investigations into the failure mechanisms of floodways during extreme flood events [BNHCRC, 2015; Wahalathantri et al., 2015; Furniss et al., 2002; GHD 2012]. The research conducted by BNHCRC (2015), Wahalathantri et al. (2015) and GHD (2012), specifically investigated the factors contributing to the many floodway failures reported after the significant and widespread flooding within Australia. These studies have recorded relatively consistent failure mechanisms and attributed floodway failure to the increased sediment loads, organic debris (logs) and boulders being conveyed by floodwaters, impacting the floodway structure and causing significant loading whilst in a submerged state. Furniss et al. (2002) explains that this type of failure is often complicated and is a factor of the specific environment and frequency of landslides, watercourse bank erosion, treefall and other processes occurring upstream of the floodway structure.

Has the respondent undertaken any investigation into different concrete cut-off wall configurations?

Respondents were asked if they had undertaken any investigations into different concrete cut-off wall configurations, including depth and width. Out of the 64 respondents, seven (10.9 per cent) stated that they had undertaken investigations into different cut-off wall configurations, while the remaining 57 respondents (89.1 per cent) indicated that they had not.

Of the seven respondents that had undertaken investigations into different cut-wall configurations, four respondents (57.1 per cent) had considered both the upstream and downstream cut-off walls, two respondents (28.6 per cent) had only considered the downstream cut-off wall and one respondent (14.3 per cent) had only considered the upstream cut-off wall as illustrated in Figure 7.

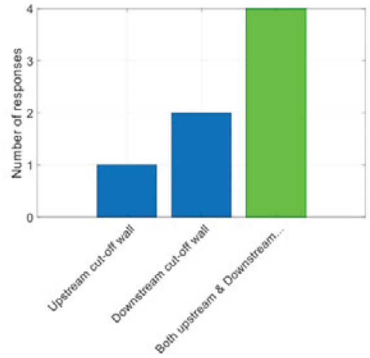

Figure 7. Cut-off wall location which respondents had undertaken investigations into.

Most of these investigations were regarding cut-off wall depth, width and the incorporation of structural steel reinforcement to mitigate floodway superstructure failure in the occurrence of downstream scouring and undermining.

The cut-off wall provides several critical functions within the floodway structures as follows:

1. Increases stabilising moment by providing a greater distribution area to the adjoining soil. This significantly increases the structure's ability to resist horizontal loading in the direction of flow resulting from debris, drag and impact.
2. Prevents groundwater from flowing freely through the underlying foundation material.
3. Protects the structures foundation material against scouring and undermining, both at the upstream, downstream and the transition from the concrete floodway back to the road surface at the approaches.

A cut-off wall depth greater than 900 mm, which extends around the entire perimeter of the concrete floodway structure was generally found sufficient to achieve the required functions as detailed in points two and three above. To satisfy point one, the bending moment distribution needs to be determined and concrete width and structural reinforcement designed accordingly.

Has the respondent trialled other investigations into floodway design improvements and modifications?

The respondents were asked if they had trialled any other improvements or amendments to floodway design to increase structural resilience against flooding. Twenty-three respondents (35.9 per cent) stated that they had, while the other 41 respondents (64.1 per cent) indicated that they had not. The improvements and modifications to current floodway construction practices from the 23 responses received can be summarised as follows:

Geometric Alignment:

Many respondents are vertically re-aligning replacement and new floodway constructions level with the creek bed. One respondent stated that raised floodway structures should only be adopted if the height differential between the headwater and tailwater level was limited to 300 mm. This statement was based on reduced damage observations to downstream floodway components after adopting such a limit.

Floodway Structure:

It was suggested that by creating a monolithic floodway structure (constructing in one continuous pour), the structure durability was significantly increased, i.e. no cold joints. Furthermore, the use of concrete structures in-lieu of sealed and unsealed floodways was found to be essential in achieving structural resilience against flooding.

Pavement Materials:

Stabilisation of the foundation pavement material was highly recommended to ensure that the pavement retains its strength while in a saturated state. Alternative foundation materials such as lean mix concrete and foam bitumen pavement have also been successfully trialled in floodway construction.

Rock Protection:

Respondents suggested that when using rock mattresses, the shoulders need to be appropriately tied into the adjoining banks and concrete nib walls constructed. Rock protection mattresses adjoining the structure should also be anchored to the concrete floodway via appropriate chemset connections to prevent displacement.

Other feedback based on the respondent's experiences in floodway construction and maintenance.

Provided below is a summary of other feedback received based on the respondent's different experiences in floodway design, construction and maintenance.

- Floodways are often a compromise between initial construction cost and ongoing maintenance requirements.
- Subcritical flow over the apron can be promoted by grading the apron towards the upstream direction. A rock pool can also be created in the downstream rock protection at the toe of the structure to help promote the hydraulic jump to occur in a controlled manner.
- The floodway structure should extend far enough past the high-water mark to prevent the road approaches scouring due to afflux.
- The floodway should be geometrically similar each side of the watercourse to ensure flow conditions aren't more concentrated in one area.
- Adopting standard engineering floodway drawings can often prove to be a false economy when it comes to structural resilience. Usually a 150 mm thick concrete apron with SL82 mesh reinforcement is specified, however, most pavement design guidelines specify a minimum thickness of 200 mm for heavy traffic applications.
- Culverts and pipes need to be aligned with the main flow direction. Sufficiently large culverts need to be incorporated when used in floodway applications. The installation of baffles in box culverts restrict flow and have the potential for debris collection and failure due to the associated additional lateral loadings.
- If the level of service for the road allows, implementation of a floodway structure level with the creek bed is beneficial and reduces downstream influences from turbulence caused by a hydraulic jump, i.e. headwater level equals tailwater level.
- The funding requirements to implement concrete floodway structures with hydraulic immunity in rural road networks is often not viable for many rural Councils. Floodway design needs to be performance-based with consideration of how the structure should function to meet the desired level of service.
- From a disaster recovery perspective, it was suggested that the time it takes a damaged floodway structure to return to service should be investigated to reduce access restrictions imposed on communities post extreme flood event. To facilitate this, it was suggested that the following should be investigated further:
- Redundancy (sacrificial damage): investigate methods which would enable only sacrificial damage to occur to floodways for events greater than the design event. This would alleviate the need to undertake full structure replacement, or significant structural rectification works post extreme flood event.
- Accepted emergent work practices: develop guidelines on accepted emergent work practices for post-event recovery, such as the minimum investigation, design work, traffic limitations and reinstatement requirements to be undertaken to enable a floodway crossing to be reopened in a reduced capacity until full reconstruction works can be achieved. As an example, one innovative solution witnessed in a flood event was the use of 1 m^3 concrete blocks (like those used in material loading bunkers) to form temporary upstream and downstream floodway edges.
- Floodways are typically specified as a no barrier type construction, which often causes issues in defining the pavement edge when in a submerged state. Some floodway constructions utilise frangible traffic barriers and pedestrian balustrades; however, these are not explicitly mentioned in the design guidelines. These systems often employ a two-bolt arrangement, where the larger bolt acts as a hinge and the smaller bolt acts as a shear pin. During flood events the pin is sheared from the increased drag and debris loading, subsequently dropping the barrier.

Conclusion

Significant damage to the built environment has resulted over the past decade due to flood-related hydrological disasters. A research survey was prepared and distributed to collate the experiences and observations of asset owners regarding floodway structures and their susceptibility to failure during flood-related events. The results analysed suggest that flooding is a significant cause of floodway failure. It was highlighted that downstream floodway components including rock protection, cut-off wall and apron, respectively, were the most likely components to fail during exposure to flooding. Raised floodway structures were also discovered to be significantly more vulnerable to failure than structures constructed level with the creek bed. This was a result of imparting a control on the watercourse and the associated hydraulic influences. Furthermore, the watercourse bed soil type was also discovered to be a significant contributor to floodway failure, thus highlighting the importance of achieving a non-erodible creek bed during design to mitigate creek bed erosion and scour, particularly in soils that lack cohesion or are dispersive. Several asset owners had also undertaken investigations into improvements pertaining to the design, construction and maintenance of floodway structures and provided their findings and success in achieving increased flood immunity. The responses received from the undertaking of this survey provide a significant knowledge base of the current experiences and observations of asset owners.

Further Work

The failure mechanisms recorded within this survey will be used to validate numerical floodway modelling outputs. Based on the numerical analysis results, a simplified structural analysis method will be derived, allowing designers to specify structural reinforcement requirements to enhance structural durability, strength and serviceability. A new design guideline incorporating this structural design process is the end utilisation deliverable for this research.

Acknowledgements

The authors would like to acknowledge the support of the Commonwealth of Australia through the Cooperative Research Centre program; *Bushfire and Natural Hazard CRC* and the *Research Training Program (RTP) Scholarship*.

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Annexure to Publication 1

Annexure to address the Examiners' suggested revisions to the published manuscript, *"Floodways and Flood-Related Experiences: A Survey of Industry Experts and Asset Owners"*, in *Engineering for Public Works.*

1. Replace Figures 4 to 7 with:

   Figure 4:

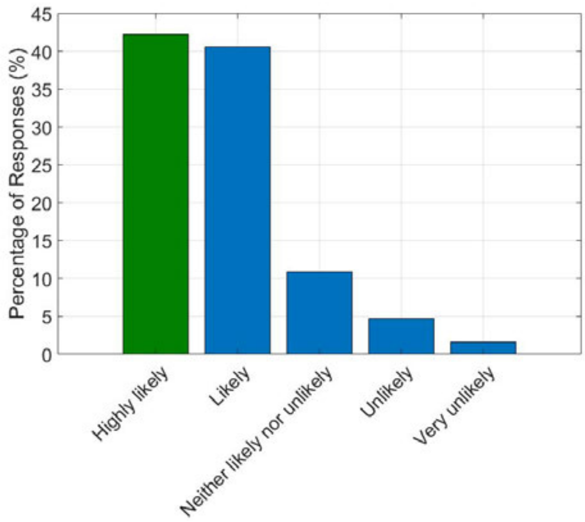


   Figure 5:

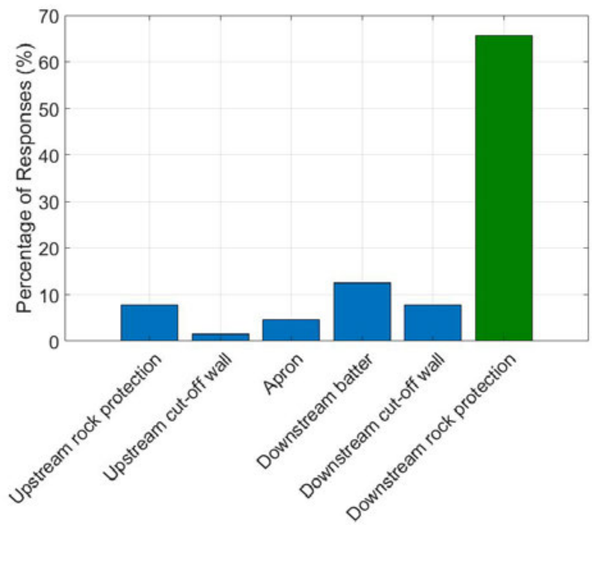


   Figure 6:

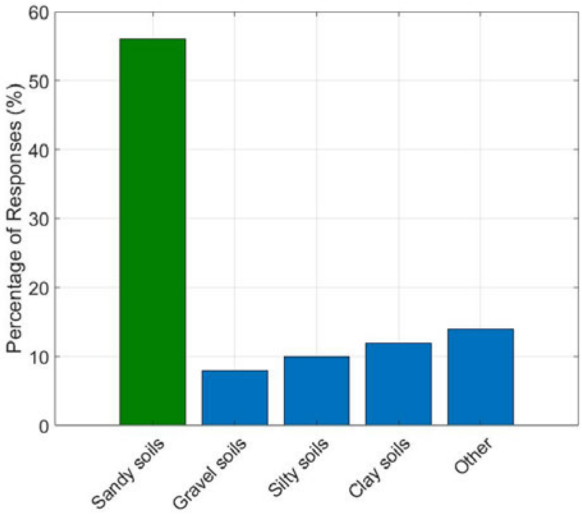
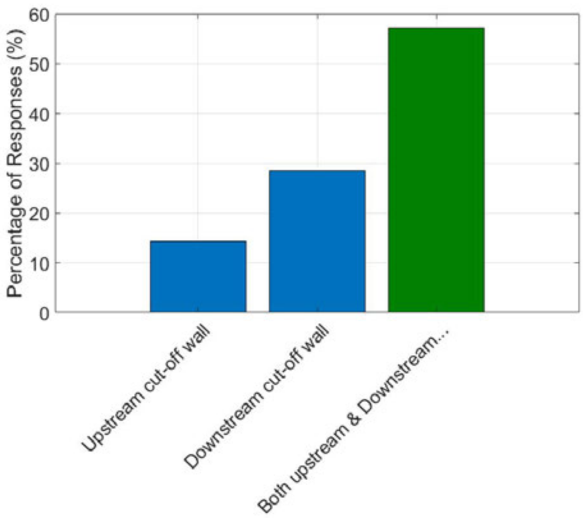


Figure 7:



2. Add the below paragraph to the end of sub-section titled, *"In your experience, which floodway component is most susceptible to damage during an extreme flood event? What is the likely cause of this damage?"*:

   Chahartaghi et al. (2021) states that the susceptibility of downstream scour and erosion in the application of hydraulic structures is well documented as presenting design challenges for hydraulic engineers. Scour and erosion in these areas results from the localised acceleration and increase in energy over the structure, causing large secondary flows and a vortex system induced by a hydraulic jump (Dargahi, 2003). This is particularly pronounced in the application of raised hydraulic structures and results in the structure loosing stability through being undermined and eventually complete structural failure. Downstream scour and erosion can also significantly change the topography of the watercourse.

3. Add the following references and sentences to sub-section titled, *"In your experience, is floodway failure more common in raised floodway structures or floodway structures situated level with the creek bed?"*:

   (a) Add the following sentence after sentence *"They are also usually a simpler structure to construct and therefore are very cost-effective to implement"*.

   This finding was also established in the research by AlTalib et al. (2019), who investigated the hydraulic jump and energy dissipation effect downstream of a weir structure. Within this research it was deduced that a weir structure with a lower height resulted in the best energy dissipation and hydraulic jump length.

   (b) Add the following sentence after sentence *"Raised floodway structures are critical in applications where control of the flow depth and velocity over the structure is required"*.

   Nourani et al. (2021) states that broad-crested weirs are commonly used in conveyance structures such as floodways and dams to control, measure and regulate the hydraulic characteristics of waterways.

   (c) add the following sentences after sentence, *"Raised floodway structures can also present issues for fish migration and suitable fish passages need to be considered during design"*.

   Grimardias et al. (2022) explains that fish passage issues result since raised hydraulic structures present physical obstacles, which act as barriers and either totally or partially block fish passage, migration routes and fish dispersion. Raised structures also alter the flow characteristics (velocity and depth) of the waterway modifying ecological conditions.

4. Add the following references and sentences to sub-section titled, *"Is floodway failure more common in certain soil types, if so which type?"*:

   (a) Add the below reference at the end of sentence, *"Achieving a non-erodible creek bed during design ensures the best probability in minimising the occurrence of creek bed erosion and scour during flooding".*

   (Mustaffa et al., 2013)

   (b) Add the following sentence after sentence *"Based on the responses from this survey, it can be concluded that soils which lack cohesion (sandy soils) or are dispersive have a higher tendency to be dissolved and erode during extreme flood events".*

   Postacchini and Brocchini (2015) explain that the action of scour and particle movement within cohesive and non-cohesive soils is vastly different. The movement threshold for non-cohesive granular sediments is a product of particle size, density, shape, packing and orientation, while erosion within non-cohesive sediments is reliant on shear stress, shear strength, and also the chemical and physical bonding of soil particles (Najafzadeh et al., 2013). Postacchini and Brocchini (2015) further explain that for cohesive soils, often much larger forces are typically required for particles to detach and for movement to be initiated, as opposed to non-cohesive particles which require much lower forces to be entrained.

   (c) Add the following sentence after sentence *"From a lateral loading perspective, soils that lack cohesion also have a reduced ability to resist loading compared to well compacted cohesive soils".*

   This is supported through Lazcano et al. (2020) experimental work into the affect of soil cohesion on bearing pressure, which concluded that the ability of a soil to resist bearing pressure increased with increased cohesion for the same pore pressure.

5. Add the following references and sentences to the end of sub-section titled, *"Has the respondent undertaken any investigation into different concrete cut-off wall configurations?"*:

   Rahim (2018) explains that hydraulic pressure directly affects the cut-off wall, which disburses these forces to the adjoining soil material. Mansuri and Salmasi (2013) conducted a sensitivity analysis using numerical methods into different cut-off wall configurations within drainage structures in terms of seepage rate and flow velocity. In this research it was deduced that the upstream cut-off wall had the largest influence on uplift force and the downstream cut-off wall had the largest influence on the exit gradient. Increasing the depth of both the upstream and downstream cut-off walls resulted in a reduction of both uplift force and hydraulic exit gradient.

6. Add the following sentences and references to the end of sub-section titled, *"Geometric Alignment"*:

   Geometric alignment in design guidelines is determined based on hydraulic capacity, geometric road design standards, vehicle safety, effect of backwater and structural stability (Main Roads Western Australia, 2006; Austroads, 2013; Department of Transport Main Roads, 2010). Within these design guidelines emphasis is placed on vehicle safety and geometric road design practises, which include; not locating the structure on a horizontal bend, ensuring grade is kept level and that adequate sight distance is maintained (Austroads, 2013). An exception to this is a crossing which is level with the creek bed, where grade should equal that of the natural stream to avoid variations in flow depth over the structure (Main Roads Western Australia, 2006).

7. Add the following sentences and references to the end of sub-section titled, *"Floodway Structure"*:

   Illangakoon et al. (2019) explains that cold joints are formed between two layers of concrete when the second layer is placed after the vibration limit has been reached on the first layer, thus resulting in a lack of intermixing between layers. Cold joints in concrete structures results in premature deterioration due to water leakage and strength reduction. Monolithic structures are cast within one pour creating greater connection integrity (Li et al., 2022).

8. Add the following sentences and references to the end of sub-section titled, *"Pavement Materials"*:

   A porous pavement, such as no fines concrete enables the efficient flow of liquid through the pavement to an incorporated drainage system, thus significantly reducing pore pressure within the pavement layer during periods of inundation (JTTE Editorial Office, 2021). Wilton (2014) undertook a case study in Inglewood, NSW into unsealed foam bitumen stabilised granular pavements and conventional granular pavements, which had been exposed to a six week deluge of rain totalling 142 mm. As a result, the conventional granular pavements were reported as destroyed, while the stabilised pavements only required light patching. This is largely due to the foam bitumen binding the pavement to form a water-tight matrix, yet providing greater flexibility due to the rubber content within the bitumen.

3.2 Experimental Investigation

Publication II: Qualitative Investigations into Floodways Under Extreme Flood Loading

Greene I, Gunasekara C, Lokuge W & Karunasena W, "Qualitative investigations into floodways under extreme flood loading", *Journal of Flood Risk Management.* Submitted - February 2022, Currently Under Review.

This chapter contains the Author's version of the above submitted manuscript.

Authorship Contribution Statement

The contribution of Isaac Greene (Candidate) was 60%. Isaac undertook experimental program development, numerical modeling of trial cases, numerical modeling of the final case, laboratory data analysis, interpretation and comparison and drafting of the manuscript. The contribution of Dr. Chamila Gunasekara (Research Fellow) was 20%. Chamila undertook experiment program development, resource procurement, laboratory work using RMIT laboratory facilities and preliminary data analysis. Assoc. Prof. Weena Lokuge (Principal Supervisor) and Prof. Warna Karunasena (Supervisor) contributed to experimental program development, guidance and technical input, and input into the interpretation of results. Their contributions were both 10% respectively.

Linking Manuscript to Research Outcomes

An experimental investigation was undertaken to analyse the behaviour of scaled model concrete floodway test specimens under a concentrated, centrally placed horizontal load. This loading was in response to the results in the industry based survey which suggested impact loading from debris in flood laden water was a significant cause of floodway failure during extreme flood events. The results from this experiment were then compared to a numerical finite element model and correlation made with the responses received in the industry based survey from Section 3.1.

The specific objective was to develop an experimental program to test floodway structures within a soil box to enable displacement and crack propagation to be observed. These observations were then compared to the results form the numerical model and the responses received within the industry-based survey. Qualitatively, the crack propagation and displacement results correlated closely to the strain concentrations and displacements identified within the numerical simulation results. Further, the tendency for the upstream end of the floodway to lift and displace vertically in the experiment also aligned closely with numerical model results.

Qualitative investigations into floodways under extreme flood loading

I. Greene[*a] | C. Gunasekara[b] | W. Lokuge[a] | W. Karunasena[a]

*Corresponding author

Email: Isaac.Greene@usq.edu.au

[a]Centre for Future Materials,

School of Engineering,

University of Southern Queensland,

Queensland, 4300, Australia

[b]School of Engineering,

RMIT University,

Victoria, 3000, Australia

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Keywords: Debris Flow; Floods; Floodways; Infrastructure; Flood Damages; Hydraulic Structures; Resilience; Modelling.

ABSTRACT

This research undertakes a qualitative comparison of floodways subjected to a centrally placed horizontal load based on experimental results, numerical analysis, and an industry-based survey. This loading replicates an accidental loading scenario experienced by concrete floodway structures when boulders or other debris impact the structure during extreme flood events, which generate significant horizontal loads. Developed during this study was an experimental program to test scaled model concrete floodway specimens within a soil box to enable displacement and crack propagation to be observed. The experimental observations were then compared to the industry-based floodway specific survey results and a numerical model. Qualitatively, the crack propagation and displacement results correlated closely to the strain concentrations and displacements identified within the numerical

simulation results. The primary outcome was the discovery that several different failure modes exist for floodways, including structural concrete failure, yielding of adjoining soil material causing displacement, and hydraulic failure through scour and erosion in the downstream vicinity.

ACKNOWLEDGEMENTS

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1. INTRODUCTION

Following a recent series of natural disasters and the expectation that weather induced events will become more common and severe due to changing climatic conditions, research into resilient infrastructure has received growing attention (Kuang and Liao, 2020; Bocanegra and Frances, 2021). Floods are a frequently occurring and damaging natural disaster causing significant economic loss and damage to the built environment (Xiao et al. 2021). Small road structures, such as floodways (Figure 1), are designed to assist in the safe and expedient vehicular crossing of waterways, increasing rural communities connectedness. Floodways are also referred to as fords and causeways in the international context. Well-connected communities are critical to efficient functionality and economic prosperity, providing vital links between services such as schools, hospitals, and major trade centres (Singh et al., 2021). Due to the reliance on a properly functioning rural road network, the prioritisation of resilient infrastructure is essential. In a flood management context, Disse et al. (2020) describes resilience as reducing negative impacts due to extreme events, which would otherwise have devastating effects on communities.

Floodways, particularly in the Australian context, received very little research attention until the Queensland floods of 2011 and 2013 (Wahalathantri et al., 2018). Research into floodway failures resulting from these flood events typically found the failure to result from extreme loads and velocities associated with flooding (GHD, 2012; Wahalathantri, et al., 2015). Currently, there are no studies documented in literature that have investigated, through experiment, the behaviour of concrete floodways under loadings equivalent to actual flood events. Research conducted by Lokuge et al. (2019) identified common structural attributes relating to vulnerable floodways and summarised the limitations of using finite element analysis to design floodways. Greene et al. (2020a) utilised finite element analysis to extensively investigate floodways and reported that the worst-case loading scenario, using load combinations documented in AS5100.2:2017, *Bridge Design - Design loads,* was impact loading. Impact loading can be considered an accidental loading, like that of an earthquake, which a resilient structure shall withstand as an ultimate limit state loading. Impact loading in a floodway structure occurs when the floodway is submerged, and debris, such as floating logs or rolling boulders, impact the superstructure.

Greene et al. (2020b), undertook a comprehensive Australian based industry survey in 2020 to investigate the experiences of asset owners concerning extreme flood events and the prevalence of impact related failures, thus providing a qualitative dataset in relation to practical experiences post extreme flood events. In the International context, two other floodway specific surveys were undertaken by Lohnes et al. (2001) and Gautam and Bhattarai (2018). The survey conducted by Lohnes et al. (2001)

was in response to the development of a design guideline with respondents from various municipals within the United States. The survey outcomes suggested a strong dependence on in-house design standards, a preference towards vented floodways, a summary of floodway applications and that floodways can be an excellent substitution to bridges and culverts when appropriate and applicable conditions exist. The survey by Gautam and Bhattarai (2018) summarised the consensus of floodway uses as being within rural settings, on roads with low average daily traffic volumes, to provide an economical solution to bridges and culverts and that the overtopping duration should be based on utilisation category and limited to less than 5% per year.

To enable the effective design and redistribution of stresses within the structure and to enhance the resilience of concrete floodways, it is important that the crack distribution within the concrete structure is known (Metwally, 2017). Concrete floodway structures, like bridges, are large and complex, creating difficulties in undertaking full-scale experimental analysis (Al-Rousan et al., 2020). Therefore, alternative methods to analyse the behaviour of these types of structures is required. Finite element analysis is a widely accepted and versatile engineering tool that can be used to analyse the behaviour of structures (Venkatachalam et al., 2021). Finite element analysis can provide solutions for non-linear behaviours that are reliable and realistic, enabling it to be used to enhance the fundamental understanding of structural response and optimising design (Metwally, 2017). Further, scale model test specimens enable a physical representation of the structure's response under loading to be observed, providing numerical model confidence based on agreeance.

Figure 1. A typical concrete floodway structure.

The purpose of this investigation is to qualitatively investigate the alignment between scale model concrete floodway test specimens, a numerical model and responses received through an industry-based survey. The loading applied within the investigation was based on a centrally placed, horizontal loading that recreates the significant horizontal loading experienced by concrete floodway structures during flood when debris such as logs, and boulders impact and accumulate against the structure during extreme flood events. The specific research objective was to develop an experimental program to test floodway structures within a soil box, enabling crack propagation and displacement results for the floodway test specimens to be observed and compared to that achieved through numerical modelling. The numerical model replicates the geometric and mechanical properties used within the experiment. Furthermore, hydraulic concepts from the industry survey were explored using computational hydraulic modelling.

2. INDUSTRY SURVEY

The authors conducted a survey of Australian engineers to collate the knowledge and obtain an overview of experience regarding floodway vulnerability and failure mechanisms during extreme flood events (Greene et al., 2020b).

2.1 Survey Rational

The survey was commissioned to further explore the research findings from GHD (2012) and Wahalathantri, et al. (2015) in the literature review, which stated that extreme loads and velocities associated with flooding typically resulted in floodway failure. Furthermore, because of the frequency of failure, many councils have undertaken betterment projects to reduce operational maintenance requirements through measures that act to increase the structural resilience of floodways. The primary objectives of this survey were to:

- Determine the prevalence of floodway failure due to extreme flood events.
- Determine the components and design influences that cause structural floodway vulnerability.
- Determine environmental factors that influence structural floodway vulnerability.
- Collate the previous investigations and improvements that asset owners have implemented to increase floodway resilience.

An online survey instrument was developed using Lime Survey (2020), with questions designed to target responses relating to the primary survey objectives. A total of twelve questions were developed, which consisted predominantly of objective-based questions, but also incorporated short answer responses. The target audience included individuals or asset owners that had direct experience in floodway design, construction, and maintenance. The survey was accessed 96 times, of which 64 complete responses were received. Partial or incomplete surveys were not considered. The survey took respondents an average of nine minutes to complete.

2.2 Survey Results and Discussion

This section summarises the key qualitative findings from the industry survey into main topics based on responses only. For complete survey results, refer to *"Floodways and Flood-Related Experiences: A Survey of Industry Experts and Asset Owners"* (Greene et al., 2020b, pp. 69-75).

2.2.1 Floodway failure likelihood and the most susceptible floodway components to fail.

Survey respondents stated that floodway structures were "highly likely" (42.2%) and "likely" (40.6%) to be damaged, inclusive of rock protection during an extreme flood event (Figure 2). The remaining respondents stated that floodways were “neither likely nor unlikely” (10.9%), “unlikely” (4.7%) and “very unlikely” (1.6%) to sustain failure due to extreme flood events.

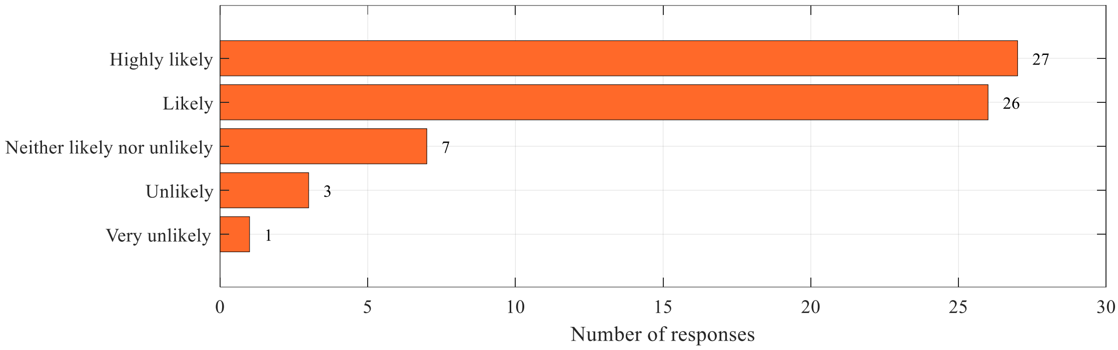

Figure 2. Likelihood of floodways to fail during extreme flood events (Greene et al., 2020b).

Investigating this further, respondents stated that downstream floodway components (Figure 3), including downstream rock protection (65.6%), the downstream batter (12.5%) and the downstream cut-off wall (7.8%) were the most likely components to sustain failure (Figure 4). The apron and upstream floodway components, such as upstream rock protection and the upstream cut-off wall were relatively unlikely to fail and received 4.7%, 7.8% and 1.6%, respectively.

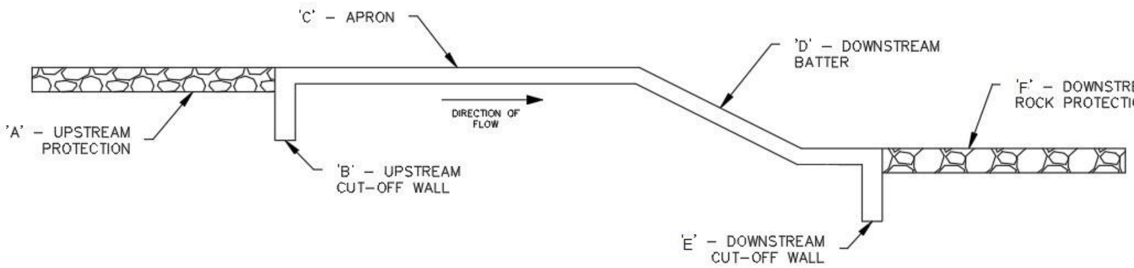

Figure 3. Components of a typical concrete floodway (Greene et al. 2020b)

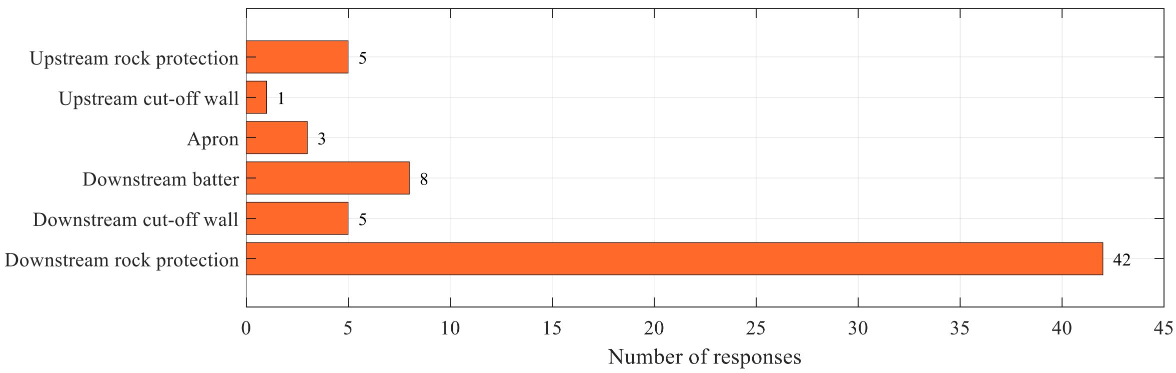

Figure 4. Floodway components most susceptible to failure (Greene et al., 2020b).

Comments received suggested that downstream floodway components were most likely to fail due to the formation of a hydraulic jump and associated turbulent conditions within the vicinity of the downstream cut-off wall and apron. Main Roads Western Australia (2016) explains that flow accelerates down the downstream batter of a raised floodway structure until it penetrates the tailwater, causing the flow to suddenly deaccelerate in a turbulent non-steady state.

A one-dimensional hydraulic computational model using HEC-RAS (U.S. Army Corps of Engineers Hydrologic Engineering Centre, 2019) was created using arbitrary values to explore the tendency of downstream floodway components to fail due to hydraulic effects (Figure 5). Within the graph of outputs, the water surface profile (WS PF) is lower than the critical flow profile (Crit PF) in the locality of the downstream batter. Flows within this region are therefore supercritical (rapid and unstable flow) and do not revert to subcritical until contact is made with the slower moving tailwater through the formation of a hydraulic jump. The formation of a hydraulic jump represents an area of high energy loss and increased erosive potential (increased bed shear stress), aligning with the comments provided in the survey.

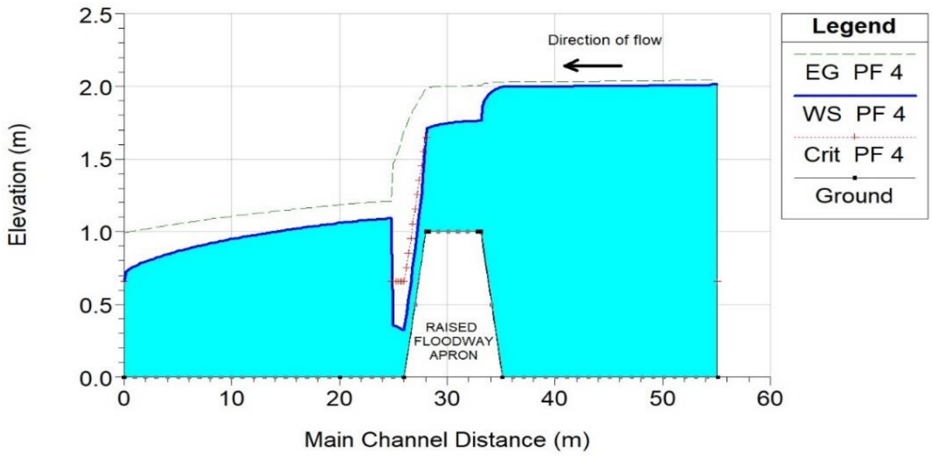

Figure 5. Hydraulic computational model output of a raised floodway structure.

2.2.2 Susceptibility to failure based on floodway type

87.5% of survey respondents stated that raised floodway structures relative to the creek bed were more susceptible to failure than floodways situated level with the creek bed.

Through further computational hydraulic modelling of a level floodway structure (Figure 6), it can be observed that the gradient of the water surface profile (WS PF) is level across the channel and no critical flow profile (Crit PF) exists. This indicates that flows remain subcritical throughout the reach, and therefore flows are expected to behave in a stable and predictable manner. Raised floodway structures as demonstrated in Figure 5 create a significant hydraulic control on the watercourse, resulting in an increase in backwater level and supercritical flows over the structure (rapid and unstable). Oppositely, level floodway structures (Figure 6) do not create a hydraulic control on the watercourse, alleviating the presence of supercritical flows. Furthermore, as there is no surface acting perpendicular to the flow direction, level floodway structures are not subjected to lateral loading cases such as debris impact.

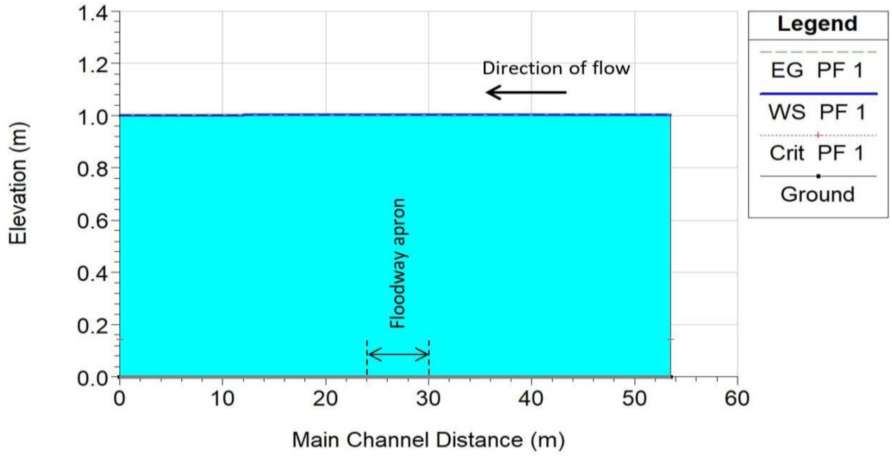

Figure 6. Hydraulic computational model output of a level floodway structure.

2.2.3 Susceptibility to failure based on soil type

78.1% of survey respondents stated that soil type had a significant influence on the prevalence of floodway failure. Out of the available multiple-choice selections, a “Sandy Soil” type received the highest response of 56% (Figure 7). The option to select “Other” and specify a soil type also existed, which received 14% of responses. Soil types defined in the “Other” category consisted of sodic and highly dispersive soils. Other options were “Clay Soils” and “Silty Soils”, which received 12% and 8% respectively. This suggests that highly erodible bed soils and soils that lack cohesion tend to disperse and scour during elevated velocities associated with extreme flood events.

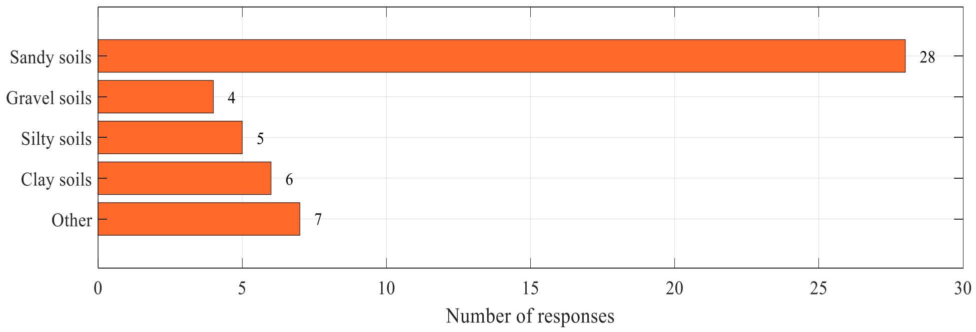

Figure 7. Susceptibility to failure based on soil type (Greene et al., 2020b).

This survey response also aligned with the international survey conducted by Lohnes et al. (2001), which concluded that floodway constructions on loess (sedimentary soils) should be avoided due to its increased erosive potential.

2.2.4 Susceptibility to failure due to debris impact

62.5% of survey respondents reported that floodways were more susceptible to failure due to increases in the organic load being conveyed by extreme floodwaters (Figure 8). More specifically, of the 62.5% of respondents, 37.5% stated that the impact from boulders was a significant contributing factor of failure.

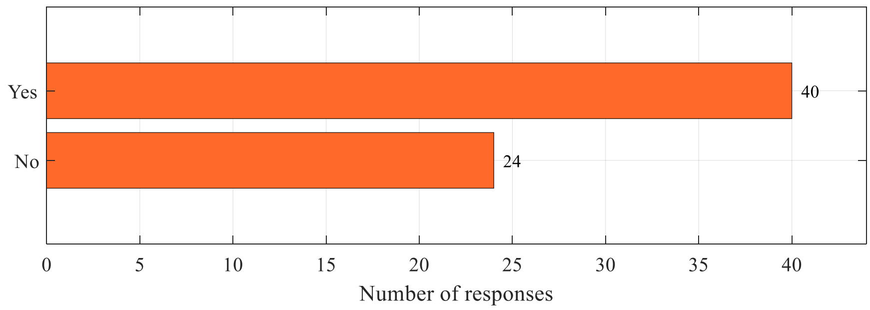

Figure 8. Susceptibility to failure due to debris impact (Greene et al., 2020b).

This failure mode will form the loading case to be investigated further within the experimental program and numerical simulation.

2.2.5 Investigations and improvements being implemented to increase floodway resilience

Although survey respondents stated that floodway failure was highly probable during extreme flood events, very few respondents (35.9%) indicated that they had undertaken investigations into improving floodway resilience through improvements and modifications to standard floodway design. These improvements covered four main categories as follows:

- Concrete cut-off wall configuration - Only 10.9% of respondents had undertaken investigations into cut-off walls, including varying depth, width, and the configuration of steel reinforcement. It was concluded that a cut-off wall depth greater than 900 mm, which extends to the entire perimeter of the floodway structure provided improved resilience.
- Geometric alignment – Several respondents favoured level floodway structures instead of raised structures based on observing reduced damage post extreme flood events.
- Floodway structure – Concrete floodway structures were suggested to significantly improve floodway resilience instead of sealed and unsealed floodway formations.

- Pavement materials – Adopting a lean mix concrete or a foam bitumen pavement material had been trialled in lieu of traditional granular materials to ensure that the pavement could retain its strength while in a submerged state.

After reviewing the survey results of industry experts regarding floodway design, construction and maintenance, validation of these ideas through undertaking an appropriate experimental program and computational modelling is required. The experimental program and numerical modelling will adopt the debris impact loading case identified within the survey with failure mode, crack propagation and displacement investigated.

3. EXPERIMENTAL PROGRAM

The experimental program sought to deduce the displacement behaviour, and visual crack propagation pattern of a 1:7.5 scaled concrete floodway specimen when exposed to a horizontal loading case. The primary focus of the experiment is the production and observation of the mode of failure, stress formation, as well as failure locations/localisations, inclination and spacing. In Section 4 the experiment is numerically simulated under the same conditions to validate the ability of constitutive models to reproduce the behaviour of the experiment concrete floodway model.

3.1 Test Specimen

'Concrete Floodway Type-1', a standard engineering drawing from the Lockyer Valley Regional Council in Queensland, Australia, was selected for use as the geometrical test specimen dimensions (LVRC, 2008). This experimental floodway is geometrically identical to that implemented in practice, however, scaled to 1:7.5 and with a slight amendment to the deck thickness (33.33 mm as opposed to 26.67 mm) to provide an adequate thickness for the use of traditional casting methods (Figure 9). The scale of the floodway was selected based on the maximum size permitted for use in the available laboratory facilities. Reinforcement was omitted within the specimen enabling tensile force localisations to be clearly apparent. The need to scale concrete elements within large-scale concrete structure experiments is a commonly reported limitation in literature, thus the importance to develop a realistic numerical model that can reproduce the behaviour of the scaled laboratory test specimen (Marzec and Tejchman, 2022).

The floodway was assumed to be in an unsubmerged (drained) state during the laboratory experiment. That is, the liquid in the soil is assumed to be free to flow when the load is applied, and pore pressure remains unaffected. This assumption was also applied in the reproduced numerical model, enabling behaviour to be replicated and comparable for validation purposes.

A B2 exposure classification was adopted in terms of durability as floodway structural members are subjected to constant wetting and drying from their continuous contact with water in accordance with AS3600, *'Concrete Structures'* (Standards Australia, 2019). This exposure classification resulted in the requirement of a target compressive strength of 32 MPa. Further, standard formwork and compaction techniques were used, and the formwork retained to ensure adequate moisture was maintained until the commencement of curing, thus, ensuring that target compressive strength was achieved.

A summary of the mix design used for casting the test specimens is provided in Table 1. General Portland cement was the binder type specified. The maximum coarse aggregate size was limited to no more than one-fourth of the thickness of the minimum member (33.33 mm). This equated to the selection of a 7 mm maximum aggregate size. A high slump value was also selected to ensure workability when forming the test specimens. As a result of choosing a high water/cement ratio, the corresponding reduction in compressive strength needed to be factored into the mix design. To measure the compressive strength obtained in practise, three concrete test cylinders were cast, and the 28-day compressive strengths were tested; these strengths correlated closely with the 32 MPa target strength.

A total of two test specimens were built for experimental testing.

Table 1. Concrete mix design used in the floodway test specimens.

Target compressive strength = f'_c 32 MPa

Portland Cement (kg/m^3)	Water/cement ratio	Water (kg/m^3)	Fine Aggregates (kg/m^3)	Coarse Aggregates (kg/m^3)	Target Slump (mm)
450	0.50	225	644	1218	155

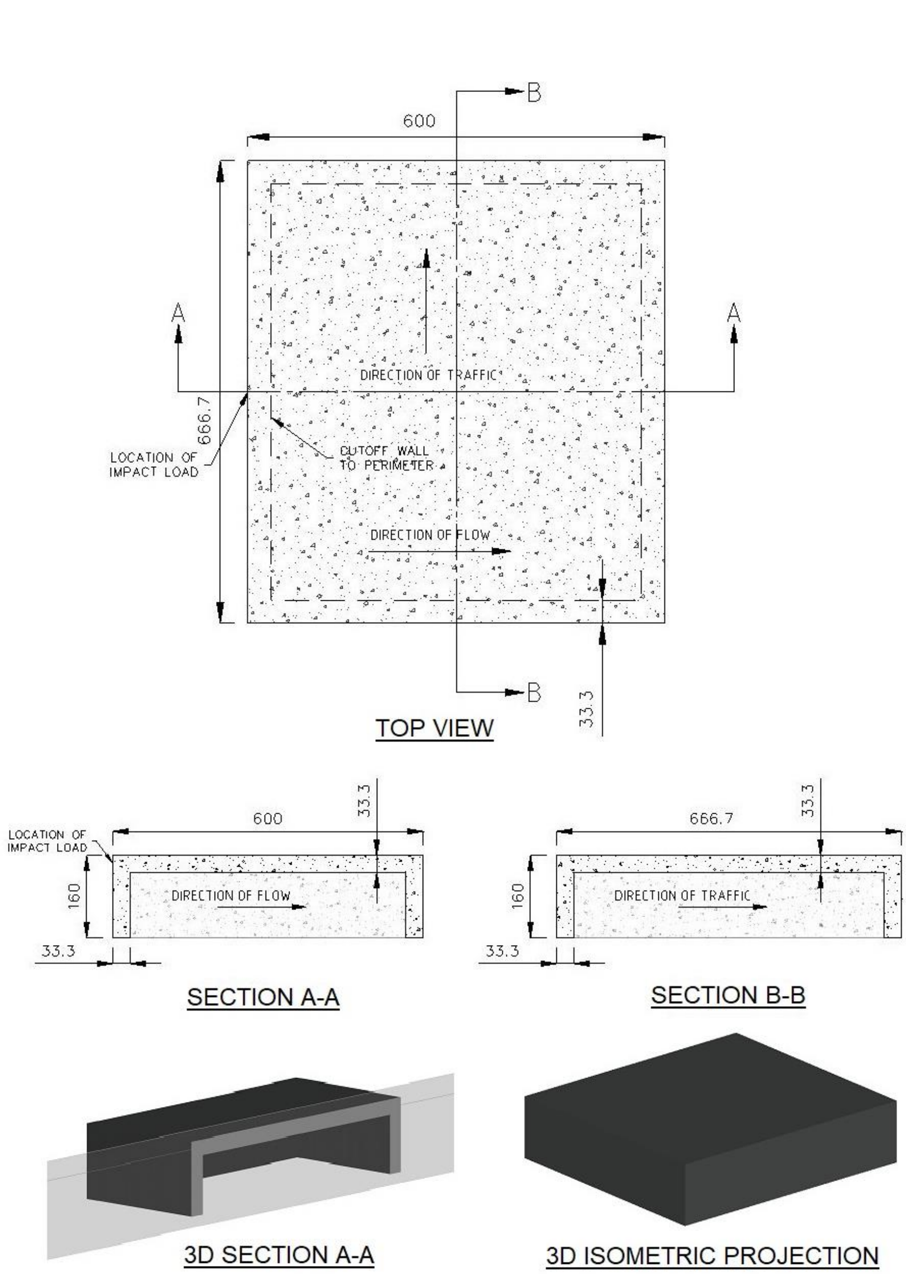

Figure 9. 1:7.5 scale floodway apron model.

Figure 10. Formwork and casting concrete test specimen.

3.2 Test Set-up and Procedure

A soil box with dimensions 1000 mm long, 1000 mm wide and 400 mm deep was constructed to house the floodway model (Figure 10). The dimensions of the soil box were tested through a sensitivity check in Strand7 to ensure no boundary influence within the load range existed. The soil box was used to emulate the conditions of a floodway that has been cast in-situ, such as that observed in practice. The soil box was fully restrained, precluding displacement and rotation in all axes. The soil box was then filled to a depth of 240 mm with soil compacted at optimum moisture content via tamping before centrally placing the floodway test specimen within the soil box. The remainder of the soil box was then filled and compacted homogenously, with the area around the load cell being the exception. Therefore, the floodway test specimen is entirely unrestrained and relies upon the subgrade reaction and frictional force between concrete and soil for support. This support is therefore a function of the shape and size of the concrete cut-off wall surface area, the distribution and intensity of the load and the mechanical characteristics of the soil. The model is thus expected to resist movement up to the maximum frictional force (limiting load) before displacement occurs, or until the soil material yields due to the distribution and intensity of the load being applied.

Loading was applied centrally and progressively increased up to the point of failure (Figure 11). This load application method represents that used in AS 5100.2, which utilises equivalent static forces to analyse the effects of impact loading. This formula is an equation of work, where force is equal to the kinetic energy of the object impacting the structure as shown in Equation 1.

$$F = 0.5\left(\frac{mv^2}{2d}\right) \quad (1)$$

Where,

F = force (N)

m = objects mass (kg)

v = objects velocity (m/s)

d = stopping distance (m)

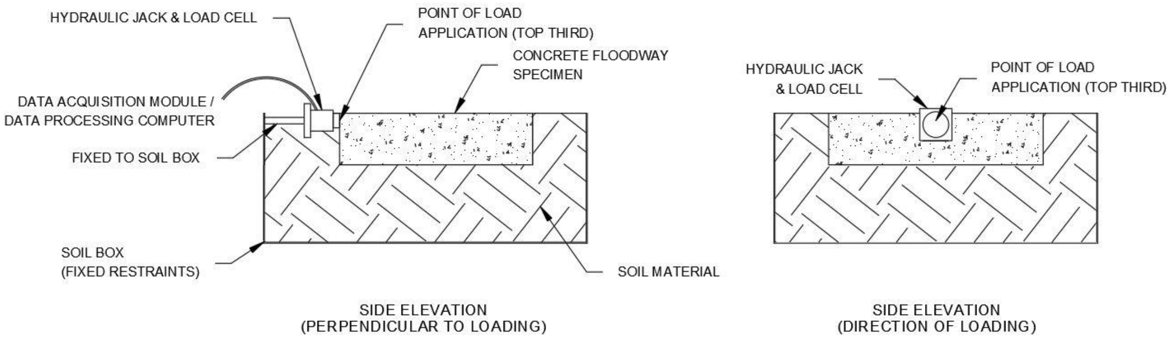

Figure 11. Sketch indicating how the load was applied to the test specimens.

The soil material used was obtained from a Melbourne land excavation site, a soil material which was used as engineering fill/subgrade material in other recent soil-based experiments at the laboratory (Pooni et al. 2020, Pooni, et al. 2021 and Karami et al. 2021). Pooni et al. (2020) explains that the soil is classified as a lean clay with sand (CL) and has a target maximum dry density (MDD) and optimum moisture content (OMC) of 1.62 g/cm^3 and 22.9% respectively.

3.3 Experimental Results and Discussion

Two identical specimens, referred to herein as Specimens 1 and 2, were tested, with loading applied progressively up to the point of failure. A third specimen was untested because of failure occurring during the demolding phase. In all instances, the specimens failed at relatively low load applications (Table 2), resulting in significant variability in recorded strain results. This was attributed to the relatively thin thickness adopted in the specimens of only 33.33 mm, which presented casting and demolding challenges.

Table 2. Description of specimen failure observed.

Description	Failure Load (kN)	Cause
Specimen 1	4.69	The specimen failed at both the upstream and downstream cut-off wall/apron interface (Figure 11).
Specimen 2	0.98	The specimen dislodged upwards within the soil material and failed at the point of load application (Figure 12).

The visual crack propagation pattern of the structure in Specimen 1 (Figure 12) first occurred at the downstream end (C1), propagating parallel to the load along the interface between the apron and the side cut-off walls. As the load application increased, cracking propagated along the interface of the upstream cut-off wall perpendicular to the loading direction (C2) before complete failure occurred (C3). Significant deflection and failure were observed within the failed specimen at the downstream cut-off wall, at the point of load application and the interface between the apron and perimeter cut-off walls, with the apron becoming dislodged. The visual failure pattern of concrete at the point of loading indicates the presence of significant strain localisation at this point. Further, the damage and deflection resulting in the downstream and upstream cut-off walls resulted from the cut-off walls attempting to distribute the loading to the adjoining soil while also providing a stabilising moment (a resistance to overturning) for the structure.

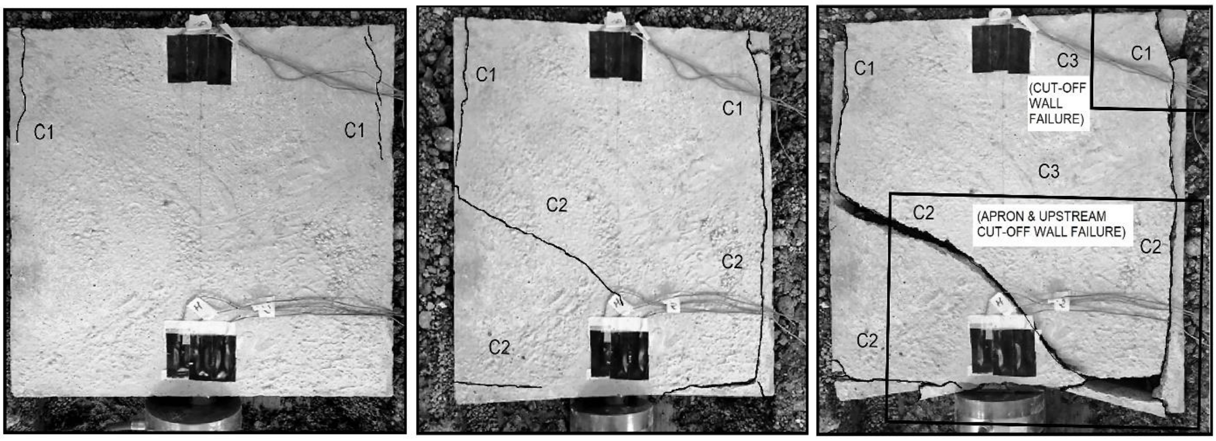

Figure 12. Crack propagation within concrete floodway Specimen 1.

The final crack propagation pattern of the structure in Specimen 2 (Figure 13) was similar to Specimen 1. The initial failure occurred at the loading point, propagating along the upstream cut-off wall and apron interface (C1) before tracking down the side cut-off wall and apron interface parallel to the loading (C2), before complete failure of the upstream cut-off wall occurring (C3).

Within Specimen 2, the structure displaced upwards due to the soil material yielding under the load application (Figure 14). This displacement providing an important insight into the significant overturning moment present and the tendency for the structure to overturn due to a concentrated centrally placed loading.

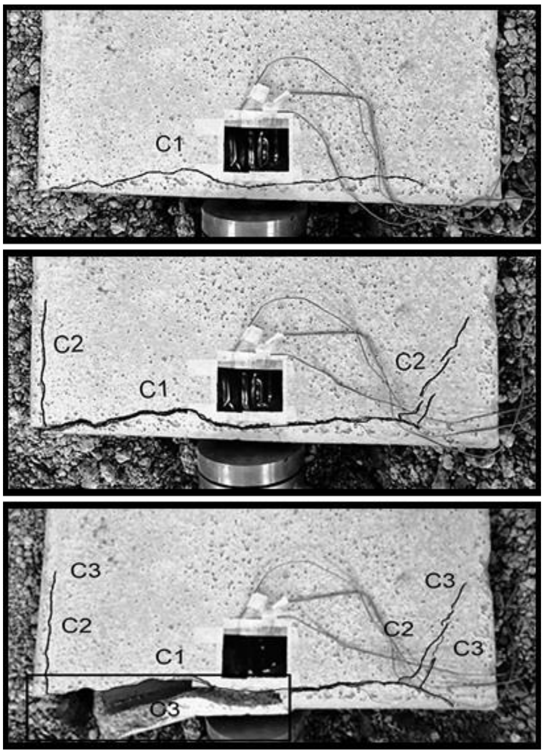

Figure 13. Crack propagation within concrete floodway Specimen 2.

The experimental program deduced the crack propagation pattern for the concrete floodway specimen and enabled visualisation of the displacement present under a horizontal load. The crack propagation and displacement experienced within the experimental program will be validated by comparing results with a three-dimensional finite element model.

Figure 14. Vertical displacement of concrete floodway Specimen 2 at failure.

4. NUMERICAL SIMULATION

4.1 Model Description

A numerical model using finite element methods was created corresponding to the experimental scenario. Strand7 finite element computational software was used to develop the numerical model (Strand7, 2018a). The finite element floodway model was created using four node tetrahedra Strand7 brick elements geometrically identical to the floodway test specimen (Figure 9). Table 3 outlines the mechanical properties of the materials used in the finite element model. The material properties used are typical materials detailed in Austroads (2012) for subgrade material (engineered fill).

Table 3. Material properties based on the soil model parameters used by Pooni et al. (2020).

Properties	Concrete	Engineered fill/subgrade (Pooni et al. (2020)
Modulus (MPa)	31,000	50
Poisson ratio	0.2	0.45
Soil (Bulk) Mass Density (kg/m3)	2,400	1,700
Cohesion (MPa)	N/A	0.01
Dilation (degrees)	N/A	10
Friction angle (degrees)	N/A	20

4.1.1 Boundary conditions

To emulate the boundary conditions of a floodway situated in-situ, the concrete was unrestrained, and the outer soil extent was fully restrained, precluding displacement and rotation in all axes. Further, it was assumed that the contact surface between the concrete floodway and soil was fully bonded.

4.1.2 Constitutive models

To account for the non-linearity in material behaviour, constitutive models were assigned to concrete and soil material types.

Max Stress Yield criterion was used to define the non-linear elastic behaviour of concrete. This required a stress versus strain curve to be assigned in Strand7 to define the non-linear material behaviour of concrete. When stress components exceed the assigned yield strength in either tension or compression then the material is said to have yielded.

Mohr-Coulomb Yield criterion was used to define the elastic-plastic and isotropic behaviour of soil. The Mohr-Coulomb Soil Model within Strand7 (2018a) utilises a generalised form of the Coulomb friction failure law and is an extension of Tresca failure criterion. The yield line defines the values that the stress can take, with the failure envelope at tangents to all Mohr's circles (Strand7, 2018c). The Mohr-Coulomb yield criterion equation utilised is outlined in Equation 2.

$$\tau_{critical} = c \pm \sigma_n \tan \emptyset \tag{2}$$

Where,

$\tau_{critical}$ = shear stress (kPa)

σ_n = normal stress (kPa)

c = cohesion (kPa)

$\emptyset$ = Internal friction angle (degrees)

Strand7 (2018c) explains that Equation 2 utilises the values for cohesion, c, and angle of internal friction, $\emptyset$ to describe the failure surface of the soil and states that the shear strength of the soil, $\tau_{critical}$ is proportional to the normal stress, σ_n as illustrated in Figure 15.

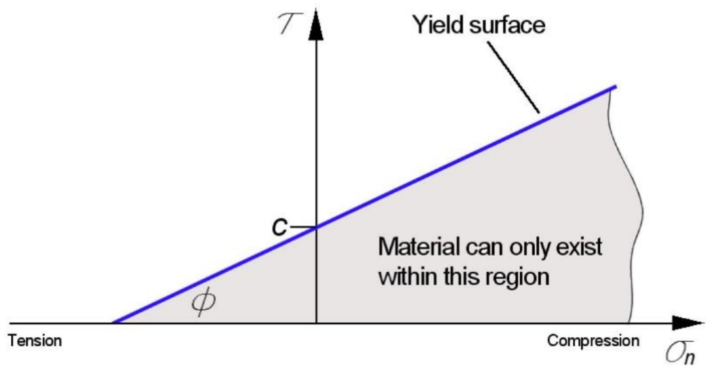

Figure 15. Illustration of the failure surface of soil in Mohr-Coulomb yield criterion (Strand7, 2018c).

4.1.3 Convergence analysis

Mesh and model refinement was undertaken for the experimental scenario. Mesh and model extent refinement is essential to improving the solution's accuracy and ensuring that the model is not over restrained. The methodology for refinement is based on an iterative approach where the model extents and density of the mesh are iteratively increased until results asymptotically converge. At the point of

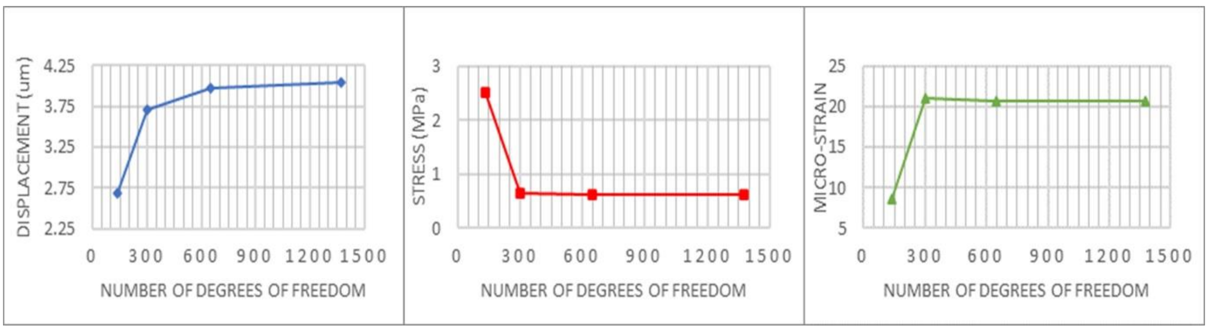

Figure 16. Scenario two finite element model convergence study.

convergence, the answer approximates the correct answer for the least mesh density and model size, thus providing an efficient model for the least amount of computational time. The mesh density was iteratively increased for the experimental scenario until convergence resulted in a minimum of three parameters (Figure 16). This occurred for a model consisting of 1,374 nodes and 648 brick elements.

4.1.4 Load application

The loading applied within the numerical simulation is a horizontal load placed centrally on the upper edge of the upstream cut-off wall and incrementally increased until model failure occurs.

4.2 Simulation Results and Discussion

Non-linear numerical simulation was performed, enabling the stress, displacement, and strain behaviours to be visually and numerically defined for the concrete floodway structure under a significant horizontal loading. Yielding within the supporting soil was discovered as the failure mode for the numerical simulation model producing a maximum compressive strain in concrete of -0.0011 (Von Mises strain value), being well below the maximum limit of -0.0022, where -0.0022 corresponds to the strain at the peak stress assigned in the stress vs strain curve for 32 MPa concrete (Strand7, 2018b).

4.2.1 Visual deformation

During load application, positive displacement in the y-axis was experienced at the upstream end of the floodway. In contrast, negative displacement in the y-axis was experienced at the downstream end of the floodway (Figure 17).

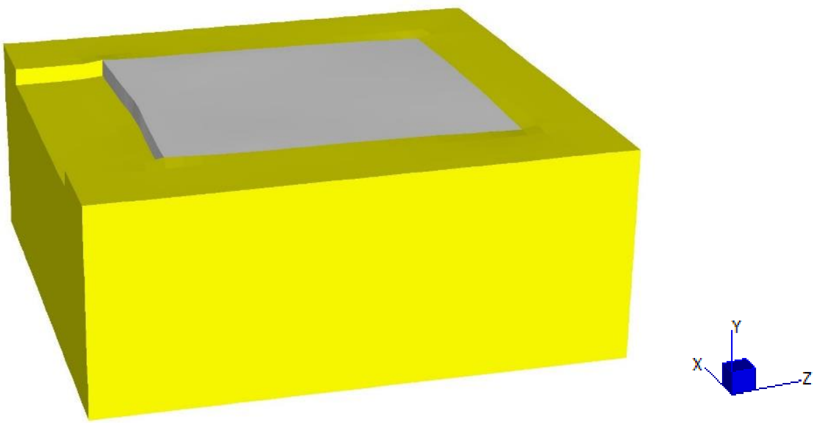

Figure 17. Floodway deformation at 2% displacement scale.

4.2.2 Von Mises stress concentrations

The most significant stress concentration occurred at the loading point and linearly increased to a maximum stress value of 17.18 MPa (Figure 18).

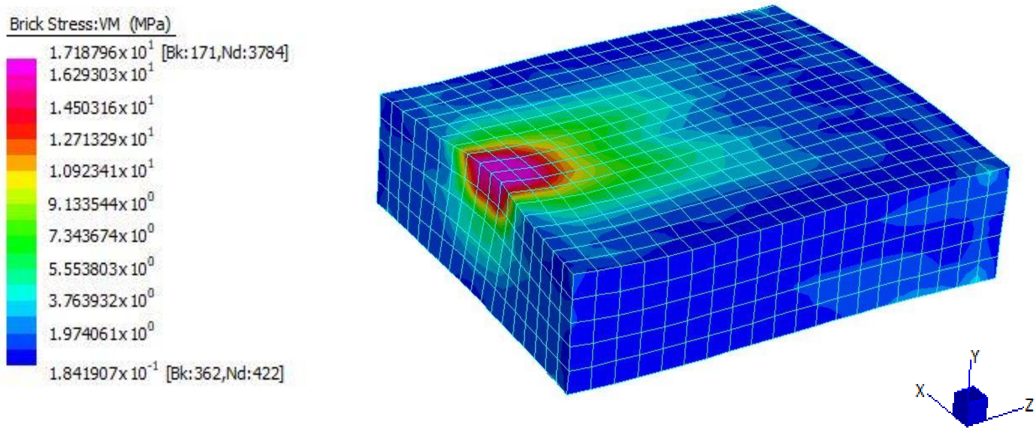

Figure 18. Peak Von Mises stress concentrations.

4.2.3 Von Mises strain concentrations

Figure 19 illustrates the significant strain localisations within the numerical model. The most significant strain localisation occurred centrally towards the upper edge of the downstream cut-off wall. Strain was also concentrated at the point of load application and extended out towards the two side cut-off walls.

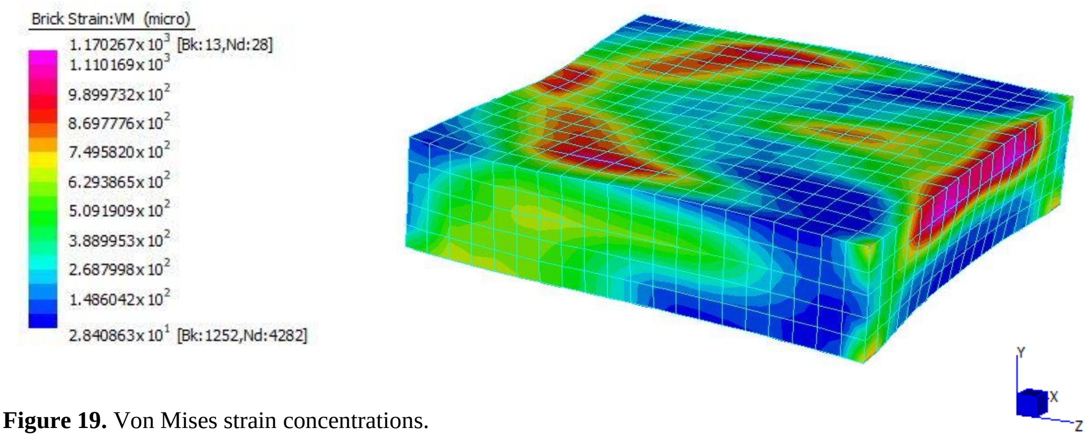

Figure 19. Von Mises strain concentrations.

4.2.4 Structure displacement

The largest horizontal deflection occurred at the loading point and in the positive z-direction (Figure 20). The largest horizontal deflection in the negative z-direction occurred centrally at the end of the downstream cut-off wall.

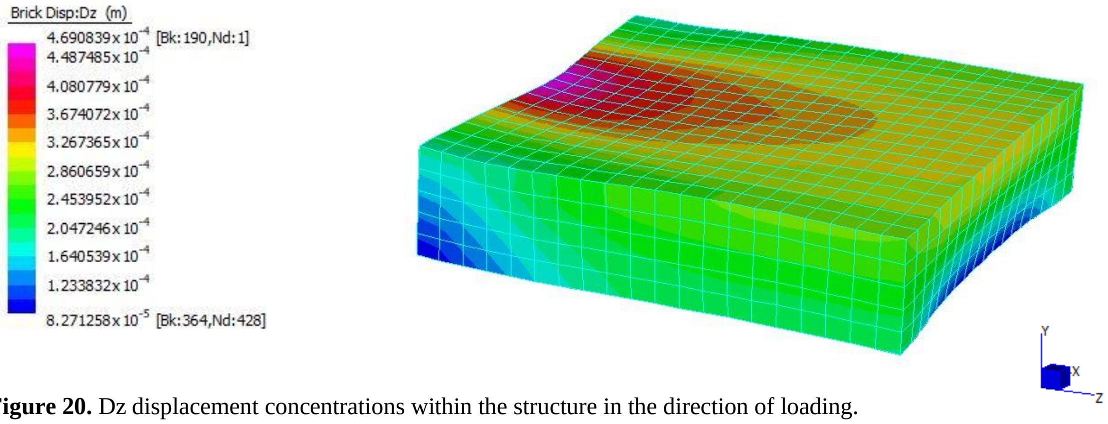

Figure 20. Dz displacement concentrations within the structure in the direction of loading.

4.2.5 Soil Model

This section sets out the element stress components and yield index for the soil model. As the soil was set as drained, the stresses reported in Figure 21 represent the effective stress, which is the stress experienced by the soil skeleton, without the addition of stress due to pore pressure.

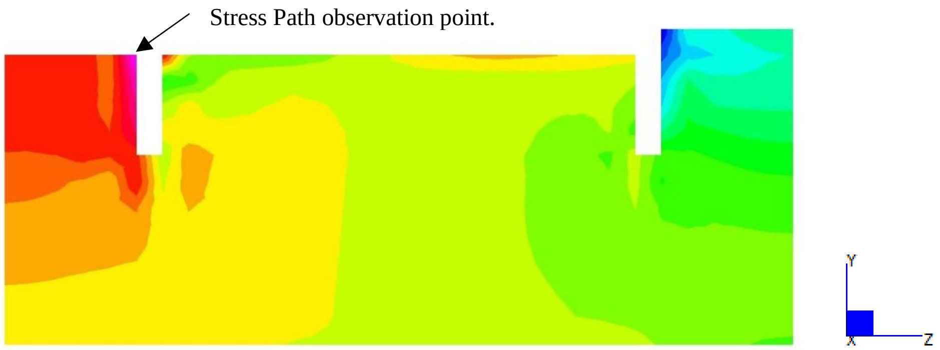

Figure 21. Peak effective stresses within the soil model (z-axis cutting plane).

Figure 22 plots the stress path for the observation point where the soil yield region was identified. The stress path initially follows an elastic path (AB) due to the initial loading of the floodway structure. At Point B the stress path begins to follow the yield surface, until Point C. At Point C stress increases significantly, as plastic shear strains begin to develop and continue to take place along the path CD.

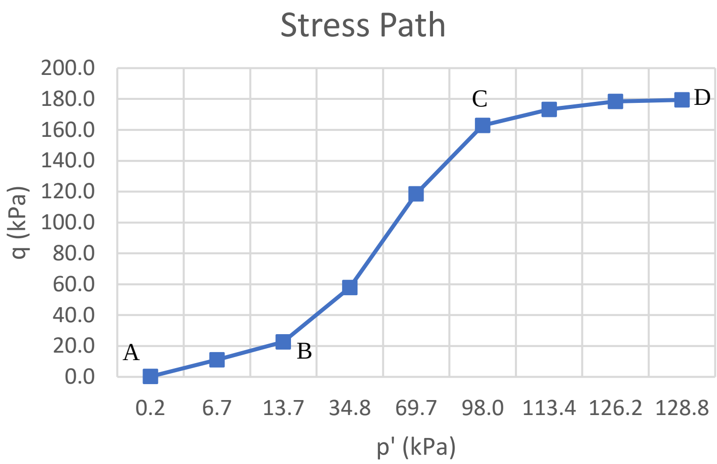

Figure 22. Stress path of Mohr Coulomb soil model during horizontal load application.

Yield Index is a criterion that describes the stress level with respect to the failure criterion employed in the soil model. Based on the contour of the Yield Index, the yield regions can be identified. If the soil model has not yielded in reference to the Gauss point, then a result of 0.0 will be displayed, and if the soil has, then a result of 1.0 will be displayed. Figure 23 illustrates the significant yielding region that exists within the vicinity of the upstream cut-off wall. This yielding results from the soil becoming displaced due to deflection in the cut-off wall as it attempts to distribute the loading to the adjoining soil. Yielding within the adjoining soil material is the failure mode for the numerical simulation model.

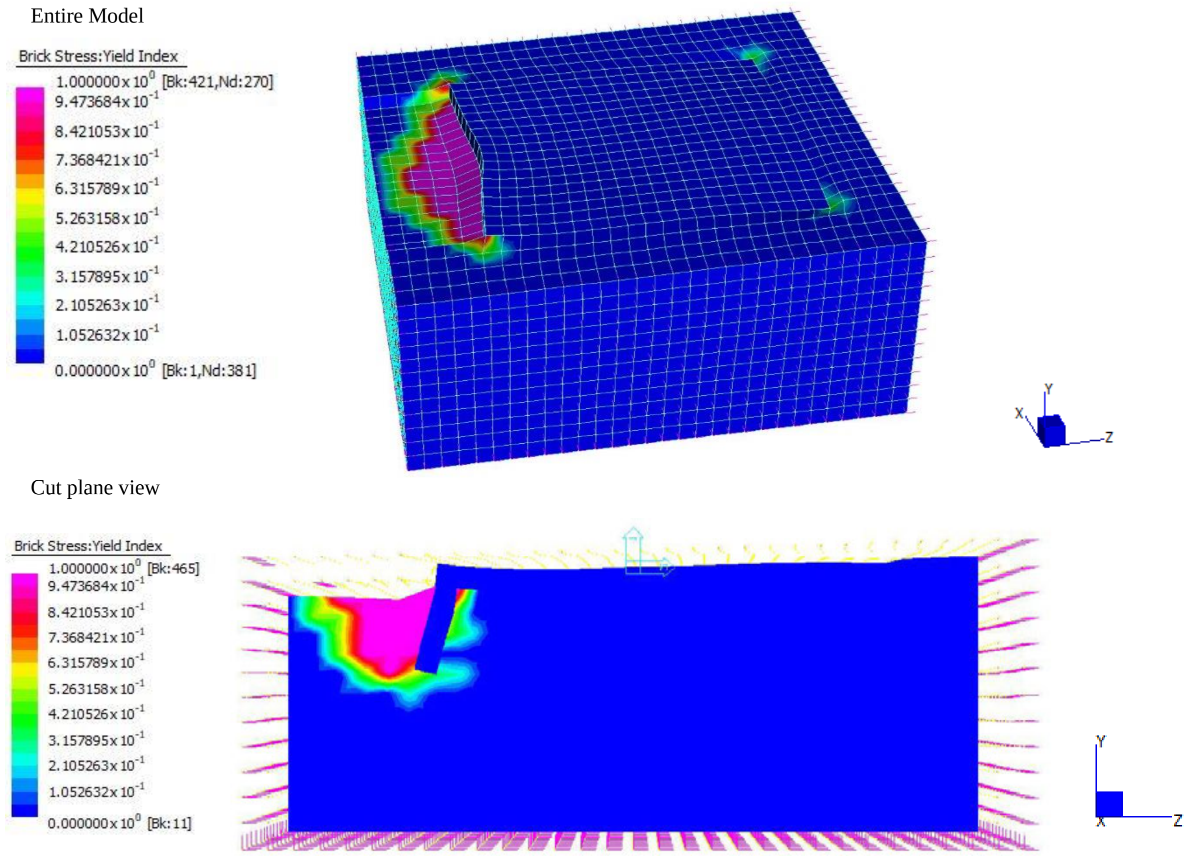

Figure 23. Soil yield index contours.

5. DISCUSSION

The consensus deduced from respondents within the survey was that floodways were significantly susceptible to extreme flood events, with the majority of damage occurring in the downstream vicinity. The susceptibility of failure was also variable based upon the structure's configuration and creek bed soil type, with raised structures and soils that are dispersive or lack cohesion being the most likely to fail. Furthermore, impact from floating debris such as boulders was also a significant contributing factor of failure.

In terms of the experimental scenario, concrete test specimen~~s~~ 2 prematurely failed at relatively low load applications, which was attributed to the relatively thin thickness adopted in the test specimens of only 33.33 mm, which presented demolding challenges. Concrete test specimen 1 failed at a load application of 4.69 kN (479 kg). Within the numerical model the maximum compressive strain did not exceed the maximum strain in 32 MPa concrete of -0.0022, however it did exceed the maximum flexural tensile strength of concrete of 3.39 MPa. This occurred at a loading of approximately 12.5 kN (1,275 kg) and would be characterised by cracking in a very localised area at the point of load application and within the outer tensile face. Further load application past this point resulted in plastic strain development and yielding within the soil material surrounding the floodway model.

Within the experiment, the visual crack propagation pattern within the concrete test specimens under a concentrated horizontally placed load correlated to the significant strain localisations observed within the numerical simulation results (Figure 24). For experimental scenarios, initial cracking was observed at the downstream cut-off wall and apron interface in Specimen 1 and at the loading point in Specimen 2. This was then followed by crack propagation along the side cut-off walls and apron interface before complete failure of either the downstream or upstream cut-off wall or both were experienced. Similarly, in the numerical simulation results, significant strain localisations were observed first at the point of loading, followed by strain propagation to the side cut-off walls, which was closely followed by significant strain being recorded at the downstream cut-off wall.

The respondents within the survey also stated that downstream components were found to be most susceptible to failure due to hydraulic reasoning (supercritical flows reverting to subcritical), thus exacerbating the situation if an accidental loading such as a boulder impacting the superstructure was experienced. Furthermore, the failure pattern from the experiment and numerical simulation (structural failure or displacement) verified the response received in the survey regarding debris loading being a significant contributing factor to floodway failure.

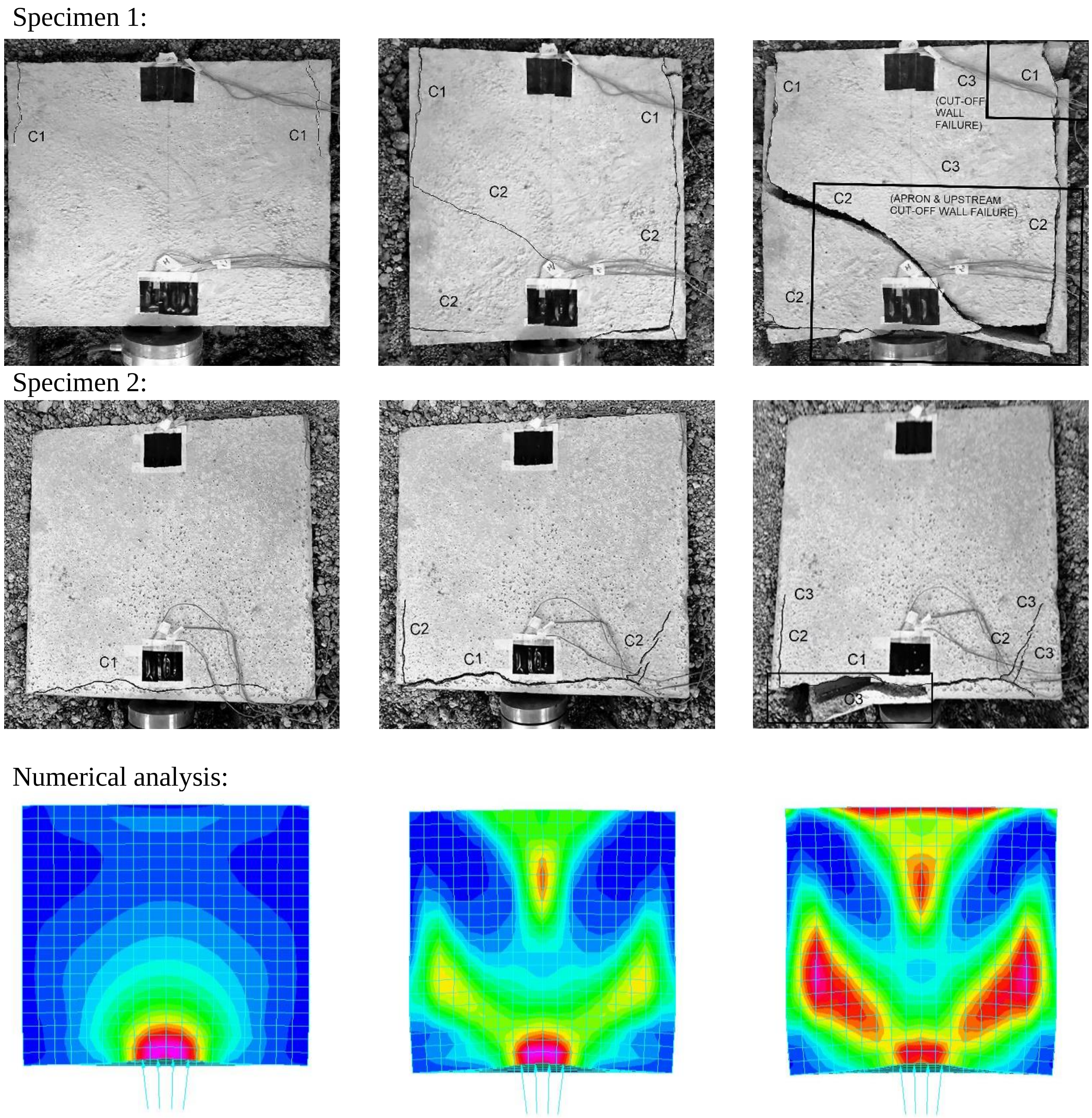

Figure 24. Comparison of crack propagation in experimental results with strain concentrations in numerical model.

Through visual comparison of deflection within the experiment and the numerical model (Figure 25), significant vertical displacement was observed in both the numerical and experiment cases as the soil material yielded. This vertical displacement caused the upstream side of the floodway to lift in the positive y-direction and the downstream side to move downwards in the negative y-direction. In the case of Specimen 2, the significant displacement occurred early in the load application, potentially influencing the crack propagation pattern; however, providing important insight into the significant overturning moment present because of a concentrated centrally placed loading. The perspectives gained through the industry survey and also the survey conducted by Lohnes et al. (2001) suggested that floodways situated within soil materials that lack cohesion had a higher tendency to fail than those that were not. This statement aligns with the significant overturning moment discovered within the experiment and numerical model results, as soils that lack cohesion have a reduced ability to resist horizontal loadings. Furthermore, the successful investigations reported by respondents through increasing cut-off wall depth were discovered to increase stabilising moment through increasing the surface area of the cut-off wall available to resist overturning.

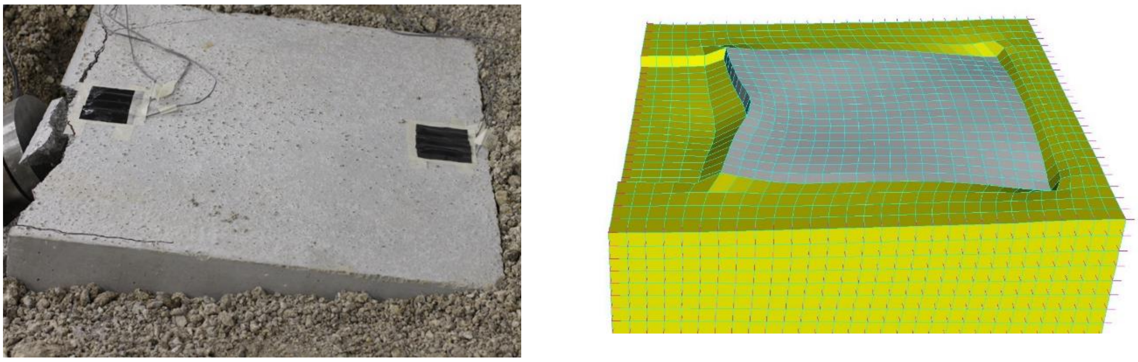

Figure 25. Visual comparison of vertical displacement between experiment and numerical models (10% displacement scale).

5.1 Future Recommendations

Several investigations using a survey instrument, experimental program and numerical simulation were undertaken within this research. Outputs from each phase provided variable levels of qualitative agreeance; however, the outcomes illustrate several different potential failure modes (structural failure, displacement, and hydraulic failure). Further work into minimising these discrepancies and improving agreeance is suggested. In terms of the experimental program, improvements to the scaled model dimensions should be made to alleviate the challenges experienced within the casting and demoulding phase of such a thin structure. Furthermore, laboratory experiment work using a soil material in an undrained state, should be investigated, resembling the real life scenario when debris flows with water. This will result in additional hydrostatic stress (pore pressure) as opposed to just considering the stresses within the soil skeleton (drained state, that is, fluid is free to flow).

6. CONCLUSION

This study undertook a qualitative comparison of floodways subjected to a centrally placed horizontal load through an experiment and numerical simulation, which was supported by the feedback from an industry-based survey. This type of loading can be considered as an accidental loading and was identified within the survey as occurring during extreme flood events when floating debris, such as logs, and boulders impact the floodway structure. Furthermore, the responses received through the industry survey confirmed that floodways were significantly more susceptible to failure during extreme flood events, particularly if the structure is raised or situated on dispersive soils, or sandy soils that lack cohesion. Qualitatively, the crack propagation within the experiment test specimens provided close correlation with the significant strain localisations identified within the numerical simulation results. Further, the vertical displacement tendency aligned closely across both the experiment and the numerical simulation results and illustrated the significant overturning moment present from a centrally placed horizontal load.

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CHAPTER 4

STRUCTURAL DESIGN METHODOLOGY FOR FLOODWAYS

Publication III - Structural Design of Floodways Under Extreme Flood Loading

Greene I, Lokuge W & Karunasena W 2020, "Structural Design of Floodways Under Extreme Flood Loading", *International Journal of Disaster Resilience in the Built Environment*, Vol. 11, No. 4, pp. 535-555. https://doi.org/10.1108/IJDRBE-10-2019-0072. Published - 9 May 2020 (Q3).

This chapter contains an extract of the above published manuscript. Please refer to Appendix B for copyright permission.

Authorship Contribution Statement

The contribution of Isaac Greene (Candidate) was 75%. Isaac undertook numerical model development, verification, result analysis and interpretation, drafting, revising and finalisation of the manuscript. Assoc. Prof. Weena Lokuge (Principal Supervisor) and Prof. Warna Karunasena (Supervisor) contributed to the formalisation of ideas, technical input and editing/co-authoring of the manuscript. These contributions were 15 and 10% respectively.

Linking Manuscript to Research Outcomes

This research article investigates floodway vulnerabilities through numerical finite element modeling. This is in response to the discovery that structural damage was a principal failure mechanism during historic flood events and since current floodway design guidelines focused primarily on hydraulic design principals and did not cover structural design aspects.

Three dimensional finite element model development was heavily investigated, which included: model discretisation, constitutive models/equations, continuum element types and material properties. Verification was also explored using linear analytical solutions and through observation of deformation types and magnitude.

Initially, a parametric analysis was undertaken using three dimensional finite element modeling to identify the current environment, external loading and design factors that cause floodways to be most susceptible to failure during extreme flood events. This work built upon the perspectives gained through the industry survey. A key outcome of the parametric analysis was the ability to quantify the geometry and loading case that resulted in the worst case loading scenario. The defined worst case loading scenario was then utilised to derive design improvements and a structural design methodology for inclusion in the design of floodways. The structural design method includes several design charts that cover a range of different soil types and allows the bending moment and shear force to be determined for a known flow depth and velocity.

Suggestions of a preliminary integrated design methodology that integrates the traditional hydraulic design principles with the structural design charts was then deduced. This formed the initial basis for revision of the current design guidelines.

Structural design of floodways under extreme flood loading

Isaac Greene
Department of Civil Engineering and Surveying, University of Southern Queensland, Toowoomba, Australia

Weena Lokuge
Department of Civil Engineering, Faculty of Health Engineering and Sciences, University of Southern Queensland, Springfield, Australia, and

Warna Karunasena
Department of Civil Engineering and Surveying, University of Southern Queensland – Springfield Campus, Springfield Central, Australia

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Abstract

Purpose – Current methods for floodway design are predominately based on hydrological and hydraulic design principles. The purpose of this paper is to investigate a finite element methods approach for the inclusion of a simplified structural design method into floodway design procedures.

Design/methodology/approach – This research uses a three-dimensional finite element method to investigate numerically the different parameters, geometric configurations and loading combinations which cause floodway vulnerability during extreme flood events. The worst-case loading scenario is then used as the basis for design from which several structural design charts are deduced. These charts enable design bending moments and shear forces to be extracted and the cross-sectional area of steel and concrete to be designed in accordance with the relevant design codes for strength, serviceability and durability.

Findings – It was discovered that the analysed floodway structure is most vulnerable when impacted by a 4-tonne boulder, a 900 mm cut-off wall depth and with no downstream rock protection. Design charts were created, forming a simplified structural design process to strengthen the current hydraulic design approach provided in current floodway design guidelines. This developed procedure is demonstrated through application with an example floodway structural design.

Originality/value – The deduced structural design process will ensure floodway structures have adequate structural resilience, aiding in reduced maintenance and periods of unserviceability in the wake of extreme flood events.

Keywords Floods, Floodways, Infrastructure, Bridge failure, Resilience, Impact load

Paper type Research paper

1. Introduction

Resilient road networks are essential for the safety and wealth of communities worldwide (Pregnolato *et al.*, 2017). Floodways (Plate 1) can be described as road infrastructure used in

The authors would like to acknowledge the Lockyer Valley Regional Council for providing information relating to floodways and the support of the Commonwealth of Australia through the Cooperative Research Centre program; *Bushfire and Natural Hazard CRC* and the *Research Training Program (RTP) Scholarship*. This comes as part of a bigger project on "Enhancing the resilience of critical road infrastructure: brides, culverts and floodways" funded by Bush Fire and Natural Hazards CRC Ltd, Australia.

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road design to facilitate the safe crossing of water courses during low flow flood events (Wahalathantri *et al.*, 2018). During such events the floodway structure is designed to be overtopped in a controlled and uniform manner dissipating flow concentration, thus reducing the likelihood of downstream scour and erosion. If floodwaters increase above safe crossing limits of 300 mm then vehicular access is precluded (Main Roads Western Australia, 2006). The incorporation of floodways offers a cost-effective solution when considering the cost benefit for use in rural road networks that do not service sufficient people to warrant large and expensive structures, such as culverts and bridges (Lumor *et al.*, 2017).

The frequency of flood events has increased, jeopardizing the resilience of the built environment (Kimura *et al.*, 2017). Flood-related hydrological disasters have the highest occurrence rate of all-natural disasters worldwide causing significant economic loss (Du *et al.*, 2019). Road structures located within waterways, such as floodways, culverts and bridges are assets that frequently sustain damage and/or catastrophic failure as a direct result of increased flood waters (BNHCRC, 2015). These road structures serve a vital role in post disaster recovery and need to be designed in a manner that allows them to remain open and serviceable both during and after extreme flood events (Hung and Yau, 2014). To achieve a flood-resilient design, time of exposure to flood events and/or minimizing vulnerabilities through incorporating proper structural mitigation measures is crucial (Chowdhooree and Islam, 2018).

Australia relies heavily on floodway structures to service its vast rural road network. Studies into numerous regional councils throughout Australia have reported repeat structural damage and consistent failure mechanisms. These studies include GHD (2012) who investigated damage to floodways in five different South Australian regional councils, Wahalathantri *et al.* (2015) who investigated damaged structures in Southern Queensland (QLD) and the authors of this paper who visited damaged floodway sites in Lockyer Valley Regional Council and Albury City Council as shown in Plates 1a and b.

In January 2011, the State of QLD experienced *widespread* flooding which again repeated in 2013. These flood events caused rapid run-off as a result of heavy rainfall inundating 62 per cent of the state of QLD and caused a reported $234m in damage to the built environment (Setunge *et al.*, 2014). More specifically, the Lockyer Valley region in QLD, the focus of this research exceeded an average exceedance probability of 1 in 200, took 19 lives and out of 330 floodways in the region, 77 catastrophically failed and 115 sustained direct damage (Wahalathantri *et al.*, 2015). These flood events highlighted the importance of the floodway design process and the need to investigate the critical design parameters, failure mechanisms and integrity of floodway structures to enhance the resilience of the rural road network (Wahalathantri *et al.*, 2018).

Plate 1.
(a) Floodway structure incorporated in a rural road and (b) scour in the immediate downstream rock protection zone

A team of researchers (BNHCRC, 2015; Wahalathantri *et al.*, 2015) undertook research into the most common failure modes of damaged floodways in the Lockyer Valley Region. This research discovered that the most common mode of failure was caused by direct impact from floating debris, such as logs and boulders being conveyed by the increased flood water. In addition, erosion was present at many of the damaged floodway sites indicated by scouring in the immediate upstream and downstream rock protection zones (Plate 1b). In some severe cases undermining of the floodway superstructure was also present. GHD (2012) reported that the presence of downstream erosion at a floodway site, if left unrepaired has the potential to form an erosion head cut. This erosion head cut can move upstream at variable rates based on the creek beds strata characteristics and may result in structural failure if it contacts the floodway.

As a direct result of the widespread damage sustained by floodways in 2011, the Lockyer Valley Regional Council investigated and implemented several revisions to the geometric features used within their standard engineering floodway designs. These revisions are summarised as follows:

- Inclusion of a cut-off wall to the entire perimeter of the floodway. Initially, Lockyer Valley Regional Council constructed floodways with cut-off walls at the upstream and downstream extents only; however, in recent years they have begun including the cut-off wall to the entire floodway perimeter to prevent ground water flowing through the underlying granular road pavement. This arrangement also provides further assurance against undermining of the superstructure.
- Trialing cut-off wall depths, defined as treatment options by Lockyer Valley Regional Council, of 900 and 1100 mm as shown in Figure 1. Lockyer Valley Regional Council currently selects cut-off wall depth based on the proposed sites average stream velocity. It is anticipated that the cut-off wall will provide increased lateral resistance as a result of the greater surface area present. Further, the increased cut-off wall depth will provide greater structural resilience if a downstream erosion head cut contacts the floodway superstructure.

Current design guidelines *for* floodway design *are* based predominately upon hydrological and hydraulic design principles [Queensland Department of Transport and Main Roads, 2010; Main Roads Western Australia, 2006; Austroads Ltd, 2013; US Army Corps of Engineers Afghanistan Engineer District, 2009; Lohnes *et al.*, 2001]. *As a result of this, current design practices tend to neglect the forces that floodways are exposed to, yet expected to withstand, during extreme flood events. These hydrological and hydraulic design principles use the Empirical Broad-Crested Weir formula and Mannings formula for raised and flush floodway structures, respectively.* These formulas contain a number of assumptions including; the water course is of an open prismatic and uniform channel and *through the application of a mean velocity in Manning's equation inaccuracies are introduced as often the*

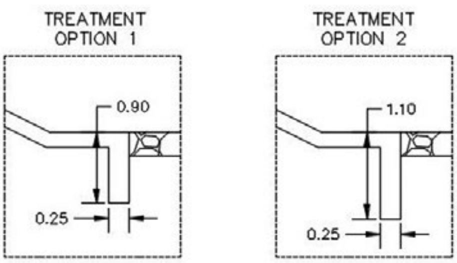

Figure 1. Schematics of treatment options applied to floodways in the Lockyer Valley Regional Council

velocity is lower at the deeper sections within a creek and subsequently higher at the edges and mid-section, which results in higher stresses (BNHCRC, 2015). The studies conducted by Wahalathantri *et al.* (2018) also concluded that floodways often exist in complex surroundings that include horizontal and vertical bends. Due to these complexities, assumptions *used* by the current *hydraulic focused* design guidelines *often create outcomes which are less than optimal.*

2. Research significance

This research, through the undertaking of a finite element methods approach expands on the authors conference paper titled "Floodway Design Process Revisited" (Greene *et al.*, 2019) to deduce what causes a standard engineering floodway type to be most vulnerable while acting in a frequently submerged state. Using this state of vulnerability as the basis of design, ultimate design bending moments and shear forces can be determined and structural design charts deduced. These design charts enable design bending moments and shear forces to be extracted *and* steel reinforcement *in* concrete to be designed in accordance with the relevant design codes for strength, serviceability and durability. This simplified structural design method incorporated with the current design guidelines will enable local government authorities to design resilient floodway structures with confidence.

3. Methodology

Three-dimensional (3D) finite element modelling and subsequent analysis were conducted using finite element computational software, Strand7 (Strand7, 2019). Finite element methods is widely adopted and *used* in a variety of numerical modelling applications and structural problem solving. Model development methods and test variables used to construct the 3D floodway model are outlined in this section.

"Concrete Floodway Type-2", a commonly implemented standard engineering floodway type from the Lockyer Valley Region in QLD was selected for modelling as shown in Figure 2. The Type 2 floodway is often implemented in creeks of relatively flat grade and where a hydraulic control is not required to be imparted on the creek to facilitate safe vehicular crossing conditions.

3.1 Model development

Details of modelling techniques, criterions and selected parameters used in this research are summarised as follows:

- *Element types and criterions:* Four node tetrahedra Strand7 brick elements were used to construct the 3D model. Mohr–Coulomb yield criterion, a commonly implemented failure model for geotechnical materials was used to analyse the non-linear behaviour of soil materials where behaviour is governed by cohesion and internal friction angle (Jiang, 2018). Max stress yield criterion was assigned to concrete brick elements and defines failure when stress components exceed yield strength in either compression or tension (Feng *et al.*, 2019). As this criterion is stress dependent a stress versus strain curve was defined in Strand7 to represent the nonlinear elastic behaviour of concrete in both compression and tension.
- *Boundary conditions:* Boundary conditions were assigned to the outer model extents to imitate *in situ* support conditions of a floodway situated in infinite length and depth of natural adjoining strata. That is, the outer faces were assigned roller support conditions and the bottom face rigid support conditions. Roller supports permitted movement in the vertical axis yet precluded movement in the horizontal

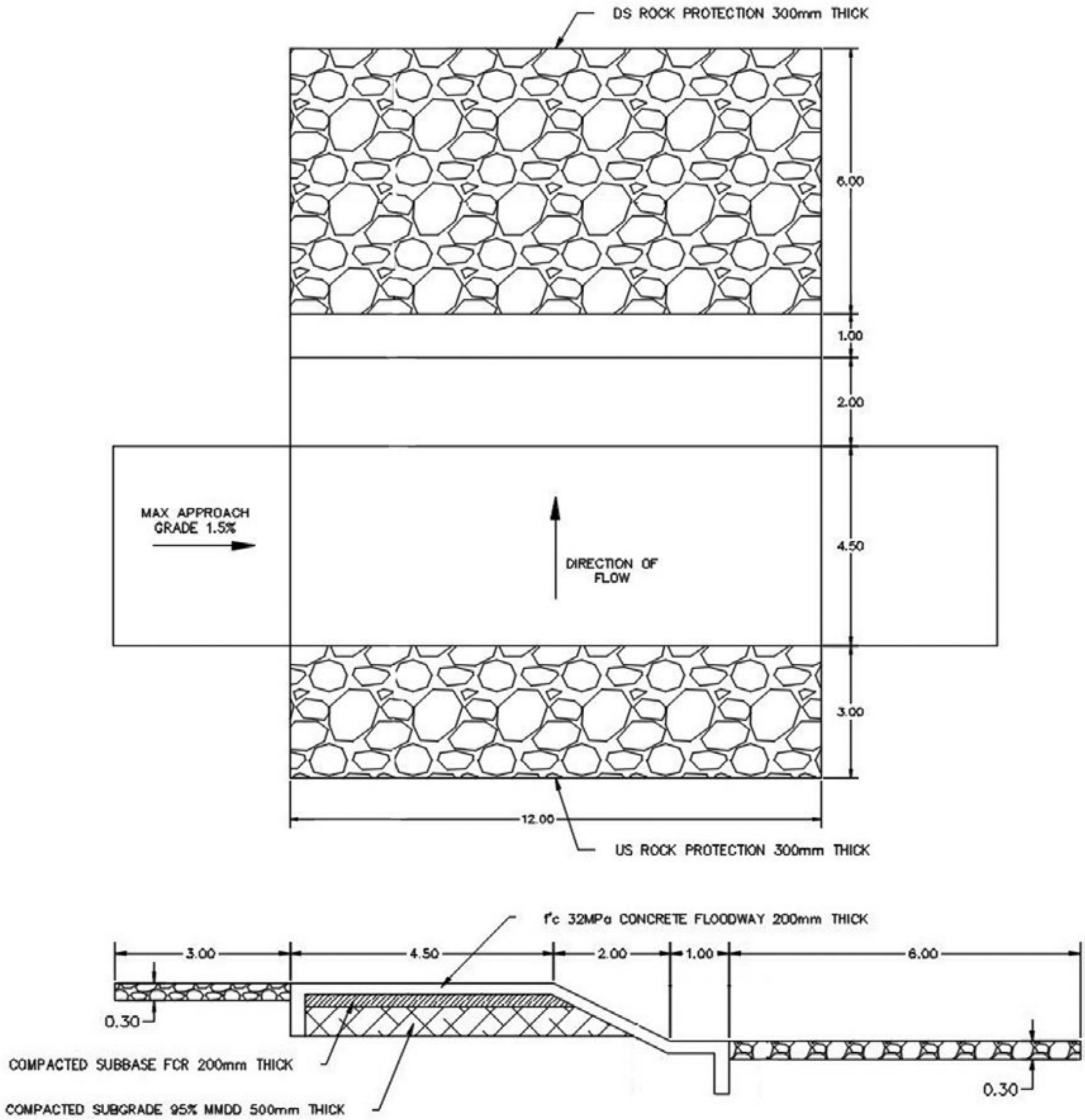

Figure 2.
Plan and cross-section of the Type 2 floodway

axis (normal to the vertical plane). Rigid supports are fixed against displacement and rotation.

- *Mesh and model refinement:* To determine the influence of the restraints and mesh size the extent of adjoining natural earth and mesh density was iteratively increased until a converged numerical solution resulted. This was found to be achieved by a model consisting of 26,244 nodes and 23,584 brick elements with a profile length of 26.4 m, width of 21.5 m and a depth of 20.9 m as shown in Figure 3. It should be noted that for the model depth to converge, Strand7 tool *"Auto Assign Insitu Stress"* was used.
- *Water level:* To model the change in water level for soil materials, fluid level was set in respect to the coordinate value of the global axis corresponding to the gravity

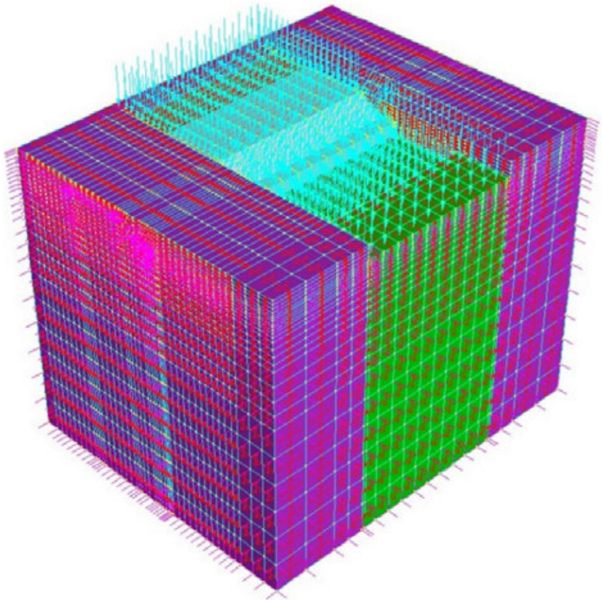

Figure 3.
Constitutive floodway model with medium mesh size

direction. This required the definition of multiple element property types for each water level, along with the approximation of the actual water level profile into a series of constant water levels for advanced and/or irregular shaped elements. On completion, *in situ* stresses were calculated and the vertical *in situ* stress profile under self-weight was observed for any variance.

- *Concrete-soil interface:* It was concluded that omitting contact interface was satisfactory for flow velocities less than or equal to 8 m/s as loading was calculated to be well below the maximum frictional force, refer sub-Section 2.2 for further details.
- *Mechanical properties:* Material mechanical properties assigned to the model are defined in Table 1. Rock protection was assumed to be made up of individual loose packed rocks (lower modulus and density) which behave as a soil material defined by Mohr–Coulomb criterion, i.e. a homogeneous, elastic–plastic and isotropic material, a criterion frequently used in practice to model geotechnical material failure (Jiang, 2018). Steel reinforcement in concrete was neglected, allowing tensile forces apparent to be determined and reinforcement designed accordingly in the structural design method presented.

Table 1.
Mechanical properties of materials used to define the floodway

Material	Modulus (MPa)	Poisson ratio	Density (kg/m^3)	Cohesion (MPa)	Friction angle (degrees)
Concrete	31,000	0.2	2,400	N/A	N/A
Rock (Obrzud and Truty, 2018)	100	0.3	1,400	1.0	30
Natural subgrade (95% MDD)	150	0.3	1,900	0.1	30
Gravel sub base	200	0.3	2,000	0.1	35
Soil 1: Silty sand	40	0.3	1,700	0.01	25
Soil 2: Sandy soil	30	0.25	1,800	0.075	34
Soil 3: Clay soil	100	0.3	1,900	0.01	20

3.2 Verification

Due to the complexity of the full-size model and the use of a nonlinear analysis, a verification model representing a component of the augmented model was used to provide model confidence. This verification model was used to validate modelling techniques, material criterion, convergence controls and model response, with errors identified, analysed and omitted.

- *Elastic response:* Hooke's law is a common engineering model used to approximate a material stress–strain relationship assuming elastic properties where force and displacement are proportional (Johnson, 2006). This relationship can be represented by equation 1 and was used to resolve vertical displacement for each of the layered elastic materials in the verification model. By comparing total elastic displacement calculated by Hooke's law to that of the linear static solver output, discrepancies in the model's response could be determined.

$$\delta = \frac{PL}{EA} \tag{1}$$

where δ is vertical displacement in [m], P is the applied force in [N], L is length in [m], E is the Modulus of Elasticity in [MPa] and A is the area in [m^2].

- *Visual response:* Visual inspection of the magnitude and shape of deformation were checked to ensure uniformity and that realistic results were being obtained for each verification case considered.
- *Mohr–Coulomb response with concrete–soil interface:* The vertical cross-section of the floodway consisted of layered materials with varying frictional interfaces. The most critical interface existed between the concrete apron and the compacted gravel sub-base. The effect of contact was analysed using the Coulomb friction/elliptical plastic model after inducing a small gap between the layers and linking the two regular meshes so that they are in immediate contact. Further, contact behaves non-linearly and so the nonlinear static solver within Strand7 was selected. To assist in obtaining a converged result, load stepping was used in the nonlinear static solver to incrementally apply loading and to prevent over-penetration during the initial load application.

Two friction coefficients, 0.55 and 0.99, were selected to represent a case with and without contact, respectively. The limiting load (maximum frictional force) was calculated using the static friction formula in equation (2) for the two friction coefficients.

$$F = \mu N \tag{2}$$

where F is force in [N], μ friction coefficient (unitless) and N is the normal force in [N].

For a friction coefficient of 0.99 and all loads well below the limiting load of 0.1049 MPa displacement results with contact remained very similar to the displacement results without contact.

Table 2 shows the results obtained for a friction coefficient of 0.55. Results for loadings well below the limiting load of 0.0582 MPa remained similar for both with and without contact. As the loading approached the limiting load, results began to diverge indicating that the concrete was on the verge of displacement. When the load exceeded the maximum frictional force, the solver could no longer converge indicating that the concrete had displaced.

It was concluded to omit contact elements since the loads being modelled remain well below the limiting load and therefore no significant discrepancy in results will occur.

3.3 Test variables and loading combinations

A range of variables were selected for use in the parametric analysis. The values used for these variables were consistent with those recorded during the 2011 flood event in the Lockyer Valley Region and the design considerations implemented by the Lockyer Valley Regional Council:

- flow depth intervals consisting of 0, 1 and 2 m above the road surface. For vehicular loading this was limited to 0.3 m corresponding to the maximum permissible crossing depth specified by Austroads Ltd (2013);
- upstream velocities of up to 8 m/s;
- varying boulder mass between 2 and 4 tonnes;
- varying cut-off wall depth between 900 and 1100 mm;
- varying adjacent soil types; and
- varying downstream rock protection extent between; full protection, no protection and no protection or soil adjacent the downstream cut-off wall. The latter simulating a downstream head cut contacting the floodway superstructure.

Three different loading combinations were selected for analysis as follows:

(1) Hydrostatic loading:
Hydrostatic pressure was assumed to behave in accordance with hydrostatic fluid force theory, that is, normal to the surface of the object and in a linear manner where the mediums density is directly proportional to height as shown in equation (3).

$$P = \rho g h \tag{3}$$

where P is hydrostatic pressure in [Pa], ρ is the mediums density in [kg/m^3], g is gravity in [m/s^2] and h is height of fluid in [m].

(2) Boulder impact and hydrostatic loading:
Boulder impact loading was calculated by applying a factor of 0.5 to the log impact formula provided in AS 5100.2:2017, *"Bridge Design, Design Loads"* (Standards Australia, 2017). The 0.5 factor was considered appropriate as boulders are not suspended articles like logs, rather they remain in contact with the creek bed.

$$F_{log} = 0.5\left(mV^2/2d\right) \tag{4}$$

where F_{log} is force in [N], m is the objects mass in [kg], V is the objects velocity in [m/s] and d is the stopping distance in [m].

Table 2.
Dx displacement versus load for with and without contact (μ = 0.55)

Horizontal load (MPa)	0.0197	0.0400	0.0575	0.0582	0.0590	0.0625
Max Dx (mm) without contact	0.720	0.830	0.940	0.945	0.951	0.970
Max Dx (mm) with contact	0.707	0.830	2.180	5.650	Not converged	Not converged

Velocities required to propagate the movement of a 4-tonne boulder were derived from Main Roads Western Australia (2006) tables for rock protection scour velocities.

(3) Vehicular, debris and hydrostatic loading:

For vehicular loading a maximum permissible crossing depth of 0.3 m was considered as specified by Austroads Ltd (2013).

Traffic loads were applied in a static state and approximated the effects induced by moving traffic and stationary queues in accordance with AS5100.1 (Standards Australia, 2017). Due to the rural setting, a W80 wheel load corresponding to an 80 kN load uniformly distributed over an area of 400 250 mm was considered appropriate.

Debris loading was calculated using equation (5) based on AS 5100.2:2017 (Standards Australia, 2017).

$$F_{deb} = 0.5 C_d {V_u}^2 A_{deb} \tag{5}$$

where C_d is the coefficient of debris.

Drag force was omitted based on the negligible effect (Cummings, 2015). A load factor of 1.3 was applied to all design loads from AS5100.2:2017 to satisfy ultimate limit state (ULS) conditions (Standards Australia, 2017). Hydrostatic force being an exception which had a load factor of 1.5 applied based on AS1170.1 (Standards Australia, 2002).

4. Results and discussion

4.1 Simulation of loading combinations

4.1.1 Load combination A – hydrostatic loading. Combination A considers the floodway, situated level with the creek bed and with no surface area acting perpendicular to the direction of flow. Lateral loadings that increase proportionally to velocity, such as debris and impact, are therefore omitted; however, a range of hydrostatic loads are applicable. This case was selected in response to the findings in GHD (2012), which reported that road authorities were removing damaged floodway structures and vertically realigning them flush with the creek bed after observing minimum damage post flood events.

Figure 4(a) shows vertical displacement increased downwards linearly as flow depth increased. This resulted since hydrostatic loading is proportional to flow depth. Similarly, Von Mises stress also increased proportionally to flow depth (Figure 4b). The maximum vertical displacement and Von Mises stress of 2.15 mm and 1.04 MPa, respectively, occurred at a flow depth of 2 m.

4.1.2 Load combination B – boulder impact and hydrostatic loading. Combination B considered a 2-tonne boulder impacting the floodway. In this combination, the floodway was modelled in a damaged state with reduced upstream rock protection causing the upstream leading edge to protrude and act perpendicular to the direction of flow.

Displacement in the direction of flow increased proportionally to flow velocity corresponding to an increase in impact loading (Figure 5a). Horizontal displacement decreased as flow depth increased due to the increase in frictional forces present. The highest horizontal displacement was 0.908 mm which occurred at 1 m and 8 m/s flow depth and velocity, respectively.

Flow depth influenced the resulting Von Mises stress up until approximately 4 m/s, this trend was more evident as flow depth increased as illustrated in Figure 5(b) by the change in direction at approximately 4 m/s for the 2 m flow depth. Once flow velocity increased past

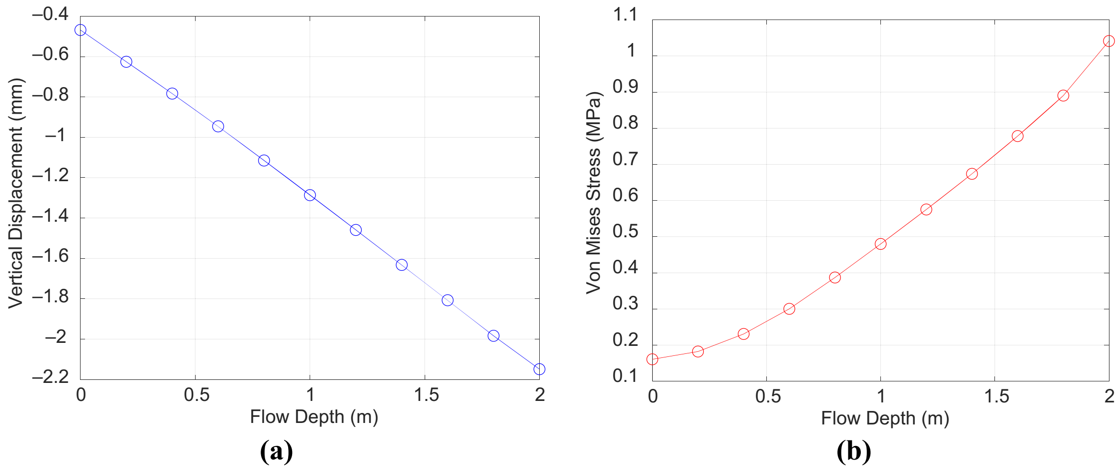

Figure 4.
(a) Vertical displacement and (b) Von Mises stress for changes in flow depth

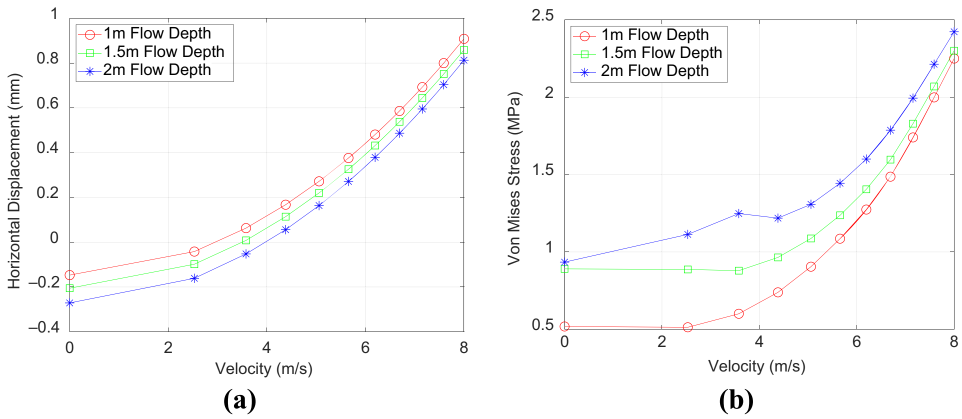

Figure 5.
(a) Horizontal displacement and (b) Von Mises stress for boulder impact

4 m/s, impact loading was the most dominate force resulting in the three flow depths increasing exponentially with flow velocity. The highest stress of 2.423 MPa occurred when flow velocity and flow depth were both at a maximum corresponding to the largest impact force.

4.1.3 Load combination C – vehicular, debris and hydrostatic loading. Combination C also considered the floodway type in a damaged state and therefore conducive to lateral loads, such as debris loading. In addition, vertical vehicular and hydrostatic loading were applied up to the maximum permissible crossing depth of 0.3 m (Austroads Ltd, 2013). Traffic loads were applied as a static loading which approximated the effects induced by moving traffic and stationary queues in accordance with AS5100.1 (Standards Australia, 2017).

Displacement in the horizontal direction (direction of flow) increased proportionally to flow velocity which corresponded to an increase in debris loading (Figure 6a). Horizontal displacement also decreased as hydrostatic loading increased due to an increase in frictional force. The highest horizontal displacement of 0.5 mm occurred when flow depth was at a minimum and flow velocity was at a maximum.

Von Mises stress increased exponentially due to debris accumulation being proportional to both flow velocity and depth (Figure 6b). The highest stress of 1.93 MPa occurred when flow velocity and depth were at a maximum.

4.1.4 Discussion. Load Combination A produced the lowest stress and displacement results for the three loading combinations considered. This combination remains applicable if no surface acts perpendicular to the watercourse, resulting in a very efficient floodway design where stress and vertical displacement results are directly proportional to flow depth. This further supports GHD (2012) who observed road authorities removing damaged floodway structures and realigning them flush with the creek bed.

Reviewing the three loading combinations, Combination B consistently produced the highest stress and displacement results and was therefore adopted for further detailed parametric analysis of design parameters.

4.2 Simulation of varying design parameters

Different design features, geometry and load configurations consistent with that currently being implemented by the Lockyer Valley Regional Council were investigated. These parameters included impact loading magnitude, cut-off wall depth, extent of downstream rock protection and different adjoining soil types.

4.2.1 Boulder mass. Boulder mass was increased from 2 to 4- tonnes, a load equivalent in magnitude to a 2- tonne floating log as considered in AS 5100.2:2017 (Standards Australia, 2017). As flow velocity increased, horizontal displacement in the 4-tonne boulder case diverged and was 61.07 per cent greater than the 2-tonne boulder when flow velocity was at a maximum (Figure 7a). Von Mises stresses also followed a similar diverging trend with stresses 52.98 per cent greater than the 2-tonne boulder case when flow velocity was at maximum (Figure 7b).

4.2.2 Cut-off wall configuration. Investigation into different length cut-off walls defined as “treatment options” provided comparison between a 1100 mm cut-off wall and the commonly used 900 mm cut-off wall depth. For the 1100 mm cut-off wall depth both maximum horizontal and vertical deflection were reduced by 1.06 and 3.94 per cent, respectively (Figure 8a). This reduction in deflection is a result of the greater distribution of forces to the adjoining soil due to the increased surface area, subsequently increasing stabilising moment. Similarly, Von Mises stresses slightly decreased by 0.781 per cent (Figure 8b).

4.2.3 Downstream rock protection. Erosion of downstream rock protection was modelled by incrementally reducing the depth of rock protection and soil extent adjacent the downstream cut-off wall. This was conducted over three increments which included; full downstream rock protection, no downstream rock protection and no downstream rock protection or natural soil adjacent the downstream cut-off wall.

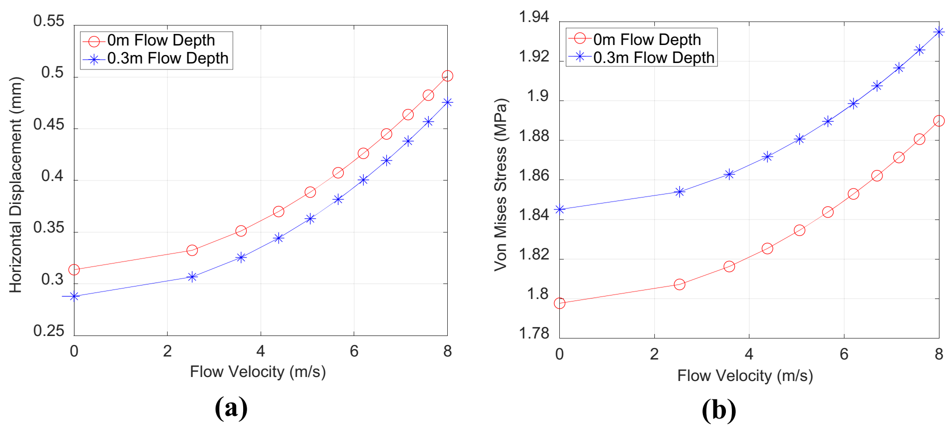

Figure 6.
(a) Horizontal displacement and (b) Von Mises stress for debris, vehicular and hydrostatic loadings

For velocities below approximately 7.5 m/s horizontal displacement decreased as material adjacent to the cut-off wall decreased (eroded). This subsequently caused water depth to increase creating *a larger* opposing hydrostatic force. *At the highest velocity of 8 m/s,* the case with the largest resistance to lateral loading *reversed to* full downstream rock protection which resulted in 5.50 per cent less horizontal deflection than the case with no downstream rock protection (Figure 9a). Von Mises stresses also converged at a flow velocity of approximately 3.5 m/s when impact loading became the most dominant force, after which a consistent increasing trend resulted (Figure 9b).

4.2.4 Varying soil types. Three different soil properties (Table 3) were selected to reflect the different strata which floodways are frequently constructed within. *Soil 2*, a sandy soil resulted in the highest stress and displacement results out of all the soil types. Changing soil types was found to have a large influence on the variability of displacement and stress results. It was therefore decided to consider all soil types in the structural design procedure. This allows the structural design method to align with a range of *in situ* soil conditions specific to the floodway site locality.

4.2.5 Worst-case loading scenario. The worst-case loading scenario occurs when the loading combination is at its most unfavourable and the design parameters that compromise floodway integrity are at their greatest. This state occurred when flow depth and flow

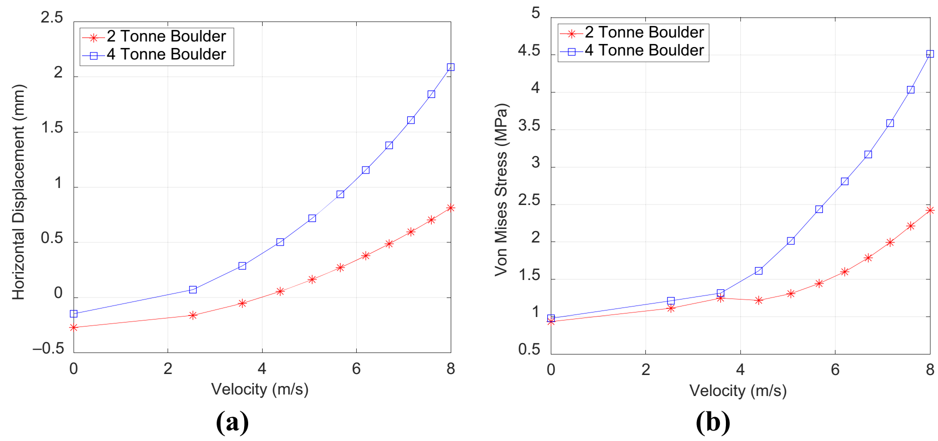

Figure 7.
(a) Horizontal displacement and (b) Von Mises stress for 2- and 4- tonne boulder impacts

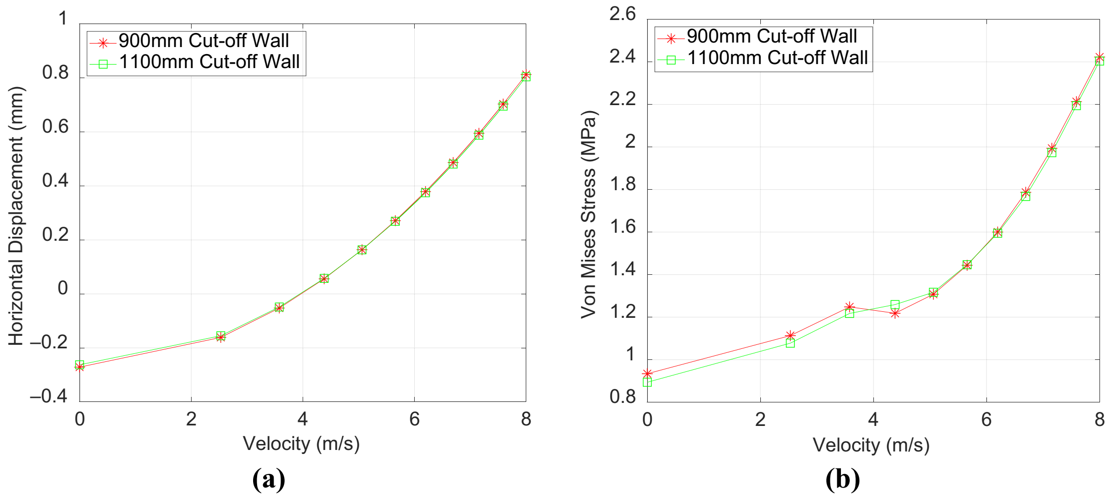

Figure 8.
(a) Horizontal displacement and (b) Von Mises stress for different cut-off wall configurations

velocity were at a maximum and for a 4-tonne boulder impact, no downstream rock protection and a 900 mm cut-off wall depth. This worst-case loading scenario formed the basis of the deduced structural design process detailed in subsequent sections. Mechanical properties of soil adjacent the floodway structure *were* considered independently.

4.3 Determining design bending moments and shear forces

To extract bending moment and shear force from the worst-case loading scenario the following process was undertaken:

- The ULS loading (1.2G + 1.5Q) AS1170 representing the worst-case loading scenario was applied to the 3D floodway model and solved (Australian Standards AS1170: 2002).
- Observation of the results highlighted areas containing the largest stress and displacement results. This was discovered to occur centrally and in the direction of flow (longitudinal). This was denoted as the line-of-action.
- Along the line-of-action, displacements were recorded either side of the floodway.
- The line-of-action was then reproduced by applying the recorded displacements to the nodes in a separate model containing a two-dimensional (2D) beam connected rigidly at joints. This 2D beam was of a nominal 1 m length in the *z*-direction and the cross section represented that of the 3D floodway model.
- Solving the 2D beam model in Strand7 produced the resulting bending moments (M*) and shear force (V*) distribution. To confirm solution accuracy the Dx and Dy displacements from the 2D beam model were compared to that of the 3D cut-plane model.

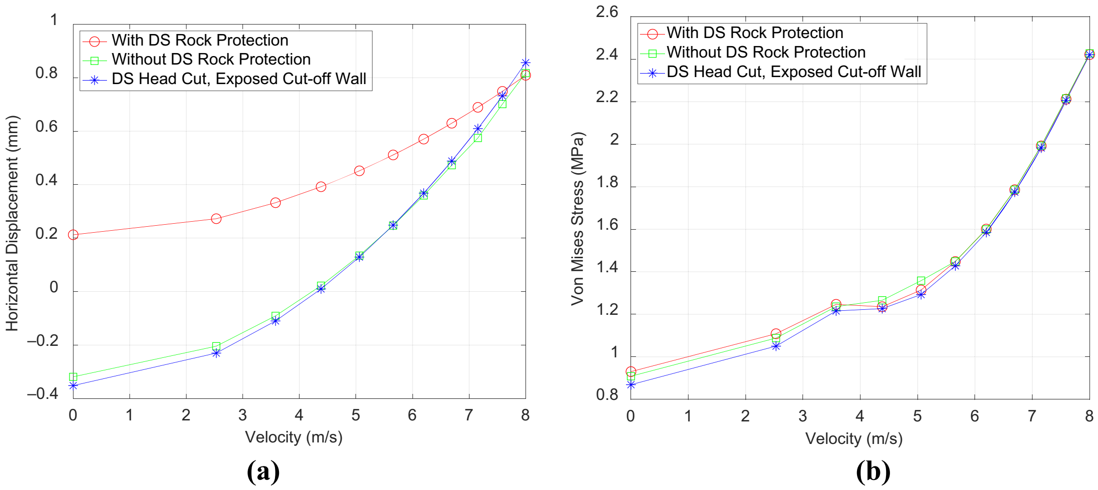

Figure 9. (a) Horizontal displacement and (b) Von Mises stress for different downstream rock protection extents

Table 3. Mechanical properties for *in situ* adjoining soils

Material type	E (MPa)	ν	ρ (kg/m^3)	c′ (MPa)	ϕ (°)	K0	e
Soil 1: Silty sand	40	0.3	1,700	0.01	25	0.426	0.4
Soil 2: Sandy soil	30	0.25	1,800	0.075	34	0.44	0.3
Soil 3: Clay soil	100	0.3	1,900	0.01	20	0.658	0.15

- These steps were repeated for the three reported commonly encountered soil types and the maximum M* and were plotted as strength capacity charts for a range of different flow velocities and flow depths.

It was discovered that both positive and negative bending moments and shear forces act against the floodway superstructure (Figure 10). Further, these maximum positive and negative moments and shear forces are concentrated at the upstream and downstream cut-off wall and apron locations. These locations were subsequently selected for the development of strength capacity design charts representing the absolute moment and force acting on the structure.

4.4 Strength capacity design charts

This section presents a simplified structural design method for floodways based on the design bending moment and design shear force values deduced from the parametric finite element analysis. These values have been assembled into strength capacity design charts and are presented in Figures 11-14. The design bending moment and shear force are absolute values for the floodway structure and are based on three commonly encountered soil types from the Lockyer Valley Region and for a range of different flow velocities and

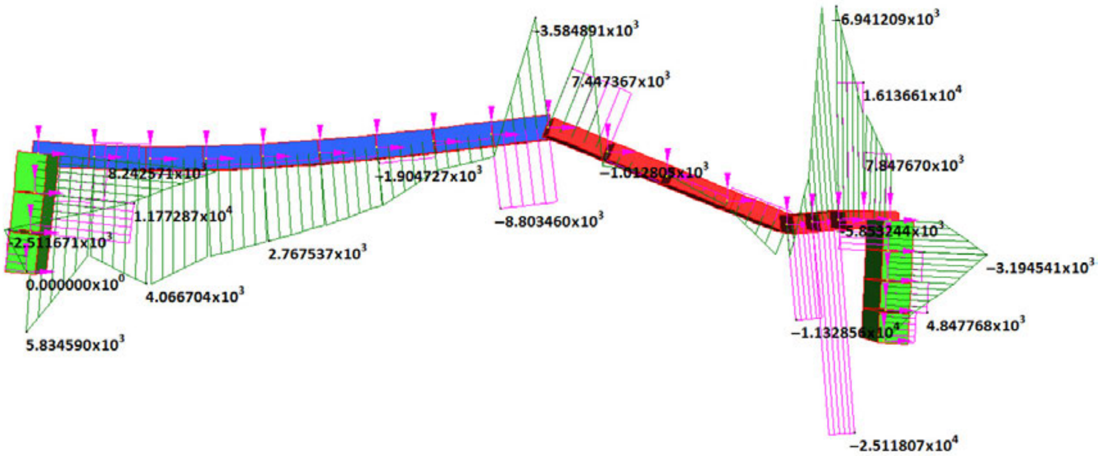

Figure 10.
Typical 2D beam bending moment and shear force diagram.

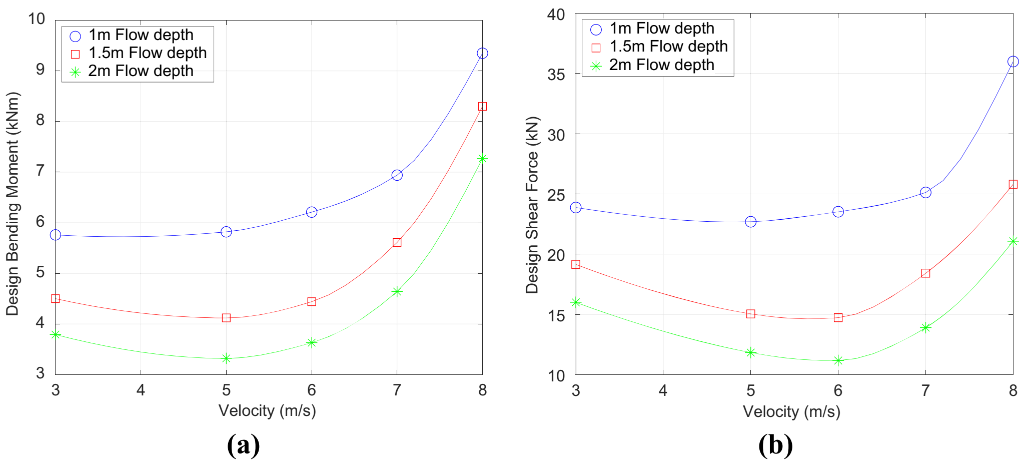

Figure 11.
(a) Design bending moment and (b) design shear force design charts for soil type 1

depths. Design of the floodway structural elements can then be conducted in reference to the relevant standards for concrete construction.

Investigating the downwards trend in the shear force design graphs it was found that hydrostatic loading (predominately a negative vertical load) and impact loading (positive horizontal load only) resulted in opposing load types and the trough of the design graph corresponded to the point at which they negated each other the greatest.

4.5 Structural design example

The following example illustrates the use of the strength capacity design charts to determine the reinforcement requirements of the floodway type. The aim of this design is to select the cross-sectional area of steel and concrete that satisfies the relevant countries code requirements for strength, serviceability and durability. For this design example, design will be in accordance with the requirements of Australian Standard AS3600:2009 (Standards Australia, 2009).

4.5.1 Step 1: Determine design parameters. Design parameters include the specific site location, soil type, maximum flow velocity and depth. These factors are determined in conjunction with current floodway design guidelines, geotechnical testing and flood modelling software. For this example, the following parameters are assumed;

- location: Lockyer Valley Regional Council (temperate environment);
- soil type: soil Type 2;
- maximum flow velocity: 7 m/s; and
- maximum flow depth: 1.5 m.

Referencing the design charts for soil Type 2 as presented in Figure 12, the maximum bending moment and shear force can be extracted as 8.47 kN.m and 30.52 kN, respectively. Note linear interpolation can be used to determine intermediate values.

4.5.2 Step 2: design for durability – Section 4, AS3600:2009.

- exposure classification is B1;
- minimum compressive strength of 32 MPa satisfies durability requirements; and
- minimum cover required is 40 mm, assuming standard formwork and compaction.

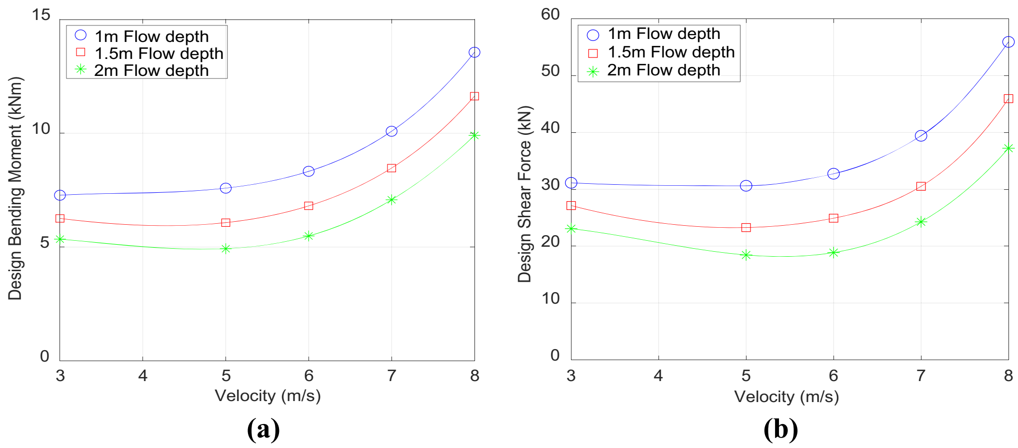

Figure 12. (a) Design bending moment and (b) design shear force design charts for soil type 2

4.5.3 Step 3: design for strength and serviceability in bending – Section 8.1, AS3600:2009. Assume the use of SL81 mesh to satisfy bending reinforcement requirements (Figure 14).

$M^* = 8.47\ kN.m$, where M^* = design bending moment from design graphs

Calculating compressive force in concrete, F_c i.e. volume of the stress block;

$$\begin{aligned} F_c &= (\alpha_2 f'_c)(\gamma d_n)b \\ F_c &= (0.85)(32)(0.826)(d_n)(1000) \\ F_c &= 22467.2(d_n) \end{aligned} \tag{6}$$

Calculating tensile forces in steel (assuming steel yields), F_t;

$$\begin{aligned} F_t &= A_{st} f_{sy} \\ F_t &= (363)(500) \\ F_t &= 181,\ 500 \end{aligned} \tag{7}$$

Equating F_t and F_c to determine the neutral axis depth;

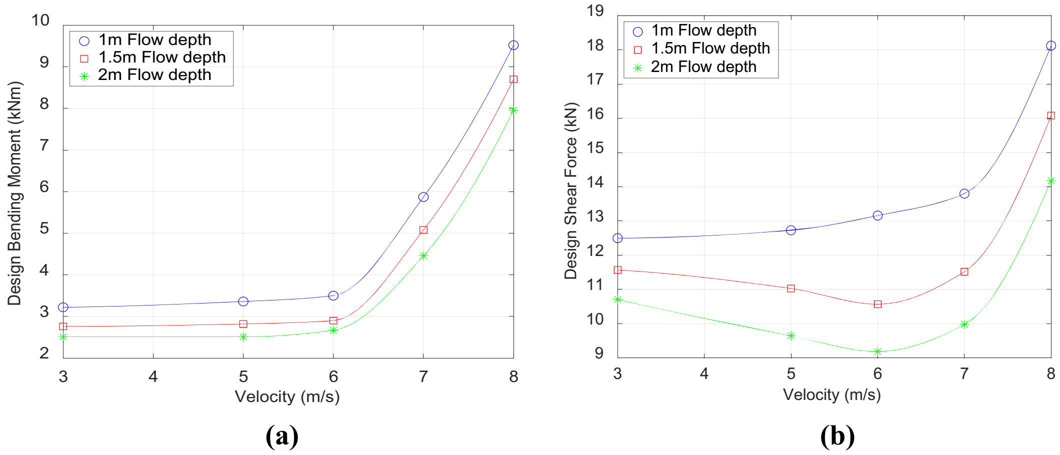

Figure 13.
(a) Design bending moment and (b) design shear force design charts for soil type 3

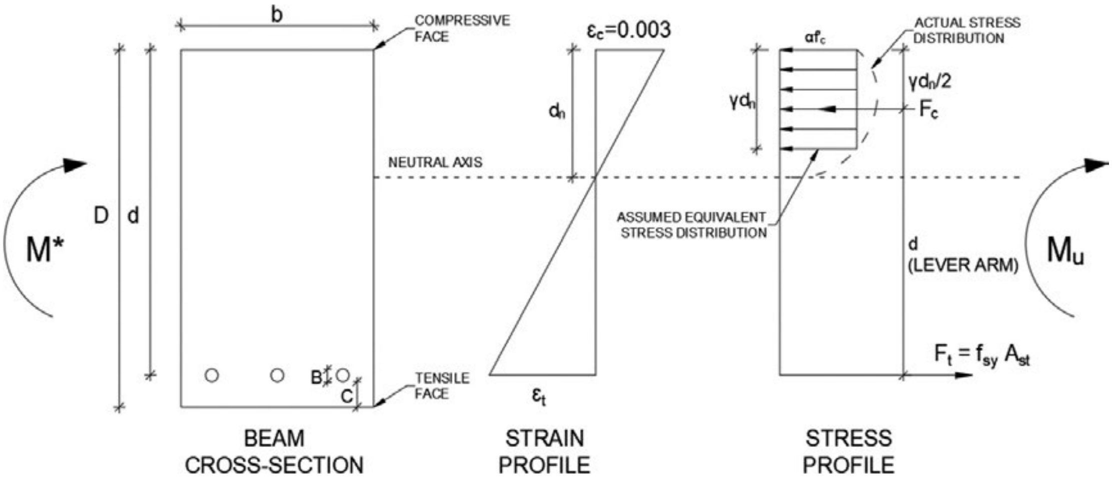

Figure 14.
Equivalent concrete stress block for cut-off wall

$$\begin{aligned} F_t &= F_c \\ d_n &= 8.078\,mm \end{aligned} \tag{8}$$

With d_n evaluated we can calculate the lever arm:

$$\begin{aligned} d &= D - c - B/2 - \gamma \cdot d_n/2 \\ d &= 250 - 40 - 8/2 - ((0.826)(8.078))/2 \\ d &= 202.66\,mm \end{aligned} \tag{9}$$

Calculating moment capacity, M_u, gives;

$$\begin{aligned} M_u &= F_t d \;=\; F_c d \\ M_u &= \left(181,500\,x\,10^1\right)(202.66) \\ M_u &= 36.8\,kN.m \end{aligned}$$

Checking if assumed bending reinforcement is satisfactory

$$\varphi M_u >= M^* \tag{11}$$

(0.85)(36.8) kNm $\geq$ 8.47 kN.m, therefore safe.

Outcome: Adopt SL81 mesh reinforcement to both the inner and outer faces of the floodway to satisfy bending moment.

4.5.4 Step 4: design for strength and serviceability in shear – section 8.2, AS3600:2009.

V^* = 30.52 kN, where V^* = design shear force at a cross section as determined from design charts.

Checking if shear reinforcement is required:

$$\begin{aligned} V_{uc} &= \beta_1 \beta_2 \beta_3 b_v d_o \left(A_{st} f'_c / b_v d_o\right)^{1/3} \\ V_{uc} &= (1.537)(1)(1)(1000)(202.66)\left((363)(32)/(1000)(202.66)\right)^{1/3} \\ V_{uc} &= 119.46\,kN \end{aligned} \tag{14}$$

$$\varphi V_{uc} \geq V^{**} \tag{15}$$

$0.5\varphi\, V_{uc}(0.5)(120.1\,kN) > V^*(30.52\,kN)$, therefore safe.

Outcome: Shear reinforcement is not required as the shear strength of 32 MPa concrete alone satisfies shear force requirements.

4.6 Integrated design procedure

The simplified structural design procedure is intended to be used in conjunction with the hydraulic design process detailed in the current design guidelines. The design procedure incorporating the structural design method is summarised in Figure 15 and outlined as follows:

- select the point of waterway crossing based on the horizontal and vertical road alignment criteria and environmental factors stated in the design guidelines;
- derive the stage-discharge curve for the water course;

- calculate the optimum reduced level for the floodway deck (road surface) along with the expected discharge rate. This needs to be less than 300 mm and for a 20-year ARI event. As the Type-2 floodway is situated level with the creek bed (i.e. does not impart a hydraulic control on the stream), design discharge can simply be calculated using Mannings equation;
- determine design bending moment and design shear force based on the values obtained from the design charts illustrated in Figures 11-13. These values are based on the soil type encountered at the specific location along with the flow velocity and depth of the water course. Design of the floodway structural elements can then be undertaken to satisfy the relevant countries code requirements.
- select appropriate scour, pavement and embankment protection in accordance with the current floodway design guidelines.

5. Implications for research and practice

This research incorporates a structural design process which addresses a gap in the current area of knowledge that focuses primarily on hydraulic design principles to deliver an improved and consistent design methodology. This structural design process considers the floodway in a submerged state and with external loadings equivalent to that experienced during an extreme flood event. As an outcome of this research, strength capacity charts containing design bending moment and shear force values for a single floodway type were derived providing designers with an accurate and expeditious method to determine the design forces apparent within the floodway structure under extreme flood loadings. Designers can then design structural elements in accordance with the relevant concrete design standards. Through the implementation of this design process it is expected that floodway structural resilience will be improved as a result of increased durability, serviceability and strength.

As floodways serve a critical purpose in the rural road network any improvement in structural design and integrity will ultimately increase rural community safety and resilience to extreme flood events. An increase in structural integrity and lowering of asset damage after a flood event will have a direct positive impact on local government expenditure while minimising financial disruptions to the local communities through the prevention of access restrictions being imposed. By maintaining a safe access it will also enable quicker disaster response and recovery efforts following a major flooding event.

As this research considers only a single standard engineering floodway type which is currently implemented in the Lockyer Valley Regional Council the opportunity exists for alternative standard floodway types to be examined through the finite element method and parametric approach adopted in this research to provide a more widely adaptable approach.

Practical implementation of this design methodology would also allow performance to be quantified in real terms, based on exposure to different flood events. Opportunity also exists

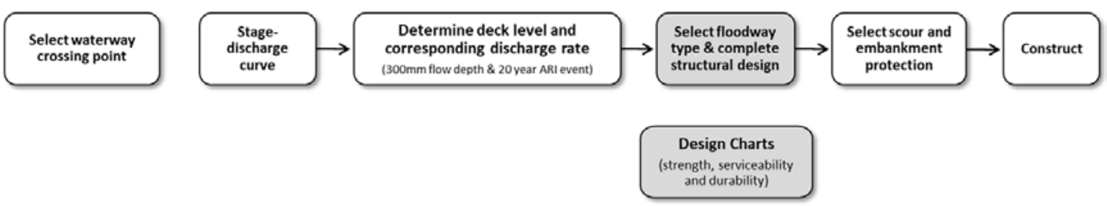

Figure 15.
Integrated floodway design process

for the application of this research and findings to be adapted to other small rural road structures, such as culverts and bridges.

6. Conclusion

This research has investigated a finite element methods approach for the inclusion of structural analysis into current floodway design procedures. A parametric analysis was conducted for the standard engineering floodway type identifying areas of structural vulnerability and the worst-case loading scenario. This worst-case loading scenario was found to occur during the impact loading case, which incorporated a 4-tonne boulder impact, no downstream rock protection and a 900 mm cut-off wall depth. This configuration is consistent with the damage experienced to floodways during the QLD floods of 2011 and 2013 as described by Wahalathantri *et al.* (2018).

Based on this investigation several structural design charts were deduced. These charts provide the maximum absolute bending moment and shear force values from the worst-case loading scenario. These charts allow steel and concrete to be designed to satisfy the relevant countries code requirements for strength, serviceability and durability of concrete structures for this floodway type while under extreme flood loadings.

An integrated design method was developed to incorporate the structural design charts into the current hydraulic design procedures stated within floodway design guidelines. This process will ensure adequate structural resilience, aiding in reducing maintenance and periods of unserviceability in the wake of extreme flood events.

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Further reading

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About the authors

Isaac Greene is a Masters by Research student and Casual Research Assistant in the Centre for Future Materials (Research) at University of Southern Queensland (USQ). His research interests focus on improving the resilience of road structures under extreme flood loading and current Masters by

Research project seeks to deduce a structural design process for floodways. Isaac completed his Bachelor of Engineering Civil (Honours) at USQ in 2018, holds membership with Engineers Australia and works full-time as a Civil Project Engineer.

Weena Lokuge is a Senior Lecturer in Civil Engineering at University of Southern Queensland. Her research areas include construction materials, composite materials, rehabilitation of concrete structures and infrastructure management. She has over 90 research papers in structural engineering area and has 17 years of academic experience. She is a member of the Institute of Engineers, Australia and a committee member of the Queensland Branch of Concrete Institute of Australia. Weena Lokuge is the corresponding author and can be contacted at: weena.lokuge@usq.edu.au

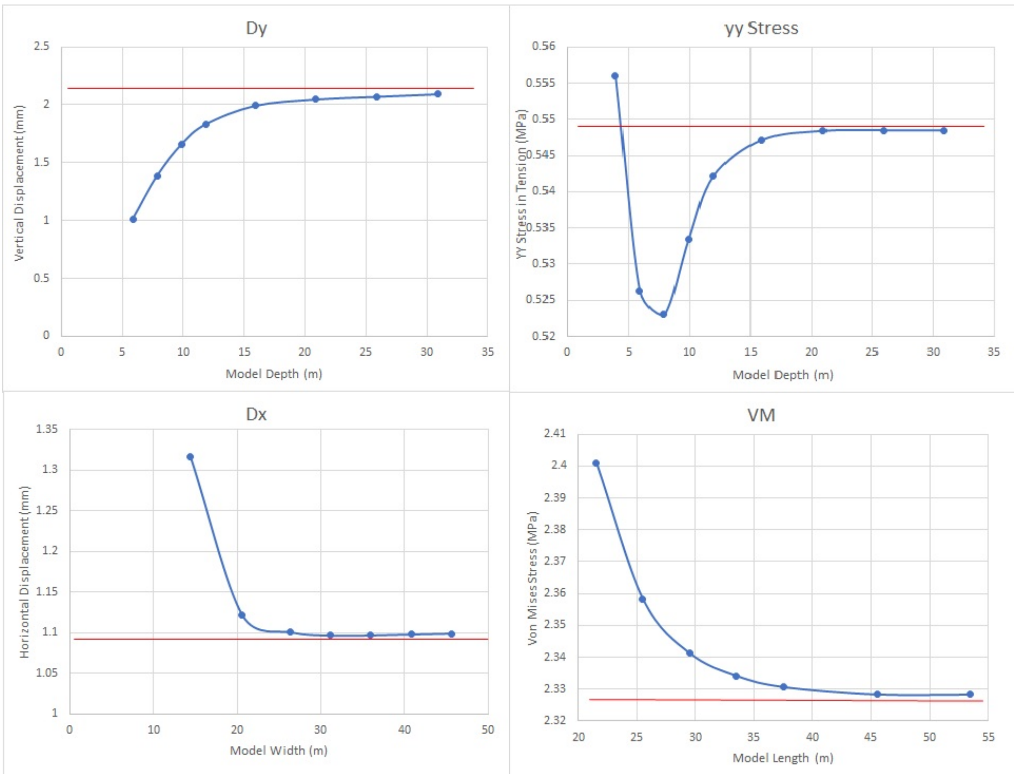
Warna Karunasena is the Discipline Leader in Civil Engineering and Construction in the School of Civil Engineering and Surveying at University of Southern Queensland (USQ). His research interests include structural behaviour modelling, structural repair and rehabilitation, fibre composite materials and enhancing resilience of road infrastructure. He has over 165 publications in structural engineering and mechanics area and has over 30 years of experience in academia and industry in his field. Karu has a top 8% world ranking in Researchgate with 19,000 reads of his research articles. His journal articles in Scopus Mendeley database have received 75000 views. Karu holds memberships in Engineers Australia and American Society of Civil Engineers. He is a registered professional engineer in Queensland and a charted professional engineer.

Annexure to Publication III

Annexure to address the Examiners' suggested revisions to the published manuscript, *"Structural Design of Floodways Under Extreme Flood Loadings"*, in *International Journal of Disaster Resilience in the Built Environment.*

1. Add the below paragraph and figures to sub-section 3.1, titled, *"Model development"*, dot point *"Mesh and model refinement"*:

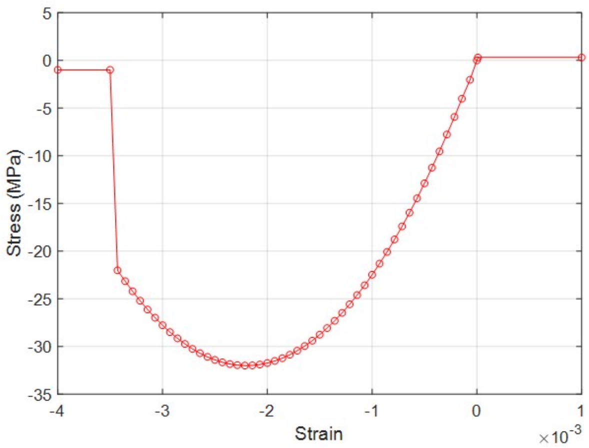
   As part of this convergence analysis the effect of different loading on key parameters such as displacements (vertical and horizontal) and Von Mises stress were plotted until results asymptotically converged. This indicating that the refinement of the mesh density or the model extents no longer influenced the output and that the output is closely correlated with the exact solution. The final derived mesh size also provided an acceptable calculation time.



2. Add the below paragraph to sub-section 3.1, titled, *"Model development"*, dot

point *"Element types and criterions"*:

Max Stress, a predefined non-linear elastic constitutive material behaviour model within Strand7 is commonly used to define concrete and brittle materials. Two main failure criteria are assumed within this model, which are compressive crushing or tensile cracking of concrete. This is achieved through defining a non-linear stress-strain table in terms of effective stress and effective strain, with the graphical output of this table shown in the below figure. In this model the effective stress is a stress variable that is a function of the stresses at a given point and the effective strain is a strain variable that is a function of the strains at that point (Strand7, 2018). These values can therefore relate the three dimensional stress state that may exist at a point, to the defined uni-axial stress-strain curve, in either tension or compression. Within the defined stress-strain curve it can be seen that once the concrete's maximum compressive strain of -0.0022 is reached the material is said to have been crushed, while the contribution of concrete in tension is ignored, however a small value is assigned to assist with non-linear convergence within the solver.



3. Add the following references and changes (due to reporting error for cohesion) within *Table 1*, sub-section 3.1 titled, *"Model development"*:

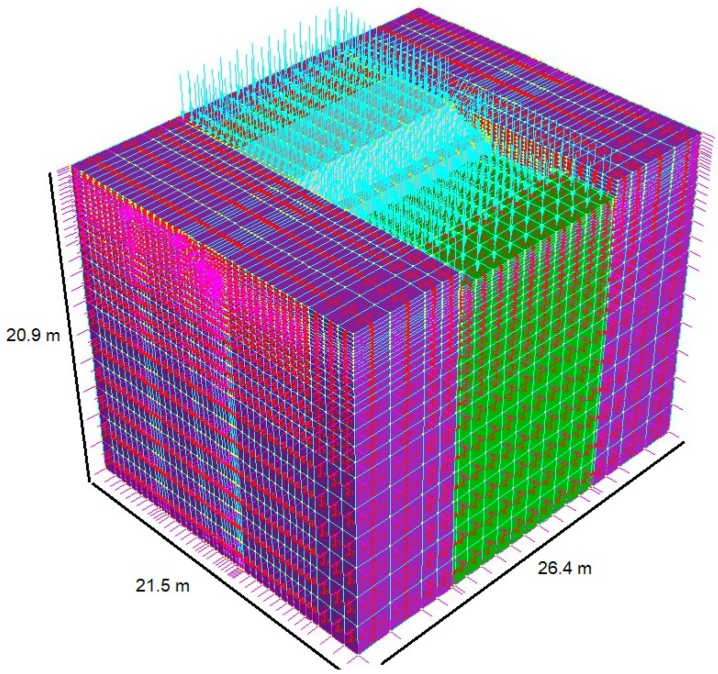
    - Rock - Cohesion (MPa) = 0.01
    - Gravel Sub-base (Cummings, 2015) - Cohesion (MPa) = 0.01

- Gravel base (Cummings, 2015) - Cohesion (MPa) = 0.01
- Silty sand (Cummings, 2015)
- Sandy soil (Koliji, 2013)
- Clay sand (Koliji, 2013) - Clay soil parameters are saturated.

4. Add the below reference to sub-section 3.2, titled, *"Verification"*, dot point *"Mohr–Coulomb response with concrete–soil interface"*:

   The coefficient of friction (friction factor) of 0.55 was based on the interface materials, mass concrete on a gravel foundation as reported by Dept. of the Navy (1982). The friction factor of 0.99 represents a case without friction for comparison purposes, while still enabling the Strand7 solver to run.

5. Replace Figure 3 with:



6. Add the below paragraph to the end of sub-section 4.1.1, titled, *"Load Combination A -hydrostatic loading"*:

   The maximum recorded stress of 1.04 MPa, which corresponded to a flow depth of 2 m did not exceed the maximum stress-strain point (-0.0022, 32 MPa) defining concretes maximum compressive strength within the Max

Stress constitutive behaviour model. Furthermore, although the contribution of concrete in tension was ignored in the constitutive model to alleviate convergence difficulties, AS 5100.5, *Bridge Design - Concrete* details that the maximum flexural tensile strength of 32 MPa concrete is 3.39 MPa for which the stress values are also well below. This indicating that the concrete is still within the elastic range and that failure due to crushing and cracking did not occur.

7. Add the below paragraph to the end of sub-section 4.1.1, titled, *"Load Combination B - boulder impact and hydrostatic loading"*:

   Similar to Load combination A, the maximum recorded stress of 2.423 MPa did not exceed the maximum stress strain point (-0.0022, 32 MPa) or the maximum flexural tensile strength of 3.39 MPa and therefore the concrete did not fail due to crushing or cracking. Impact loading, however is a very concentrated loading, and if boulder size was increased above the 2-tonne load simulated within this sensitivity study, then localised damage is likely and would be characterised by cracking on the outer tensile face.

8. Add the below paragraph to the end of sub-section 4.1.1, titled, *"Load Combination C - vehicular, debris and hydrostatic loading"*:

   Similar to Load Combination's A and B, the maximum recorded stress of 1.93 MPa did not exceed the maximum stress strain point (-0.0022, 32 MPa) or the maximum flexural tensile strength of 3.39 MPa and therefore the concrete did not fail due to crushing or cracking.

9. ρ in Table 3 has been amended to state *"ρ (soil (bulk) mass density)"*.

CHAPTER 5

APPLYING DESIGN METHODOLOGY TO SEVERAL FLOODWAY TYPES

Publication IV: Structural Analysis of Five Standard Engineering Floodway Types

Greene I, Lokuge W & Karunasena W, "A Numerical Approach to Improving the Resilience of Floodway Structures Under Extreme Flood Loading", *Sustainable and Resilient Infrastructure.* Submitted - September 2021, Currently Under Review.

This chapter contains the Author's version of the above submitted manuscript.

Authorship Contribution Statement

The contribution of Isaac Greene (Candidate) was 75%. Isaac undertook numerical model development, verification, result analysis and interpretation, drafting, revising and finalisation of the manuscript. Assoc. Prof. Weena Lokuge (Principal Supervisor) and Prof. Warna Karunasena (Supervisor) contributed to the formalisation of ideas, technical input and editing/co-authoring of the manuscript. These contributions were 15 and 10% respectively.

Linking Manuscript to Research Outcomes

The research undertaken in Chapter 4, Publication III stated an initial method of investigating the vulnerabilities and the requirements needed to satisfy structural design of a single standard engineering floodway type through the use of three-dimensional finite element methods. This paper extends this research to provide a more widely adaptable approach by analysing an additional four commonly used standard engineering floodway types, with structural variation including structures level and raised in relation to the channel bed. The primary outcome of this manuscript was to deduce a conclusive numerical analysis methodology for the analysis of floodways as well as increase the scope of knowledge pertaining to the numerical simulation and investigation of floodways.

To achieve the research outcomes further numerical simulation was performed using a parametric and deterministic approach. Comment relating to design resilience, efficiencies, and comparison with the qualitative survey results from Publication 1, *'Floodways and Flood-Related Experiences: A Survey of Industry Experts and Asset Owners'* was made. Design shear force and bending moment values were also documented in the form of design charts.

The increased level of design input achieved within this manuscript lead to improved structural integrity and resilience outcomes within the design guideline in Publication V, 'Floodway Design Guideline'. The engineering floodway types analysed in this publication, as well as the Type 2 floodway in Publication III, 'Structural design of floodways under extreme flood loading' form a complete set of standard drawings for use in the Floodway Design Guideline, with typical details provided for generalised situations.

A numerical approach to improving the resilience of floodway structures under extreme flood loading

I. S. Greene[a]*, W. Lokuge[a] and W. Karunasena[a]

[a]*Centre for Future Materials, School of Civil Engineering and Surveying, University of Southern Queensland, Queensland, 4300, Australia.*

*Corresponding author (Isaac.Greene@usq.edu.au)

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Isaac Greene (BEng(Civil)(Hons), GradCertEngMgt, MIEAust) - is a Researcher at the Centre for Future Materials, University of Southern Queensland (USQ). His research interests focus on improving the resilience of road structures under extreme flood conditions.

Weena Lokuge (BSc, MEng, PhD, PGCertTertT&L, MIEAust) - is an Associate Professor in Civil Engineering at USQ since 2010. Her research areas include composite materials, construction materials, rehabilitation of concrete structures and infrastructure management.

Prof Karu Karunasena (BScEngCivil), MEng, PhD, FIEAUS, MSCE, CPEng, RPEQ) - is the Discipline Leader in Civil Engineering and Construction in the School of Civil Engineering and Surveying at the University of Southern Queensland (USQ). His research interests include structural behaviour modelling, structural repair and rehabilitation, fibre composite materials and enhancing the resilience of road infrastructure.

A numerical approach to improving the resilience of floodway structures under extreme flood loading

Floodways are small road structures directly subjected to extreme flood loadings. Flooding in the past decade has presented a reoccurring threat to the built environment, with significant damage to floodway structures documented. This damage carries significant social and economic consequences through the delay in the safe reopening of roads. Limited research has been undertaken into the effect of flood loadings on floodway structures, and current design guidelines consider hydraulic design principles only. This study involves the concept of resilience and evaluates the vulnerability of different floodway structures against accidental actions resulting from debris impact during extreme flood events.

Within this study a numerical analysis procedure was developed, refined, and applied to four floodway cases. In each case, the effect of floodway geometry on the stress and displacement relationship was investigated. Bending moment and shear force diagrams were also deduced and documented as design charts. These charts provide a design basis for resilience in terms of strength and serviceability across multiple floodway scenarios with varying channel applications.

Keywords: extreme events; floods; floodways; infrastructure; finite element analysis; resilience; road structures.

1. INTRODUCTION

Flooding is a natural disaster resulting from short and intense rainfall associated with adverse weather patterns and tropical cyclone events (Sultana, Chai, Chowdhury, & Martin, 2016). Flooding presents a reoccurring threat to the built environment and human life with studies reporting a change in magnitude and geomorphic state over the past century (Nasr, 2019; Sassu, Giresini & Puppio, 2017; Thompson and Croke, 2013; Sultana et al., 2016). From December 2010 to January 2011 severe weather events, including Tropical Cyclone Yasi, caused widespread flooding to Queensland, Australia

(McCall, 2012). These weather events resulted in 59 flooded rivers, broke 12 flood records and devastated critical road infrastructure with a restoration expense of approximately 5 billion (Sultana et al., 2016). Floodways were among the critical infrastructure affected. The Lockyer Valley Region situated in Queensland, Australia, which this research references, reported 58% of floodways within their municipality were damaged due to being directly subjected to extreme flood loadings whilst in a submerged state (Lokuge, Setunge, & Karunasena, 2014). Due to the elevated likelihood of damage to floodways during flooding, delays in the safe reopening of roads were experienced, exacerbating the emergency, and preventing humanitarian efforts from commencing expeditiously. Therefore, improvements to the design and performance of floodway infrastructure is required to reduce both social and economic impact to affected communities post extreme flood events (Nofal & van de Lindt, 2020).

In the Australian context, floodways are structures designed to improve the trafficable crossing surface of waterways to enable the safe and predicable passage of vehicles (Figure 1). This is often achieved by incorporating an appropriately designed concrete structure within a channel to allow flows to pass over or through it in a controlled and uniform manner (North Central Catchment Management Authority, 2012). Floodways are often implemented in ephemeral, ill-defined, and perennial channels. These channel types are classified within the research by Rosgen (1994). Once a predetermined flow depth of 300 mm is reached, traffic access is precluded due to the possibility of vehicle buoyancy, and the floodway is no longer considered serviceable (Conesa-García, García-Lorenzo, & Pérez-Cutillas, 2017; Main Roads Western Australia, 2006). In continuous and low flow watercourses, culverts are incorporated to prevent the road from continually overtopping, thus increasing amenity (Keller &

Sherar, 2003). Floodways, which are considered small road structures, are often a desirable alternative to bridges and culverts due to economic savings derived during the construction phase. They are also suited to low-volume roads, when site-specific hydraulic and hydrological conditions permit and when periods of closure can be accepted by the community being serviced (Keller & Sherar, 2003).

FIGURE 1. A typical low-water floodway crossing structure situated in a perennial waterway.

Regional municipalities typically use generalised engineering floodway drawings to initiate design. They then adapt the design to the channel of concern, subject to local conditions and constraints derived from site-specific investigations. These standard drawings are often adopted from standard floodway structures included within floodway design guidelines (Main Roads Western Australia, 2006; Department of Transport and Main Roads, 2010; Austroads, 2013). Current design guidelines consider hydrologic and hydraulic design criteria only to control flow over the structure and to prevent the development of supercritical flows that result in structural damage from erosion and scour (Keller & Sherar, 2003). Floodways, however, operate in a complex environment and are often in a frequently submerged state, making them particularly vulnerable to changes in water level, flow rate and associated flood loadings (Conesa-García et al. 2017). These loadings are comparable to those described in AS5100.2:2017, *Bridge Design - Design Loads*, including hydraulic, impact, debris, drag, and lift (Standards Australia, 2017). Research and investigation have also been undertaken into the primary mechanism of floodway failure during extreme flood events (GHD, 2012; Wahalathantri, Lokuge, Karunasena, & Setunge, 2015). These studies attribute failure to the associated sediment-laden floodwaters and impact from conveyed debris such as logs and boulders (Furniss, Ledwith, Love, McFadin, & Flanagan, 2002; Wahalathantri, 2015; Lokuge et al., 2014). The research by Conesa-García et al. (2017) reported that impacts from the strong dragging of debris associated with extreme flows was not only isolated to incised channels and was present in all floodway sites, including wide floodway crossings at relatively flat channel beds and grades.

Studying the performance of complex concrete road structures using non-linear finite element analysis is highly effective, as large-scale experimental testing and analysis is often prohibitive due to cost, equipment limitations and behaviour complexity (Al-Rousan, Alhassan, & Al-wadi, 2020). A preliminary study, applying linear-elastic theorems and two-dimensional plane strain assumptions was first adopted to model floodway structures (Cummings, 2015). This initial study was recently extended using three-dimensional modelling techniques to allow for the non-linearity of material properties (Greene, Lokuge, & Karunasena, 2020a). Both studies numerically investigated the vulnerabilities of only a single floodway type and had limited emphasis on the vulnerability of floodway structures under impact loading. Therefore, investigating the vulnerability of multiple floodway cases during flood-related loadings is required to deduce more resilient design criteria for floodway structures (Sassu et al., 2017). Standard constitutive models used within the latter study to predict the non-linearity of materials included Maximum Stress and Mohr-Coulomb yield criterions for concrete and soil, respectively (Jiang, 2018; Feng, Malingam, & Irulappasamy, 2019; Ferdous, Manalo, Aravinthan, & Fam, 2018; Greene et al. 2020a).

2. STUDY AREA

The Lockyer Valley Region in Queensland, Australia, was selected as the study area for this research. The Lockyer Valley Region is situated approximately 60 km West of Brisbane and at the base of the Great Dividing Range. The Lockyer Creek is the main waterway that traverses the region and is the largest tributary watercourse of the Brisbane River. A significant number of upper catchments feeds the Lockyer Creek. It has a total catchment area of 2,600 km^2 and has a long history of flooding due to its susceptibility to significant flow variations from high rainfall events (Wahalathantri, 2015). The Lockyer Valley utilises a vast number of floodways (347) to supplement its road network and are critical for the accessibility and functionality of communities within the municipality.

The Lockyer Valley commonly utilises five (5) standard engineering floodway types to enable design. These standard engineering floodway types are referred to as Type 1, 2, 3, 4 and 5. These Floodway types consist of raised and level designs relative to the channel. Floodway Types 1, 3, 4 and 5 were selected for this study (Figure 3), with floodway Type 2 analysed in previous research (Greene et al. 2020a).

3. RESEARCH SIGNIFICANCE

Traditional floodway design is based on hydraulic aspects that control the development of supercritical flows to mitigate the effects of scouring and erosion. Limited research has been undertaken into the impact of flood loadings on floodways structures. This study aims to consider the vulnerability of different floodway cases against impact loading resulting from the movement of boulders during extreme flood events. In this study, a refined and conclusive numerical analysis procedure for floodways is derived and discussed. Utilising this numerical simulation procedure, the investigation of floodway structures under extreme flood loadings is performed utilising a parametric and deterministic approach. This enables informed comment on design resilience, efficiencies, and comparison with qualitative results within literature. This procedure also extends to deriving the resulting bending moment and shear force diagrams and proposes a method to collate these results into structural design charts, thus providing a design basis for floodway structures to ensure that they are resilient to flood-related loadings.

4. METHODOLOGY

4.1 Numerical Analysis Procedure

The numerical analysis procedure derived from this study for the numerical simulation of floodway structures under extreme flood loadings can be described using the methodical process outlined below. This process has also been represented schematically in Figure 2. Steps 9 and 10 provide an advanced post-processing approach to determining the resulting bending moment and shear force diagrams based on the numerical simulation at a point of concern. These derived bending moments and shear force values provide essential parameters for the resilient design of structural floodway elements through the assembly of structural design charts.

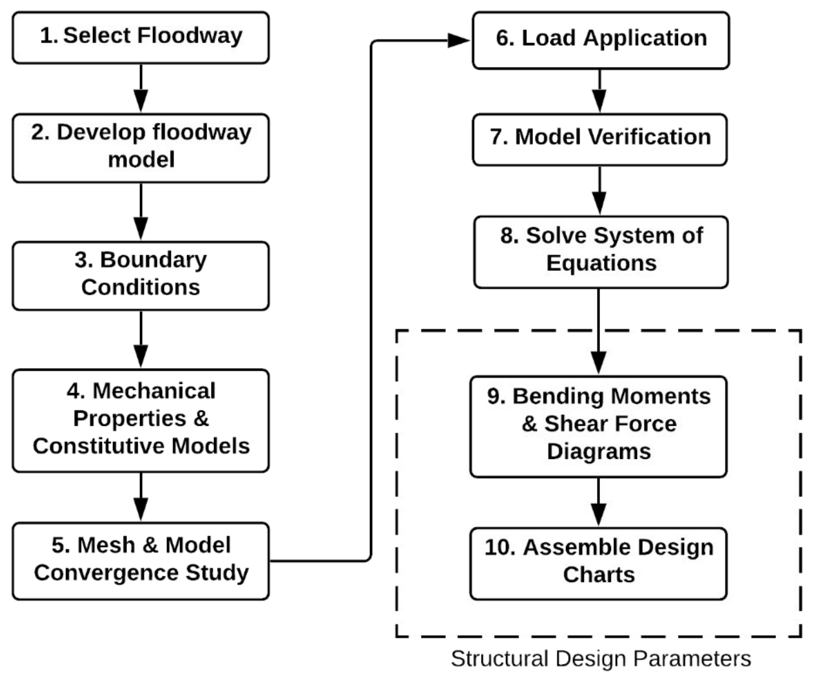

FIGURE 2. Schematic of floodway numerical modelling procedure.

Methodical Procedure:

(1) Select the floodway case for modelling.

(2) Develop the floodway geometry into coarse domains using four node tetrahedra solid three-dimensional brick elements. This shall include all floodway elements, including the concrete floodway structure, upstream and downstream rock protection, road pavement (subgrade and gravel sub-base) and adjoining in-situ soil.

(3) Assign boundary conditions to the numerical floodway model to represent in-situ support constraints. Typically, the outer soil extents would be assigned roller support conditions and the bottom of the model, rigid boundary conditions.

(4) Assign mechanical properties and constitutive models to elements to describe the non-linearity of materials within the floodway model. Max Stress Yield criterion allows the difference in behavior exhibited by concrete in both compression and

tension to be considered, with failure defined when stress exceeds the maximum allowable stress within the assigned stress versus strain relationship. A stress strain model shall be used to define this relationship and enable stress to be calculated for any given strain value. Furthermore, Mohr Coulomb yield criterion is defined based on cohesion and friction angle and considers hydrostatic stress influences within the soil elements.

As the numerical model aims to determine the structural effect of flood related loadings, the floodway shall be considered as acting in a submerged state. This assumption requires the water level to be defined for the various soil element properties in respect to the global axis, which corresponds to the direction of gravity. This also requires the vertical in-situ stress profile to be calculated via Equation 1 for the elements under the model's self-weight (Strand7, 2018). Greene et al. (2020a) also reported that when modelling advanced or irregular shaped models, the water level needs to be interpolated into a series of constant water levels and applied.

$$S_v = -\rho_{soil} g h_{soil} + -\rho_{fluid} g h_{soil\_submerged} \qquad (1)$$

Where,

ρ = density (kg/m^3)

g = gravity (m/s^2)

h = height (m)

Furthermore, significant friction exists between the floodway concrete and soil interface due to the concrete floodway elements mass and the hydrostatic loading acting normal to the concrete floodway surface. A study by Wu, Griffith, & Oehlers (2004) also explained that when significant normal forces exist, the increased friction force at the interface tends to delay failure and shall not be adopted for design purposes. Therefore, a perfect bond shall be assumed for the design of floodways to mitigate discrepancies due to a potential delayed response/failure. Furthermore, the loadings applied in all trialled scenarios within this paper were found to be significantly below the limiting load as derived by the static friction formula (Equation 2).

$$F = \mu N \tag{2}$$

Where,

F = force (N)

μ = friction coefficient

N = normal force (N)

(5) Discretisation of the continuous domain into several sub-domains with a finite number of degrees of freedom. This involves the requirement to undertake a mesh and model convergence study. This study is undertaken through an iteration-based approach, where the degrees of freedom within the finite element model extents and mesh density are iteratively increased until results asymptotically converge within a plot of deflection or stress versus the number of available degrees of freedom. At the point of convergence, the solution approximates the correct answer for the least mesh density and model size, thus assuring an accurate model for the least computational time.

(6) Loading scenario and application. AS5100.2:2017 mathematically defines the ultimate state static loadings that bridge design needs to satisfy (Standards Australia, 2017). These loadings include drag, debris, lift, impact, and vehicular. Prior research undertaken by Greene et al. (2020a) and Cummings (2015) have adopted these loadings to floodway structures. These studies have also investigated the different loading combinations to determine which combination controlled maximum displacement and stress within the concrete floodway structure. The loading combinations trialed were as follows:

(a) Hydrostatic loading.
(b) Boulder impact and hydrostatic loading.
(c) Vehicular, debris and hydrostatic loading.

From these studies, the combination considering boulder impact (velocities up to 8 m/s) and hydrostatic loading (up to 2 m flow depths) consistently produced the highest stress and displacement profiles and was defined as the controlling loading scenario. AS5100.2:2017 states that log impact should not be considered concurrently with debris loading (Standards Australia, 2017). Furthermore, as the floodway is submerged, vehicular access is precluded and therefore not considered. Drag force was also negligible in relation to the other loadings and consequently omitted from the studies (Cummings, 2015). Scenario 2, *Boulder impact and hydrostatic loading* shall therefore be applied to the modelling of floodway structures under extreme flood loading scenarios.

In addition to the above loading case, a load factor of 1.3 shall be applied to velocity within the impact loading scenario (Standards Australia, 2017) and

1.2G + 1.5Q to all other loadings in accordance with AS1170.1 (Standards Australia, 2002).

(7) Verification of model response in the elementary phases shall be undertaken by componentising the full augmented model and correlating stresses and displacement with linear analytical solutions. This can be achieved using theorems such as Hooke's Law, along with visual observations of deformation and magnitude. To provide augmented model confidence, qualitative agreement can be achieved through testing numerical solutions against hypotheses derived from a questionnaire undertaken by Greene, Lokuge, & Karunasena (2020b). This questionnaire contains the responses of subject matter experts regarding floodway vulnerability under extreme flood loading conditions. If agreement exists, then it can be concluded that the finite element methods being applied qualitatively describes the loading-failure interaction of the structure.

(8) At this point, the derived finite element model can be solved for the floodway case with meaningful results; this enabling post-processing to be undertaken, such as parametric and deterministic analysis.

(9) Through post-processing, bending moment and shear force values can be determined by transposing the resulting deflections at the point of concern from the three-dimensional model to a two-dimensional beam model as restraint conditions. The two-dimensional beam model shall be connected rigidly at joints and be geometrically identical to the cross-section of the three-dimensional model. To verify the response of the two-dimensional beam model, Dx and Dy displacement solutions shall be compared to that of the three-dimensional vertical cut-plane model to ensure that no significant discrepancies exist.

(10) Collating the resultant maximum bending moment and shear forces, structural design charts can be assembled for the floodway case for different flow depths, velocities, and soil types. These charts shall be based on the worst-case loading scenario, boulder impact and hydrostatic loading. These charts then form the design basis for structural members in accordance with relevant concrete design standards, providing an expeditious method to satisfy ultimate strength criteria based upon a selected design flood event.

4.2 Modelling Assumptions

Due to the inherent complexity of floodway structures, numerical simulation via non-linear finite element methods requires several assumptions to be applied to simplify the complex geometry, relationships and behaviours exhibited in practice. For the modelling of floodways within the above numerical analysis procedure the following assumptions were applied:

- The applied loadings are static equivalent loads with dynamic factors applied as detailed within AS5100.2, Bridge Design, Design Loads (Standards Australia, 2017), providing steady-state conditions.
- A prismatic channel profile was adopted with a constant cross-section and material properties along the length of the floodway model. Geometric variation in the vicinity of the floodway exists for an accurate representation of the floodway case considered.
- Steel reinforcement within concrete is neglected to determine the actual stresses and strains present within the unreinforced concrete floodway structure and to

enable the failure and damage profile of concrete to be explored. Steel reinforcement is designed in subsequent sections to satisfy the internal forces and moments determined for the load combinations within the numerical modelling.

- The interface between concrete and soil was omitted based upon the sensitivity analysis of different coefficients of friction values undertaken by Greene et. al (2020a) using the Coulomb friction/elliptical plastic model. Greene et al. (2018) concluded from this analysis that the effect of contact can be omitted for floodways as the loads being modelled remain well below the limiting load due to the normal force resulting from the self-weight of the concrete structure.

5. MODEL DEVELOPMENT

Through the application of the above methodical procedure four standard engineering floodway types, Type 1, 3, 4 and 5 from the case study area were selected for analysis (Figure 3). The key differences between these floodway types are noted below:

- Floodway Type 1 is suited to waterways of relatively flat hydraulic grades and where no hydraulic control structure is required (level) to facilitate safe vehicular movements. As a result of the level structure, time of closure during rain events will be more frequent.

- Floodway Type 3 and 4 are both variants of raised floodway structures, with slight changes to the upstream and downstream configurations. They are suited to waterways that require a hydraulic control to facilitate safe vehicular movement (larger catchment areas) and can achieve a higher level of service then level crossings through a reduced time of closure. Raised floodway structures can also facilitate the addition of a drainage culvert for low flow conditions, and to reduce the effect of backwater level upstream.
- Floodway Type 5 is also a raised floodway structure, which can impart a hydraulic control on the waterway and thus well suited to larger catchment areas. Floodway Type 5 also incorporates a V-shaped downstream rock protection arrangement, which creates a localised deepening in the waterway promoting energy dissipation and protection against increased bed shear stress.

Specific components modelled within these cases were the concrete floodway structure, upstream and downstream rock protection, road pavement (subgrade with gravel sub-base) and the adjoining in-situ soil.

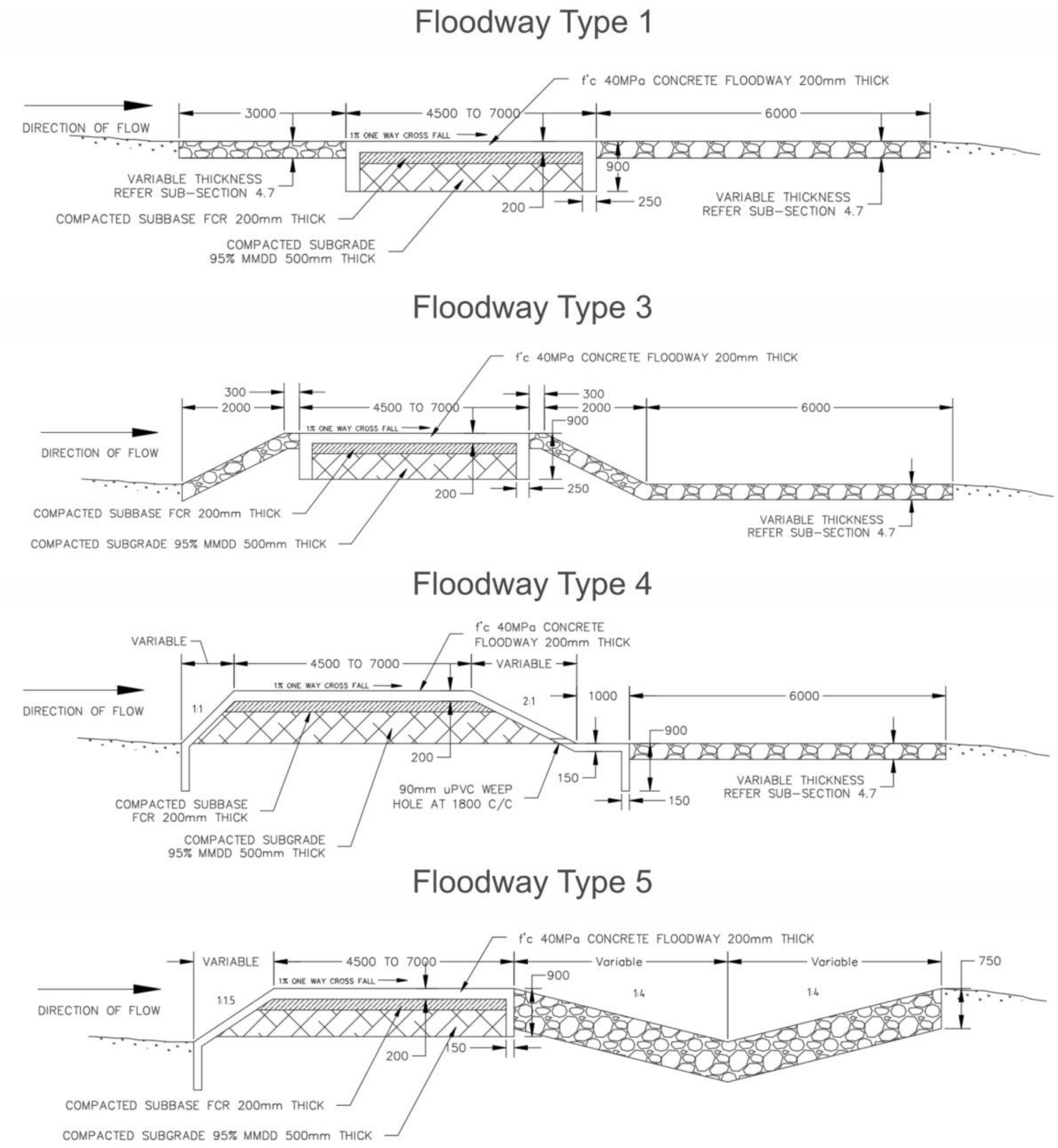

FIGURE 3. The four analytical floodway cases for analysis.

Strand7 (2018), Release 2.4 was the finite element analysis software used within this research. Discrete three-dimensional finite element models were created using four node tetrahedra solid brick elements. Each of these nodes have three translational and three rotational degrees of freedom in the X, Y and Z directions.

Figure 4 provides an XY vertical cut-plane representation of the Type 3 floodway model. This provides a typical example of the final mesh adopted across all four floodway models. It can be observed that a smaller mesh size was adopted at the point of load application. This provides additional physical domains to assist the solver to accurately discretise the larger stress gradients associated with the load across the geometric intricacies of the various floodway components.

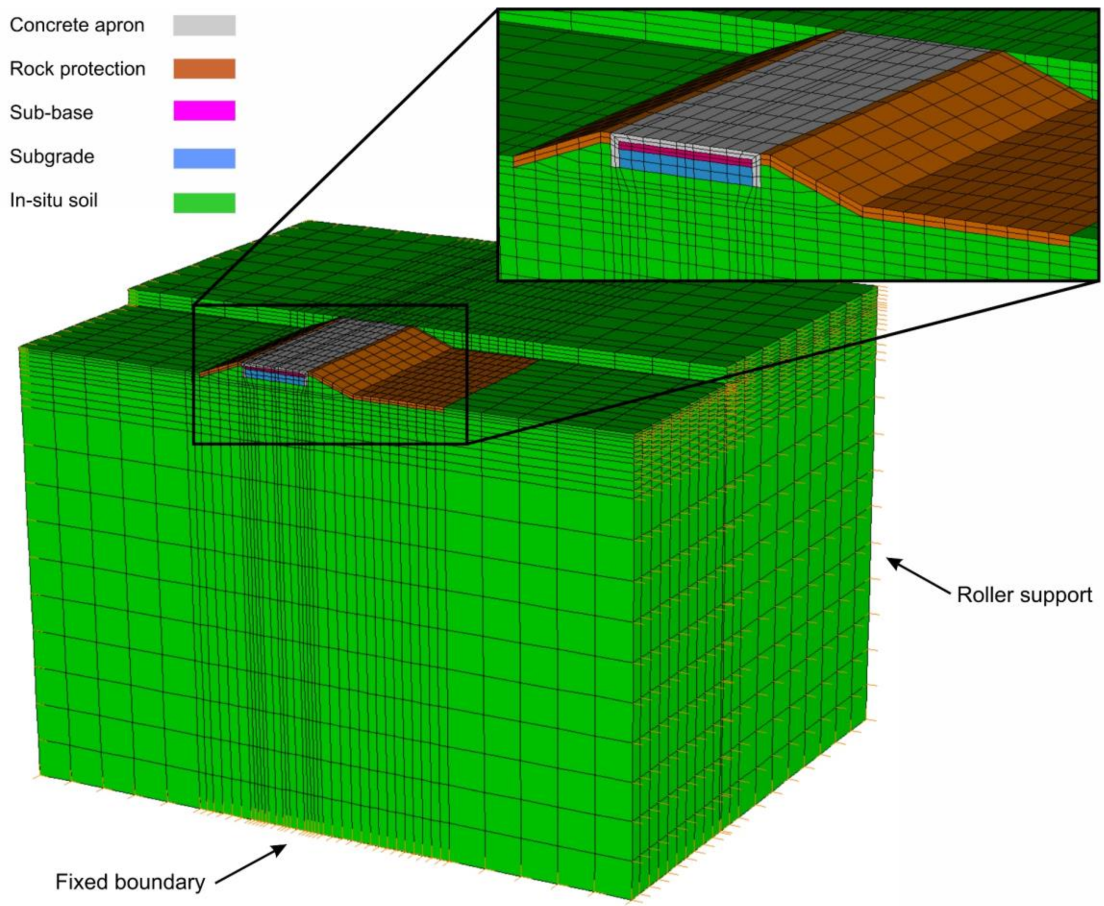

FIGURE 4. The numerical finite element model of Floodway Type 3 (cut-plane).

To represent the boundary conditions, the outer soil extents of the model were assigned roller support conditions, allowing movement in the vertical axis, yet precluding movement in the horizontal axis. The bottom of the model was assigned rigid boundary conditions, that is, the degrees of freedom for all nodes were fixed, precluding movement and rotation in all axes.

Mechanical properties of materials used within the floodway cases are defined in Table 1. Rock protection is assumed to act as a soil material, with a low modulus and density resulting from the significant air voids within the lattice of individual loose packed rocks. Furthermore, several soil types covering the range of typical soil types present within waterways were selected. This was in response to soil type having a significant effect on the resulting numerical solutions. The constitutive models adopted to predict the non-linearity of materials were Max Stress and Mohr-Coulomb yield criterions for concrete and soil, respectively.

TABLE 1. Properties assigned to elements within the model (Greene et al., 2020a).

Material	Modulus (MPa)	Poisson ratio	Density (kg/m3)	Cohesion (MPa)	Friction angle (degrees)
Concrete	31,000	0.2	2,400	N/A	N/A
Rock	100	0.3	1,400	0.01	30
Gravel sub-base	150	0.3	1,900	0.01	30
Gravel base	200	0.3	2,000	0.01	35
Soil 1: Silty Sand	40	0.3	1,700	0.01	25
Soil 2: Sandy Soil	30	0.25	1,800	0.075	34
Soil 3: Clay Soil	100	0.3	1,900	0.01	20

The water level was defined for the various soil elements in respect to the global axis. Strand7 (2018) was used to automatically calculate the vertical in-situ stress profile of individual brick elements under the model's self-weight. Figure 5 illustrates the in-situ stress profile throughout floodway model Type 1 under the model's self-weight and with the corresponding fluid level assigned set at the top of the floodway to reflect submerged conditions. To demonstrate this, Strand7 (2018) calculates undrained soil in-situ stresses below the fluid level by Equation 3.

$$s_v = -(\rho_{soil})(g)(h_{soil}) + (\rho_{fluid})(g)(h_{soil\_submerged}) \quad (3)$$

Where,

ρ = density (kg/m^3)

g = gravity (m/s^2)

h = head (m)

Therefore, at a depth of 2 m, the brick soil stress under self-weight is calculated as follows:

$$s_v = -(1700)(9.81)(2) + (1000)(9.81)(2)$$
$$s_v = -0.0137\ MPa$$

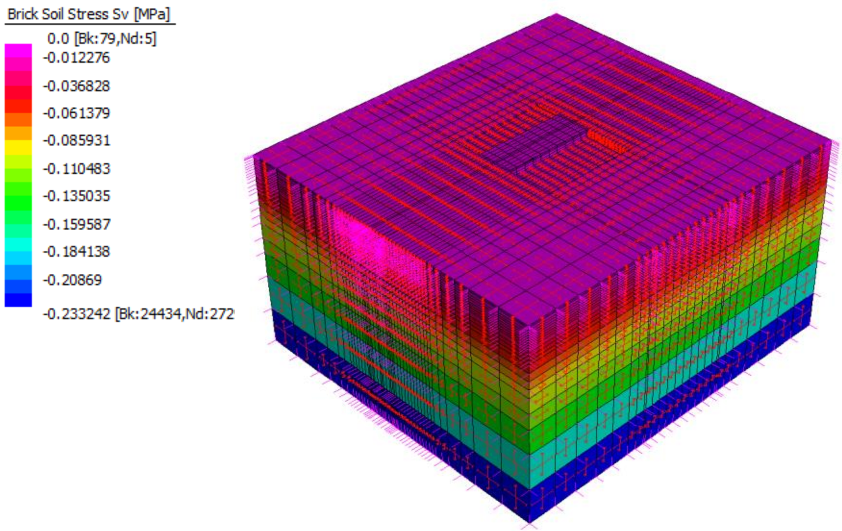

FIGURE 5. Soil in-situ stresses under model self-weight for Floodway Type 1.

Mesh and model refinement of the four floodway cases was undertaken to derive efficient floodway models in terms of degrees of freedom and solution time (Table 2). This was undertaken through an iteration-based approach. Figure 6 shows a typical example of this analysis for floodway Type 3. In the case of mesh refinement, a coarse three-dimensional mesh was developed, with a finer mesh created in the vicinity of the point of loading and the concrete structure. The model was then 'sub-divided' to increase the available domains (degrees of freedom) in increments. Resulting stresses and displacement results were interrogated at the same brick location (brick 1317 and node 367 in the case of Floodway Type 3) on the concrete floodway structure until results converged. The solution selected was just past the point of convergence, therefore providing a model that approximates the correct answer for the least mesh density, model size and computational time.

TABLE 2. Final model size and mesh density.

Floodway	Length (m)	Width (m)	Depth (m)	Mesh construction
Type 1	41.5	45.6	20.9	29,591 Nodes, 26,760 Bricks
Type 3	41.5	55.2	28.7	39,534 Nodes, 36,160 Bricks
Type 4	51.65	52	45.9	103,533 Nodes, 96,384 Bricks
Type 5	62	45.6	38.3	50,098 Nodes, 46,312 Bricks

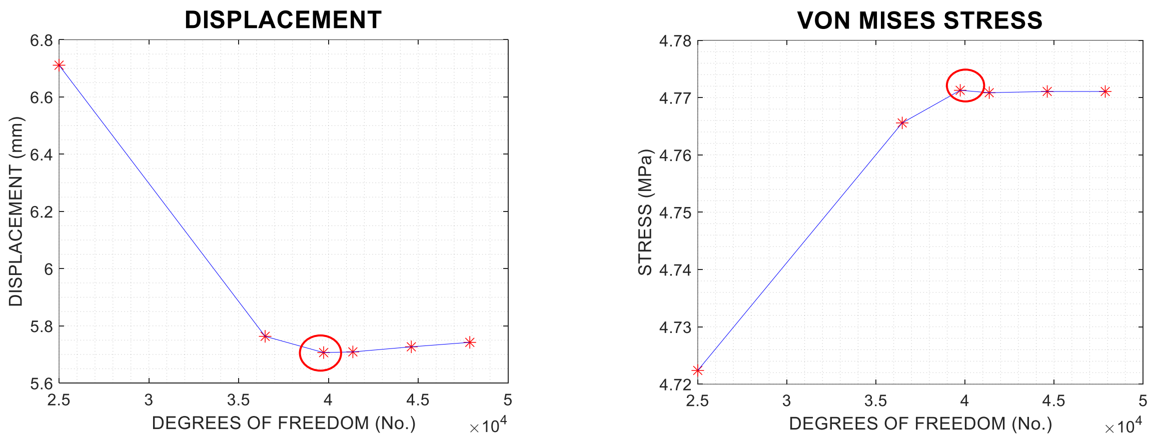

FIGURE 6. Converged plots of deflection and stress versus degrees of freedom (Floodway Type 3).

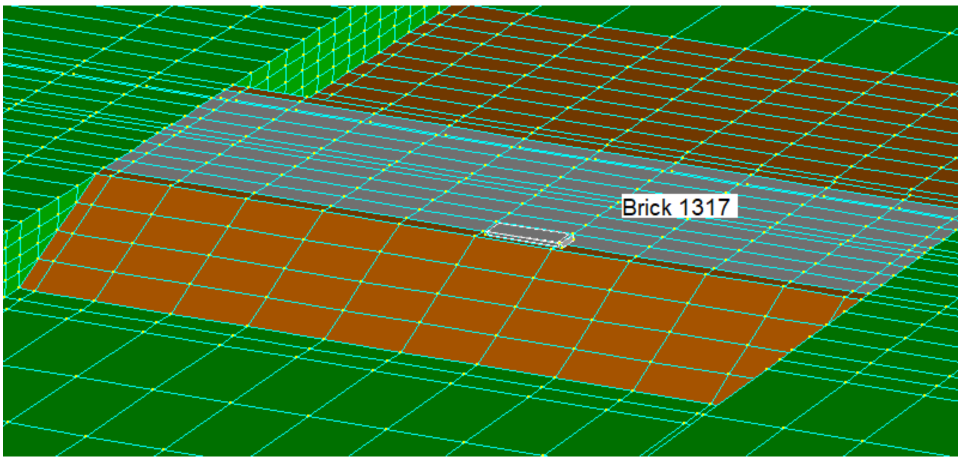

FIGURE 7. Location interrogated for mesh and model refinement of Floodway Type 3.

The final finite element models deduced for the four floodway cases are illustrated in Figure 8.

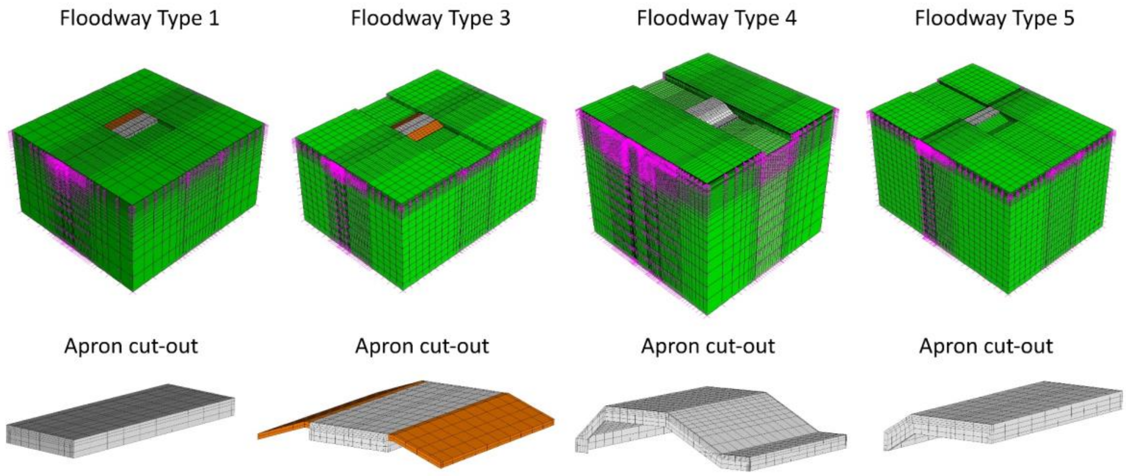

FIGURE 8. Finite element floodway models.

5.1 Loading Scenario & Application

In relation to this study, the loading scenario, *'boulder impact and hydrostatic loading'*, was adopted as the worst-case loading combination for analysis. This worst-case loading scenario is analytically defined in Equation 3.

$$F_{boulder} = 0.5\left(\frac{mV^2}{2d}\right) \quad (3)$$

Where,

$F_{boulder}$ = force (N)

m = objects mass (kg)

v = objects velocity (m/s)

d = stopping distance (m)

This formula is an equation of work with force equal to the kinetic energy of the object impacting the structure. This equation originated from the log impact equation

AS5100.2, Bridge Design, Design Loads (Standards Australia, 2017), which assumes that an object such as a log is buoyant and therefore moving at the same velocity as flow, that is, the velocity of flow is equal to the velocity of the object and no net acceleration is present. Similar research into impact problems relating to bridges have also applied kinetic energy based equivalent static forces after concluding that mass and velocity are the most influential parameters (Abdelkarim and ElGawady, 2017; Zhao and Ye, 2021).

The movement of boulders due to floodwaters is a complex phenomenon as they are submerged and roll, slide and saltate along the waterway channel in both steady and unsteady state flow conditions. Consequently, a factor of 0.5, aligning with the coefficient of drag for a near-spherical boulder was applied to the original impact force equation within AS5100.2, Bridge Design, Design Loads (Standards Australia, 2017) to create Equation 3. This provides a worst-case estimation for the velocity of a submerged boulder which is either in intermediate contact with the channel (with friction force) or fully mobile in the floodwaters (without friction force) and with movement a resultant of drag force, impulsive force, buoyancy, and mass. This approach also coincides with the maximum impact value for log impact within AS5100.2 (Standards Australia, 2017) and the largest dumped graded rock class in Main Roads Western Australia (2006). This, therefore, provides a relevant worst case loading scenario for all waterways even ones where boulders are not present. A constant of 75 mm was also used as the stopping distance within Equation 3. This distance corresponds to the distance used for solid concrete piers in AS5100.2 (Standards Australia, 2017).

To simulate the impact of a boulder, a concentrated distributed force in the form of a pressure was applied to an area of approximately 0.35 m^2 at the very upstream edge of the floodway (Figure 9). This loading was applied centrally, corresponding to the mid-section of the channel, where flows are typically deepest and velocity greatest. In this study a 4-tonne boulder and a range of velocities from 0 to 8 m/s were simulated.

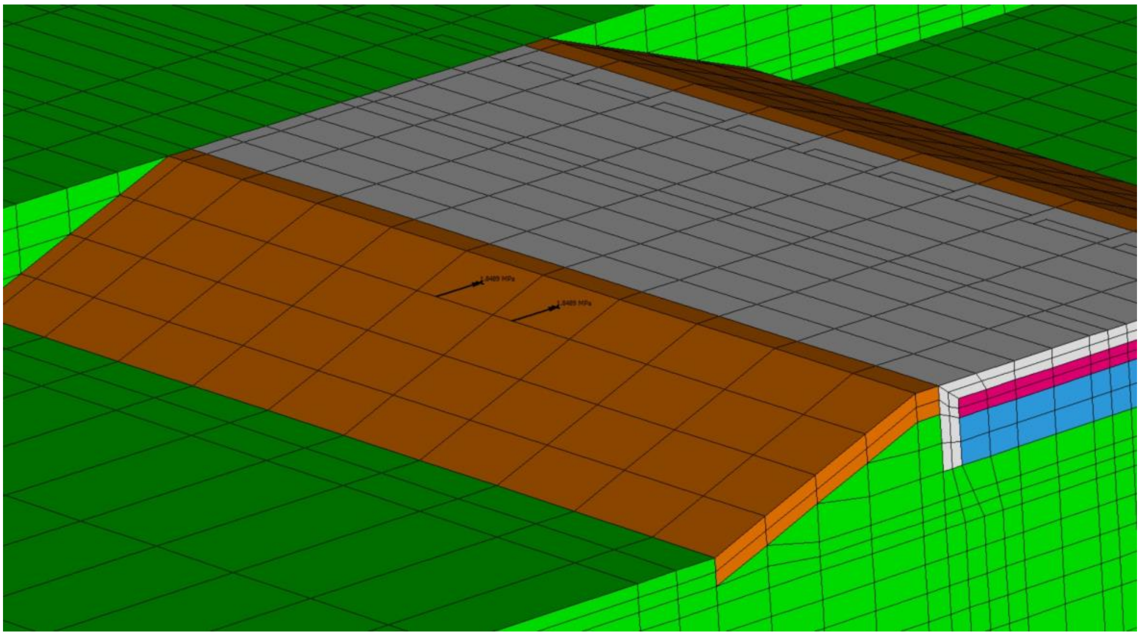

FIGURE 9. Typical impact load application, Floodway Type 3.

Hydrostatic pressure (Equation 4) is a linear relationship proportional to density, gravity and water level and acts perpendicular (normal) to the surface. In this study water level was based on a variable flow depth between 0 and 2 m and applied as a distributed pressure (Figure 10).

$$P = \rho g h \tag{4}$$

Where,

P = hydrostatic pressure (Pa)

ρ = medium density (kg/m^3)

g = gravity (m/s^2)

h = fluid height (m)

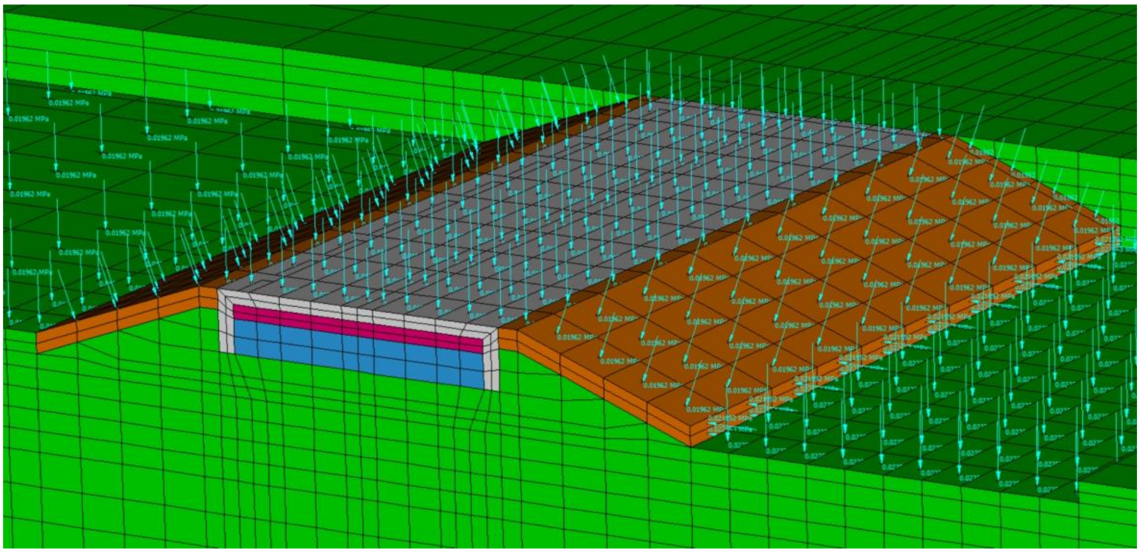

FIGURE 10. Typical hydrostatic load application.

In addition, to the above loading case (boulder impact and hydrostatic pressure) a load factor of 1.3 was applied to velocity within the impact loading scenario (Standards Australia, 2002) and 1.2G + 1.5Q to all other loadings in accordance with AS1170.1 (Standards Australia, 2002).

5.2 Bending Moment & Shear Force Diagrams

Following the post-processing methodology in section 4, the bending moment and shear force diagrams for each floodway case were deduced and provided in Figures 11 to 14. These diagrams are based on the derived loading case, and at the maximum flow depth and velocity analysed (2 m and 8 m/s, respectively) and with 5% exaggeration.

The largest bending moment and shear forces occur at the downstream cut-off wall for all floodway types, except Floodway Type 5. For Floodway Types 1, 3 and 4 the horizontal impact loading is transferred through the horizontal apron in the same plane as the load until it encounters the downstream cut-off wall interface. The transferred load causes the cut-off wall to bend generating a significant bending moment at this location.

Similarly, the transferred load generates forces perpendicular to the downstream cut-off wall member, therefore generating a shear stress within the downstream cut-off wall. In the case of Floodway Type 5, the horizontal impact load is distributed over the upstream batter, which is not in the same plane as the load direction. As a result, the largest bending moment and shear force is induced at the interface of the upstream batter and the apron, followed by the downstream cut-off wall. In both cases, the magnitude of bending moment and shear force generated is a function of the cut-off wall surface area, the distribution and intensity of the load and the mechanical properties of the adjoining soil. Furthermore, the more complex the geometry of the floodway, like in the case of Floodway Type 3, the greater the bending moment and shear force distribution, due to the transferred load having more locations to induce bending action and force perpendicular to the member.

Moments and shear force generated in Floodway Type 1 (Figure 11) where larger than that in Type 3 (Figure 12). This is largely attributed to the mechanical properties of the adjoining soil types and their ability to support the structure against bending. In the case of Floodway Type 1 natural adjoining soil material was present, while Floodway Type 3 had rock batter protection present. Floodway Type's 4 and 5, also had relative high bending moments and shear forces due to their geometric complexity and due to them also bearing against natural adjoining soil material (Figures 13 and 14).

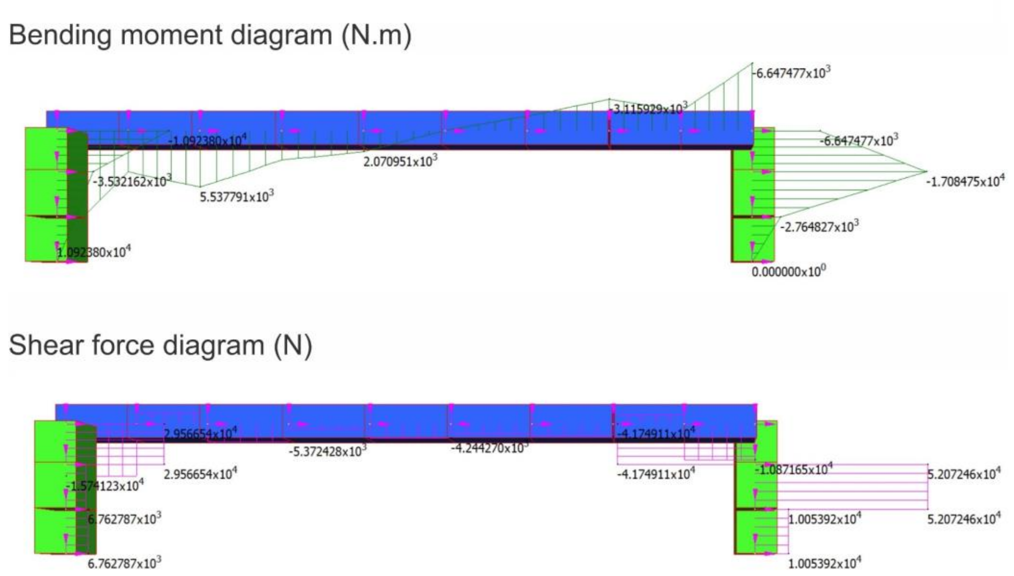

FIGURE 11. Floodway Type 1 typical bending moment and shear force diagram.

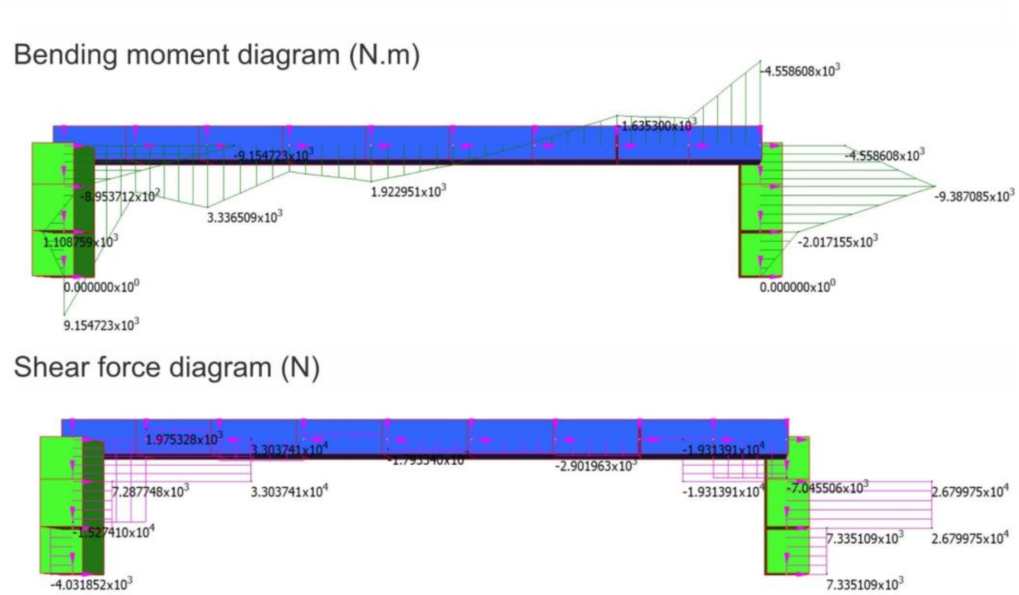

FIGURE 12. Floodway Type 3 typical bending moment and shear force diagram.

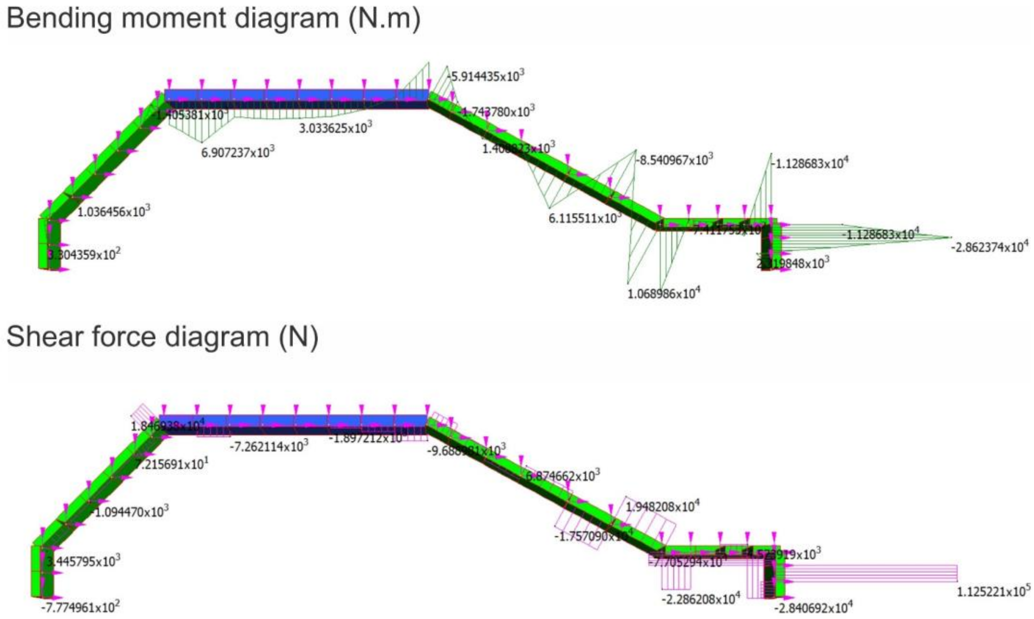

FIGURE 13. Floodway Type 4 typical bending moment and shear force diagram.

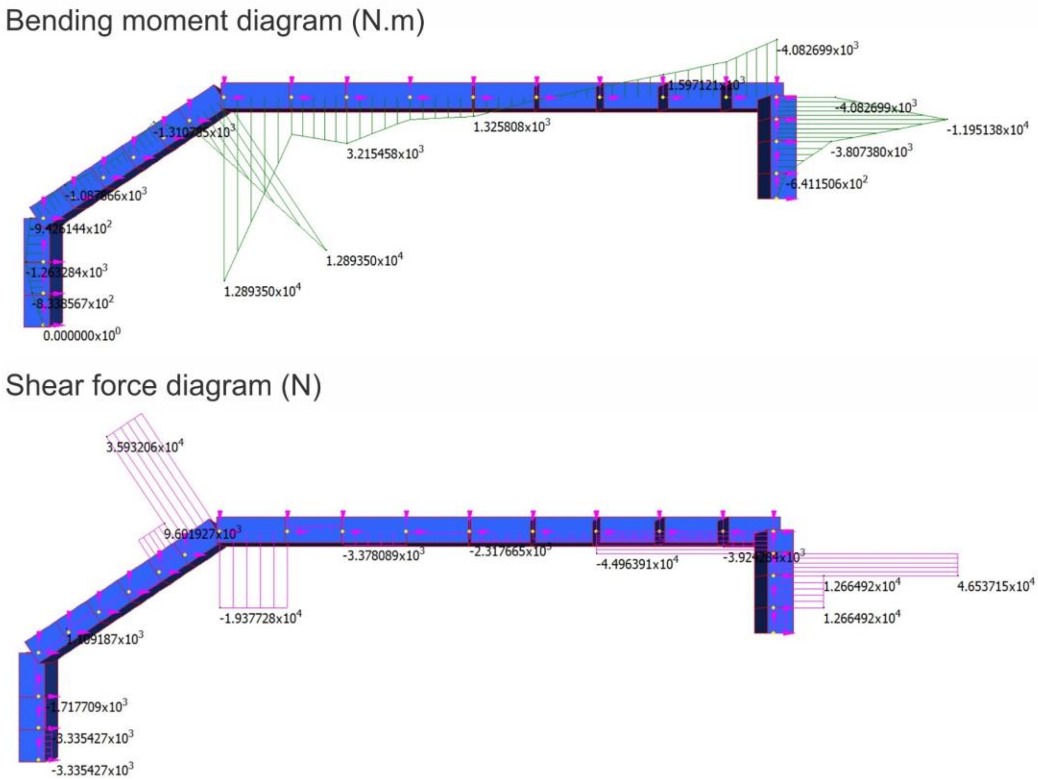

FIGURE 14. Floodway Type 5 typical bending moment and shear force diagrams.

5.3 Structural Design Charts

As the final step of the floodway numerical modelling procedure, the resultant maximum bending moment and shear forces can be collated and arranged as structural design charts for the four floodway cases based on different flow depths, velocities, and soil types (Figures 15 – 18). These charts provide the design basis of strength and serviceability requirements for the concrete floodway structures.

Investigation into why shear force for higher flow depths is lower in some instances then that of lower flow depths in the design graphs was discovered to be a result of the load combination and resultant vectors of loading opposing each other. This was more pronounced in floodway types with complex geometries where hydrostatic loading (predominately a negative load with both horizontal and vertical components) and impact loading (positive horizontal load only) resulted in opposing load types.

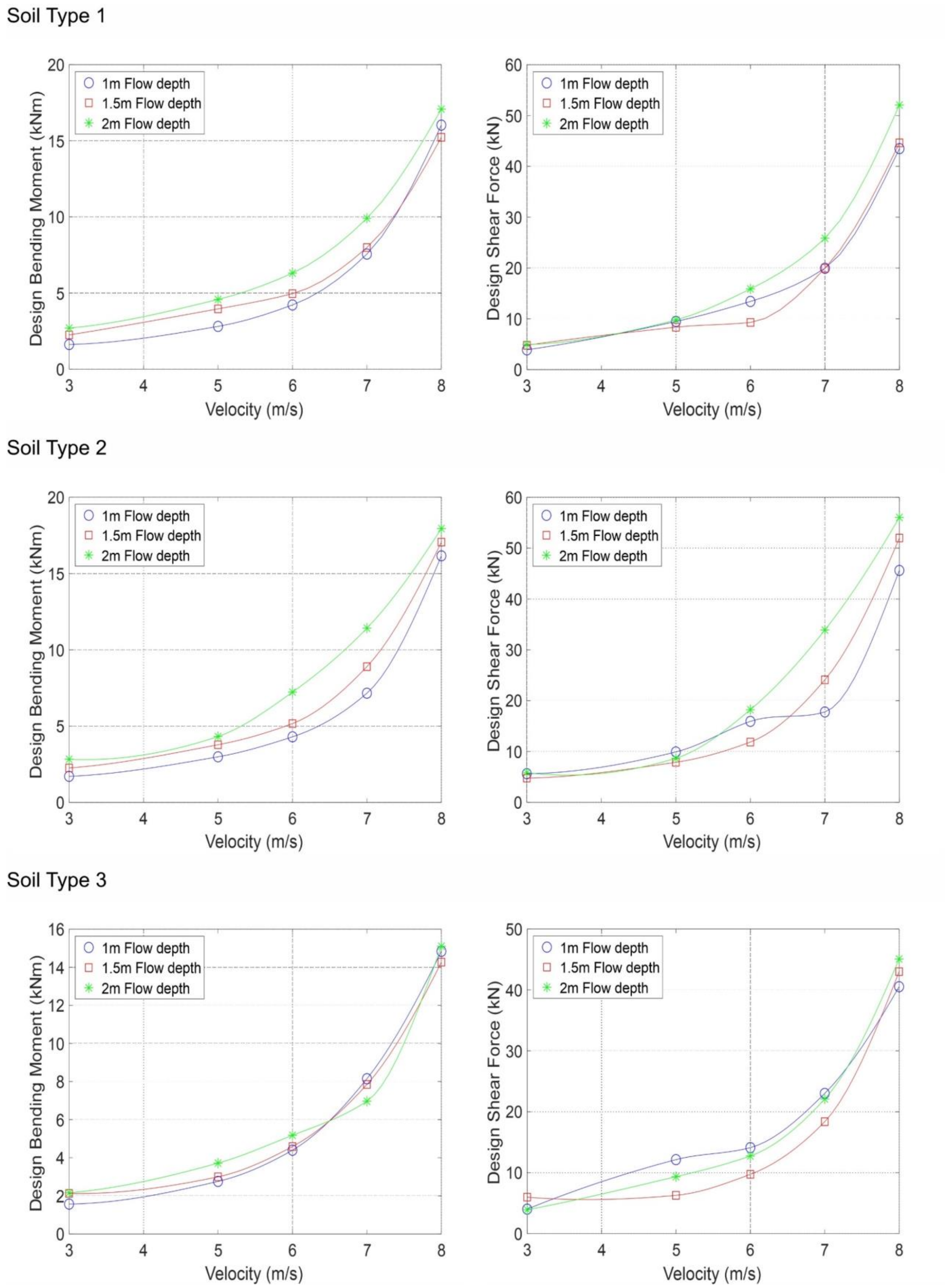

FIGURE 15. Floodway Type 1 structural design charts.

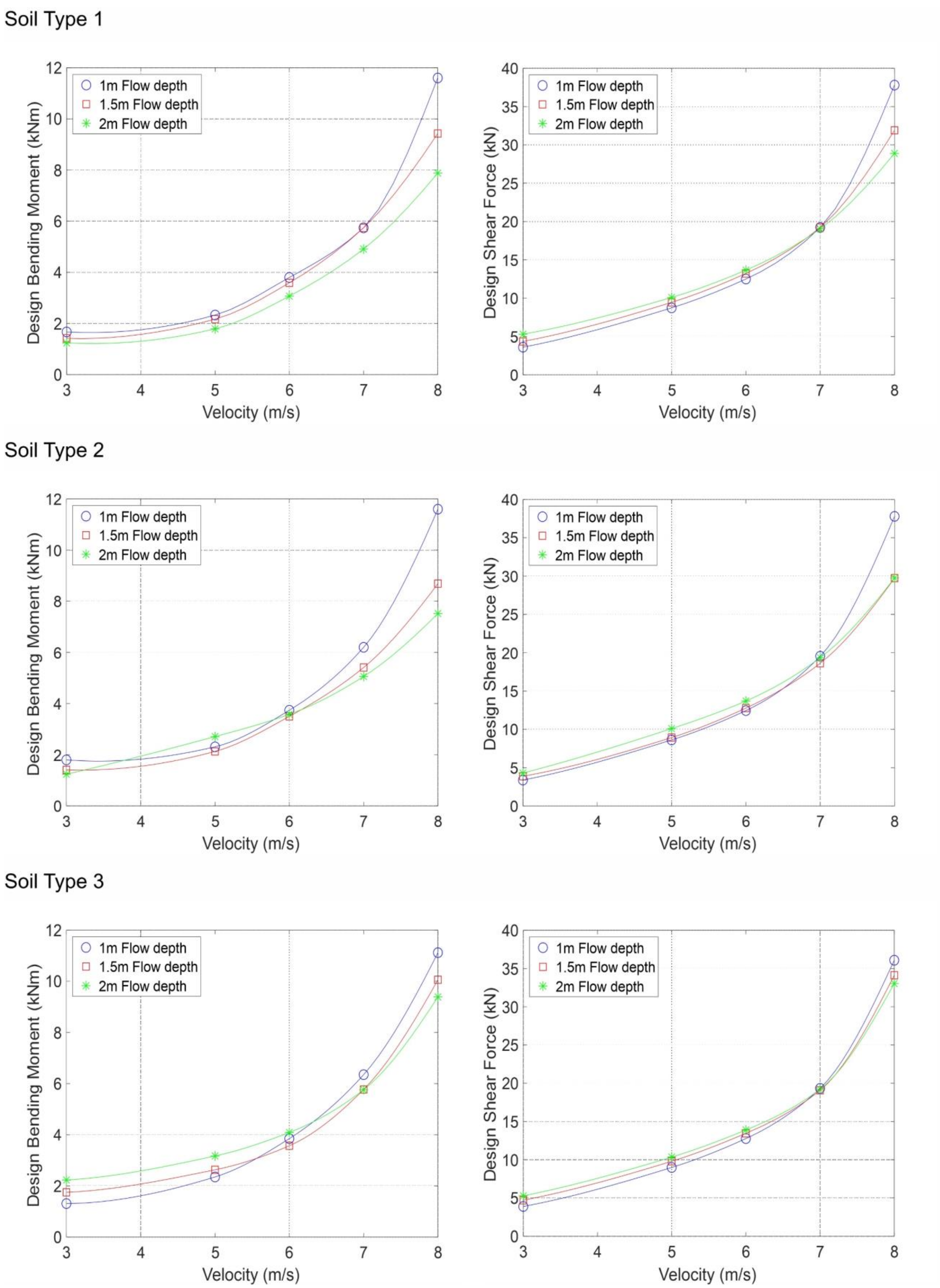

FIGURE 16. Floodway Type 3 structural design charts.

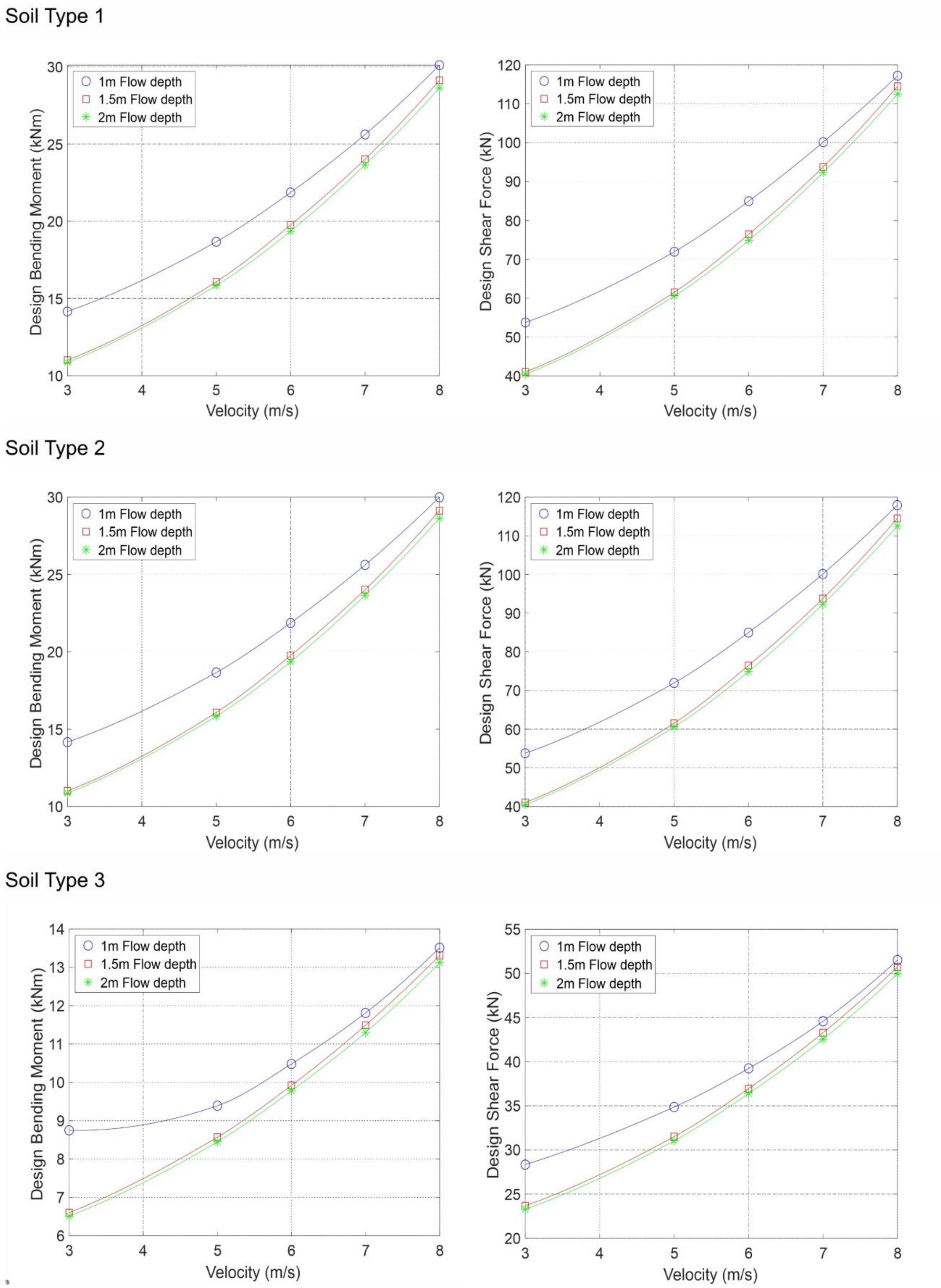

FIGURE 17. Floodway Type 4 structural design charts.

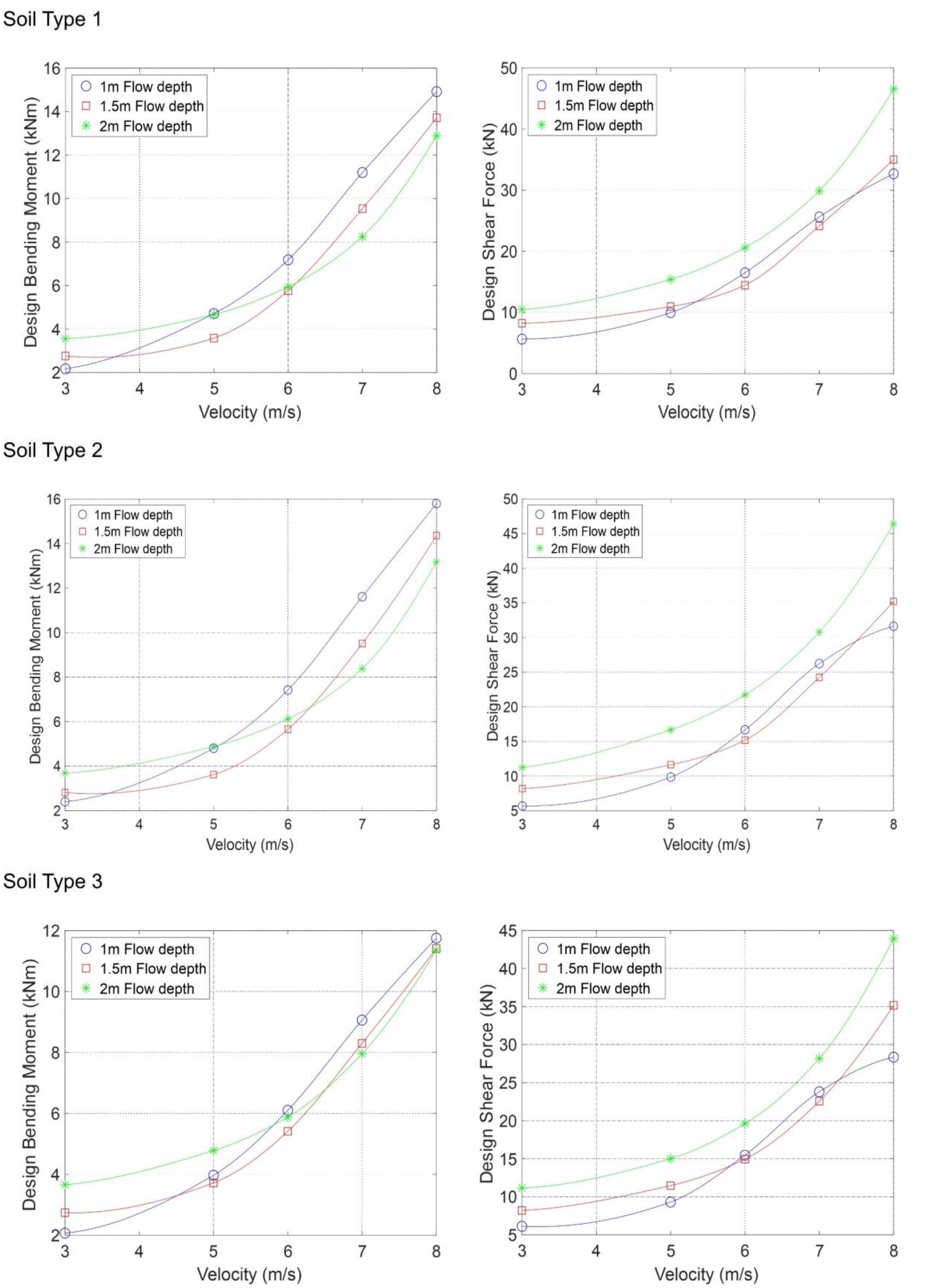

FIGURE 18. Floodway Type 5 structural design charts.

6. RESULTS

This section outlines the numerical simulation results obtained. The degree of agreement between the numerical solutions and industry survey results (Greene et al. 2020b) is also provided. The analysis relates to the trafficable concrete floodway structure.

6.1 Static Behaviour of the Floodways

Initially, the static behaviour of the floodway types was observed under the worst-case loading scenario. The maximum loading of 2 m flow depth and 8 m/s flow velocity produced the largest stress and deflection profiles enabling visual observation of deflection shape and magnitude at an exaggeration of 5% to be made.

The highest horizontal stress profile occurred centrally at the point of boulder impact for all floodway types (Figures 19 to 22). Displacement was also greatest in the horizontal x-axis direction, in the same global axis direction as the resulting impact loading from the boulder. Both the upstream and downstream cut-off walls on all four cases were deflected in the negative x-axis direction as they act to distribute the loading to the adjoining in-situ soil, providing a stabilising moment for the concrete floodway structure and a resistance to overturning. The distributed hydrostatic load, which was applied perpendicular to the floodway surface, produced a downwards deflection. In the case of the floodway Type 4 and 5 models, the hydrostatic pressure also contributed to the horizontal stress and deflection profile due to the upstream batters.

Investigation into the failure limits of the four numerical models shows that the maximum compressive stress was well below the maximum compressive strength of concrete of 32 MPa. In the case of maximum tensile stress, all four cases exceeded the maximum flexural tensile strength of concrete of 3.39 MPa. The area of exceedance is localised to the point of load application and in practise would be characterised by localised cracking within the outer face of the upstream cut-off wall and extending to the upstream potion of the apron (based on the stress contouring). On investigation the inside concrete face across all Floodway Types was below the maximum flexural tensile strength of concrete.

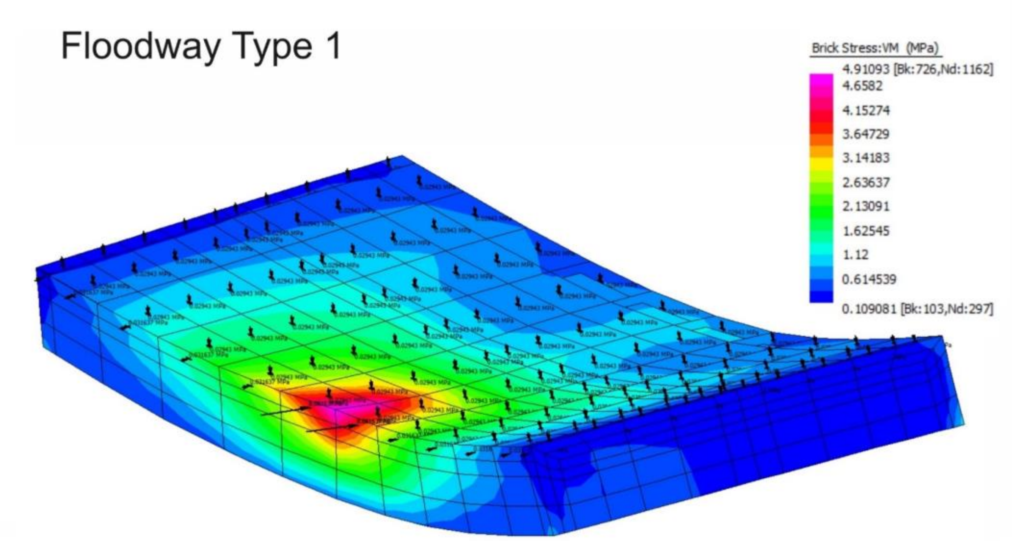

FIGURE 19. Floodway Type 1 static Von Mises stress behavior.

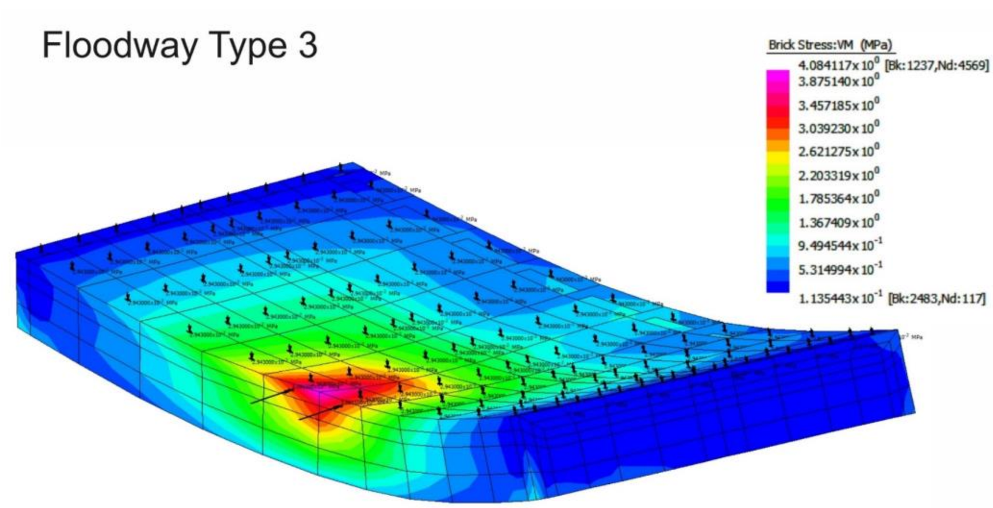

FIGURE 20. Floodway Type 3 static Von Mises stress behavior.

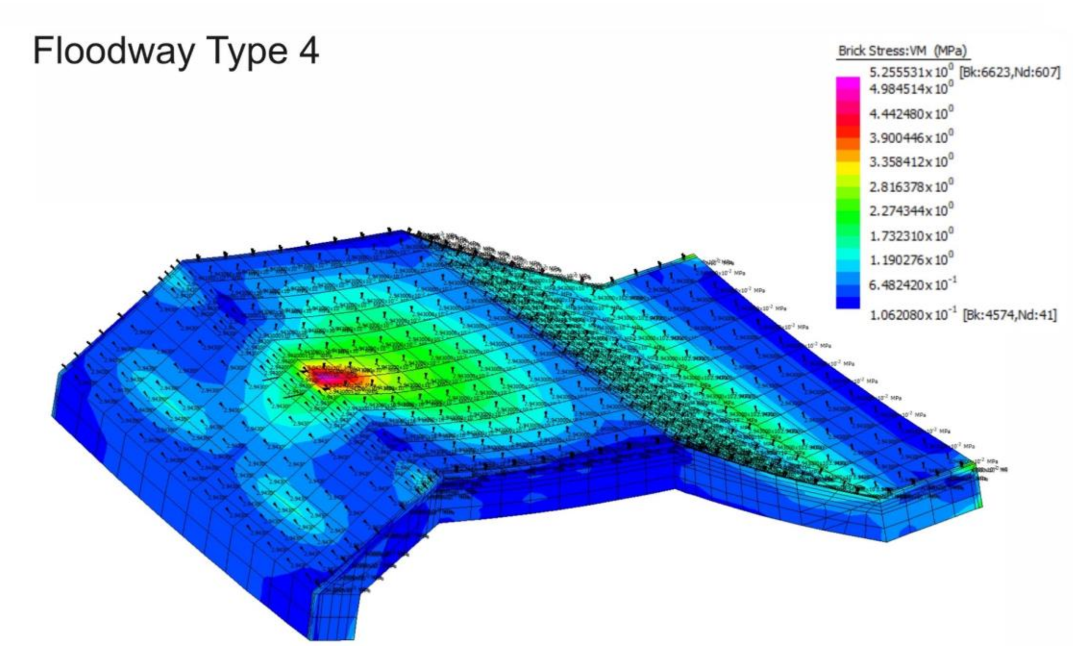

FIGURE 21. Floodway Type 4 static Von Mises stress behavior.

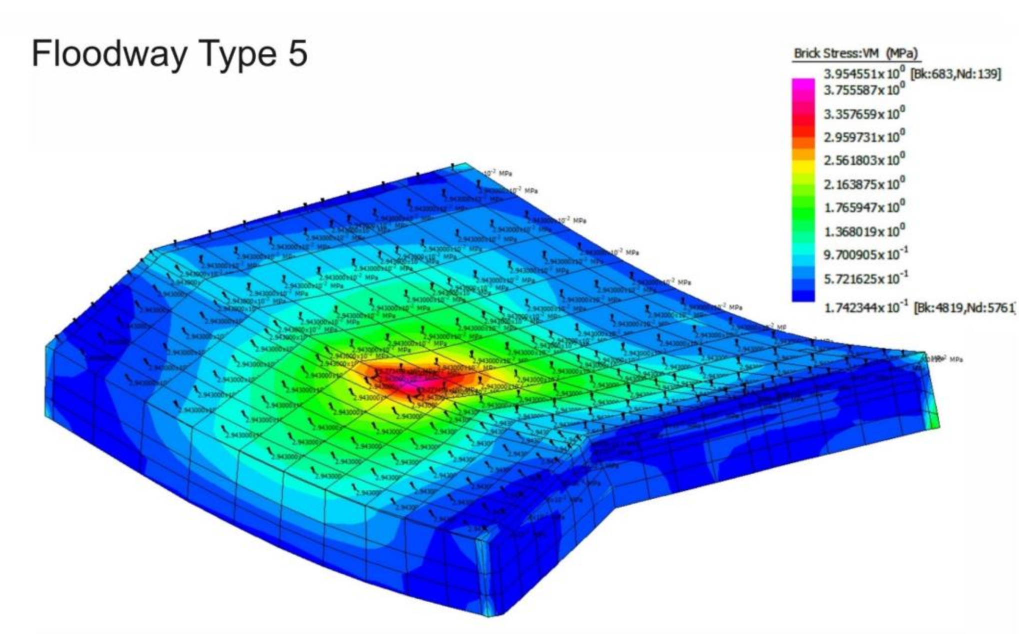

FIGURE 22. Floodway Type 5 static Von Mises stress behavior.

6.2 *Variability Due to Flow Velocity*

Figure 23 illustrates the variability of Von Mises stress and displacement profiles in the direction of the boulder impact loading (xx-section) for a variable flow velocity and a constant flow depth (2 m). Type 4, a raised floodway structure, exhibits the largest Von Mises stresses, with a maximum Von Mises stress of 5.26 MPa. Horizontal displacement for all four floodway types was similar, ranging from 2.85 mm (Type 1) to 2.77 mm (Type 3). Type 1 and Type 3 are the simplest concrete flood structure formations, consisting of an apron and perimeter cut-off wall, and do not feature concrete batters like Floodway Types 4 and 5. As a result, there is less surface area and thus less lateral resistance to oppose horizontal loading in these structures.

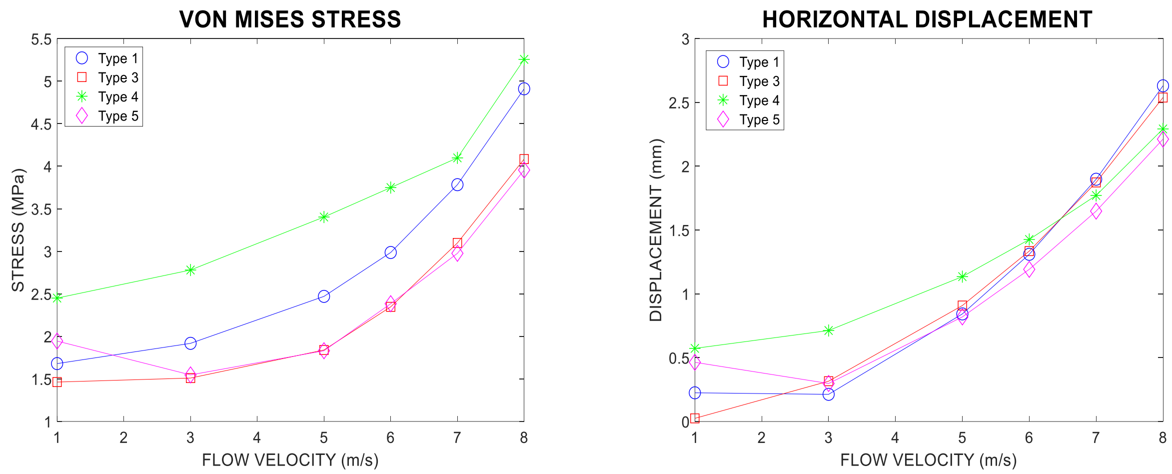

FIGURE 23. Comparison of varying flow velocity on stress and displacement results.

6.3 *Variability Due to Flow Depth*

Figure 24 illustrates the variability of Von Mises stress and displacement profiles in the direction of boulder impact loading (xx-section) for a variable flow depth and a constant flow velocity (8 m/s). Like the changing flow velocity scenario, Floodway Type 4 exhibits the highest Von Mises stresses across all flow depths, and stresses slightly increased with flow depth for all floodway types, with floodway Type 4 being the exception. Oppositely, horizontal displacement slightly decreased with increased flow depth for all cases except for Floodway Type 5. This is due to the upstream batter of Floodway Type 5 having a resultant component of hydrostatic force that acts in the same direction of the boulder impact force, thus resulting in an increase in displacement as flow depth increases. Comparing the two scenarios, that is, variability due to flow velocity (Figure 23) and flow depth (Figure 24), it is evident that flow velocity is the controlling parameter within the loading scenario applied.

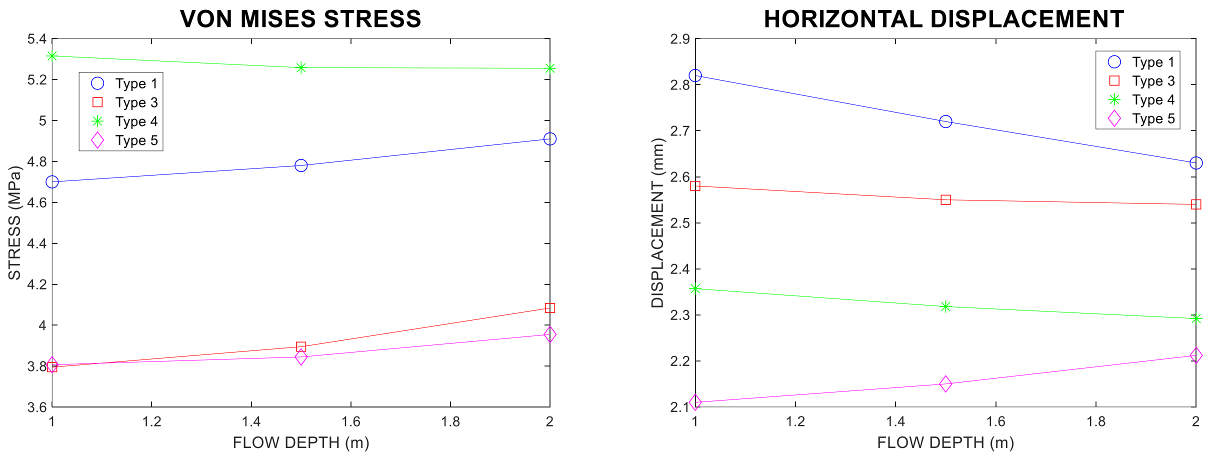

FIGURE 24. Comparison of varying flow depth on stress and displacement results.

6.4 Effect of Raising Floodways

Floodway Type 3 and 4 are both raised floodway structures, however in the case of floodway Type 3 no element of the concrete floodway structure is exposed or acts perpendicular to the direction of flow, rather, it is protected by the upstream rock protection and in-situ soil extent. This protection buffers the concrete structure from the direct stresses associated with boulder impact loading. Oppositely, floodway Type 4 is exposed to the direction of flow and is directly affected by the boulder impact loading. The elevated stresses in comparison to the other floodway types can be observed in

Figures 23 and 24. This result has qualitative agreement with an industry survey of floodway asset owners undertaken by Greene et al. (2020b), which reported that 87.5% of respondents stated that raised floodway structures were more susceptible to failure than level structures.

6.5 Variability Due to Soil Type

The effect of varying soil type on stresses and displacements was investigated and is illustrated in Figure 25. The soil type properties were previously outlined in Table 1 and are briefly described as follows:

- Soil Type 1 – a silty sand material.
- Soil Type 2 – a sandy soil consisting of poorly graded sands with fines.
- Soil Type 3 – a clay soil material.

For all floodway types trialled, Soil Type 2 resulted in the highest Von Mises stresses and horizontal displacement results. It is also highlighted that the variability of soil mechanical properties significantly changes the resulting stress and displacement results. This result has qualitative agreement with an industry survey of floodway asset owners undertaken by Greene et al. (2020b), who reported that floodway failure was 78.1% more noticeable in certain soil types, with a sandy soil type receiving the most responses at 56%. This is predominately due to the absence of cohesion, reducing the ability of the soil to resist lateral loadings compared to clay soils, such as Soil Type 3. Furthermore, this is supported by design guidelines which state that the placement of floodway crossings on scour susceptible fine-grained soils shall be avoided (Keller & Sherar, 2003).

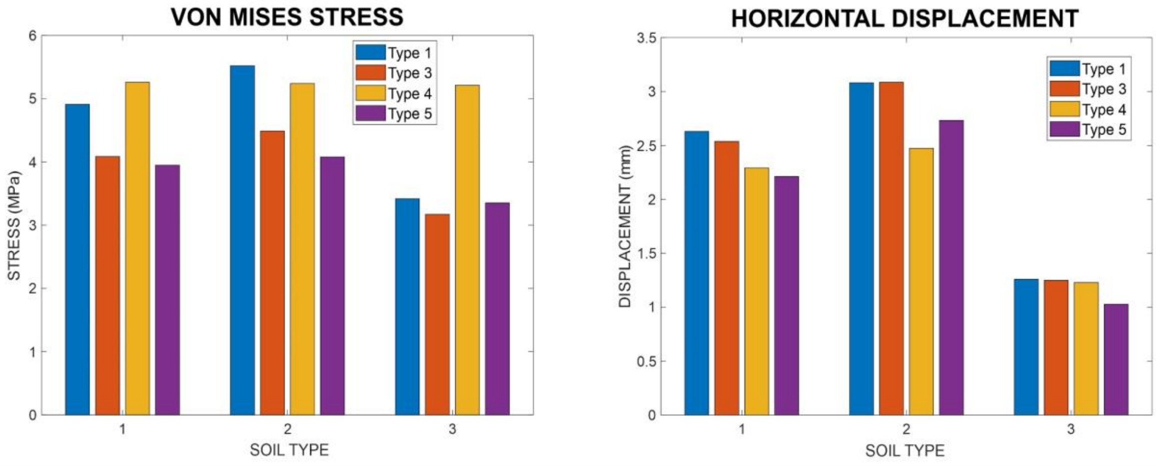

FIGURE 25. Comparison of varying soil type on stress and displacement relationships.

7. CONCLUSION

This study deduced and reported a conclusive numerical analysis procedure for the simulation of floodway structures under extreme flood loadings after the modelling of four standard engineering floodway structures. This procedure also provides a methodology to derive bending moment and shear force values from the finite element models at the point of concern, which can be collated into design charts based on floodway type for the subsequent structural design of the structure.

A comparison of stress and displacement results across the four floodway cases was also conducted. For all floodway types the highest stresses occurred at the point of impact, and magnitude was found to be directly proportional to the flow velocity and depth. Floodway Type 4, a raised floodway structure was found to have elevated stresses compared to the other floodway types because of being directly exposed (perpendicular) to the direction of flow. This also had a qualitative agreement with industry survey results that stated this type of structure was most susceptible to failure. Furthermore, soil mechanical properties significantly changed the resulting stress and displacement results, with Soil Type 2, a sandy soil type having the most elevated results. This also aligned with industry survey results and literature, which stated floodways situated in sandy soils were more prone to failure due to the absence of cohesion and reduced ability to resist lateral loadings.

By applying the numerical analysis procedure across four floodway types, an increased scope of knowledge pertaining to the numerical simulation and investigation of floodway resilience was achieved. This analysis also provided data to create design charts for an additional four standard engineering floodway types. These charts provide the ability to select design inputs for ultimate bending moment and shear force, enabling the accurate structural design of floodways under extreme flood loading to be undertaken. This increased level of design input will improve structural integrity and resilience against flooding to that provided in current design guidelines. Subsequently, local communities that rely upon these structures will have reduced social and economic impacts.

DISCLOSURE STATEMENT

No potential conflict of interest.

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CHAPTER 6

FLOODWAY DESIGN GUIDELINE

Publication V: Floodway Design Guideline

Greene I, Lokuge W & Karunasena W 2021, "Floodway Design Guideline", *Bushfire & Natural Hazard Cooperative Research Centre (BNHCRC) and Institute of Public Works Engineering Australasia, Queensland Division (IPWEAQ).* Submitted - November 2020, Currently Under Review.

For continuity purposes an extract of the above preliminary manuscript is located in Appendix D. This chapter contains an overview of the manuscript only.

Peer review undertaken by subject matter experts for this manuscript to date is as follows:

1. First review: IPWEAQ - Completed.
2. Review of hydraulic: Pitt & Sherry Pty Ltd - Completed.
3. Second review: IPWEAQ - Completed.

4. Review of structural: BECA Pty Ltd - On-going.

5. Third review: IPWEAQ - On-going.

Authorship Contribution Statement

The contribution of Isaac Greene (Candidate) was 70%. Isaac undertook formalisation of ideas and layout, drafting, revising and finalisation of the manuscript. Assoc. Prof. Weena Lokuge (Principal Supervisor) and Prof. Warna Karunasena (Supervisor) contributed to the formalisation of ideas, technical input and editing/co-authoring of the manuscript. These contributions were 15% and 15% respectively.

6.1 Design Guideline Overview

The design guideline collates the research outcomes of prior investigations and provides a practical end-user outcome. This floodway design guideline has an increased focus on ensuring floodways are structurally adequate to withstand flood-related loadings whilst in a submerged state. The guideline utilises a structural model to predict failure during a worst-case flow and an associated boulder impact load, as well as the design methodology deduced in Chapters 4 and 5. The floodway design guideline steps the user through all stages of floodway design, including preparatory works, floodway selection and hydraulic, protection and structural design. The floodway design process flowchart is provided in Figure 6.1

The following sub-sections provide a summary of the design guideline. The full design guideline is located in Appendix D.

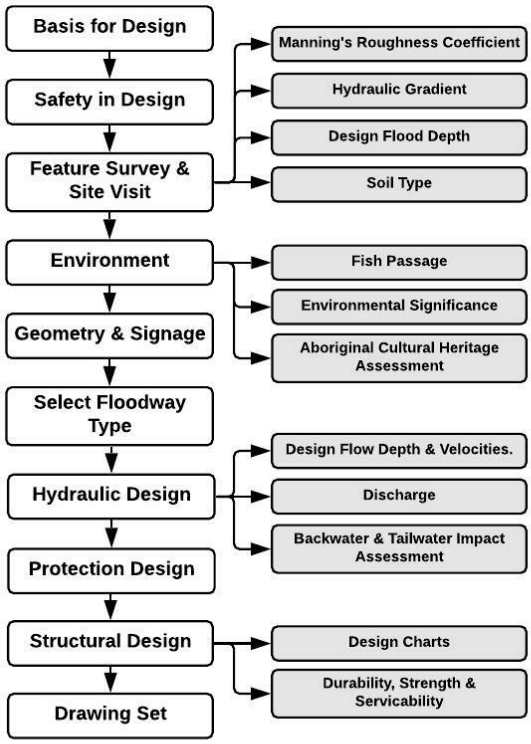

Figure 6.1: Floodway design process flowchart.

6.1.1 Preparatory Works

The design guideline provides input into the following preliminary design considerations:

1. Basis for Design - summarises the primary assumptions, rationale, design criteria and considerations for the project. In a floodway context this includes the acceptable closure periods, maintenance periods, design rainfall events and the required level of service.

2. Safety in Design - Discusses relevant legislative requirements and guidelines for the identification, management and mitigation of safety risks and hazards within floodway design.

3. Feature Survey and Site Visit - Details the survey data in the form of a feature survey required to develop a detailed understanding of the

topography and geometrical characteristics of unrestricted waterways. This data is then utilised in the hydraulic analysis to calculate the channel cross-sectional area, hydraulic gradient, geometric alignment, peak flow analysis and for the preparation of design drawings. The design guideline also details the relevant field data required, including Manning's coefficient of roughness, hydraulic gradient, design flood depth and soil type.

4. Environment - The design guideline explains that the implementation of a floodway crossing within a waterway has several environmental sensitivities that need to be well managed during design to avoid environmental degradation. The design guideline summarises the need of a Review of Environmental Factors (REF), which includes a flora and fauna biodiversity assessment, fish passage considerations and aboriginal cultural heritage assessment.

6.1.2 Geometry and Signage

Geometry - The design guideline provides geometric considerations for new floodway constructions where flexibility of geometric alignment exist. The considerations provided act to reduce the likelihood of undue deterioration of the floodway superstructure and to increase vehicle crossing safety when navigating the floodway structure.

Signage - The design guideline provides a summary of signage requirements for floodways based on Australian Standard AS1742.2:2009, including figures of real-life signage applications.

6.1.3 Floodway Type Selection

The five standard engineering floodway drawings detailed within the design guideline are those analysed within this research. Through the numerical analysis conducted the design considerations for optimal floodway selection have been detailed, as well as a floodway selection matrix to assist in the selection of a standard engineering floodway type based upon watercourse hydraulic characteristics. These standard engineering drawings provide typical detail for generalised situations and include floodways that are both raised and level in relation to the channel.

6.1.4 Hydraulic Design

The hydraulic design within the design guideline takes on four main sections, design discharge, hydraulic design, flow analysis, and tailwater and backwater analysis.

Design Discharge - Hydrological assessment of the contributing catchment is utilised to determine two design discharge values, that is, the serviceability design discharge value and the structural design discharge value. The serviceability design discharge value defines the rainfall event as an ARI which the road must remain trafficable and the structural design discharge value defines the rainfall event as an ARI which the floodway must structurally be able to withstand. The design guideline details how to determine this through a hydrological assessment of the contributing catchment, with methods adopted based upon the Queensland Urban Drainage Manual (QUDM) (IPWEAQ, 2016). The development of a stage discharge curve is then detailed to determine the relationship of flow versus depth at a particular cross-section within the unrestricted waterway. This curve enables the following basic flow characteristics to be determined:

1. Maximum flow capacity of the channel.

2. Velocity-discharge relationship.

3. Depth and velocity of flow for a specific discharge.

Hydraulic Design - Hydraulic design within the design guideline is based upon the hydraulic techniques broadly adopted in traditional design guidelines (Department of Transport Main Roads, 2010; Main Roads Western Australia, 2006; Austroads, 2013). These techniques seek to determine the height of flow over the trafficable floodway surface for the chosen serviceability design discharge rainfall event to determine if the floodway geometry satisfies the maximum permittable crossing depth of 300 mm as specified in Main Roads Western Australia (2006).

For floodways constructed level with the channel, and therefore do not cause a hydraulic control on the waterway, Mannings formula is prescribed to calculate the height of flow over the floodway. This is done by referencing the stage-discharge curve of the unrestricted channel for the serviceability design discharge value.

For floodways that are raised in relation to the channel, and are in a submerged state, the Empirical-Broad Crested Weir formula is prescribed for use. Once the floodway is in a submerged state it can be analysed as an open channel (Manning's formula) with no hydraulic control.

Flow Analysis - The design guideline acknowledges that the construction of a floodway increases the likelihood of supercritical flow conditions through the introduction of either steep bed gradients or constrictions. As a result of this the development of supercritical conditions need to be controlled through engineering design to ensure adequate protection against scour exists, particularly at the point where supercritical flows revert to subcritical flows. The design guideline provides examples on how to use hydraulic computational methods from Hydrological Engineering Centre River Analysis System (HEC-RAS) to analyse flow over both raised and level floodway constructions (U.S. Army Corps of Engineers Hydrologic Engineering Center, 2019). This method enables supercritical flow locations to be determined so scour protection can be provided to create a non-erodible channel bed at interfaces.

Tailwater and Backwater Analysis - The design guideline details that the tailwater level provides valuable input into determining the flow regime over a raised floodway structure due to its influence on rate and height of discharge. The design guideline determines tailwater level through the use of Manning's equation or the stage-discharge curve for uniform flow conditions within regular shaped channels.

The design guideline also details that the increase in backwater level, if not managed can have a significant effect on upstream assets, land use and infrastructure. The design guideline provides examples of how to use hydraulic channel modeling software to determine water surface profiles for both tailwater and backwater level. The design guideline explains this as the simplest and most reliable method to calculate backwater level and enables a backwater versus discharge curve to be defined based on a user specified range of design rainfall events. Traditional design guidelines, particularly Main Roads Western Australia (2006) details an iterative approach based on hand calculations to calculate backwater level (based on Manning's equation); however, this method is lengthy due to its reliance on a trial-and-error based approach.

6.1.5 Scour Protection Design

Within the industry survey documented in Chapter 3, downstream rock protection was reported as the most susceptible component to fail during extreme flood events. The floodway design guideline provides a procedure, inclusive of design tables for the selection of scour protection materials based upon the maximum permissible velocity. The design guideline also summaries the different forms of rock protection available for use in floodway design, which includes identification of advantages and disadvantages for each type, along with critical locations for scour protection placement. Extending the current knowledge base further, the design guideline introduces the reader to hydraulic computational modelling, including a step by step guide to determine the critical velocity at a specific location, enabling greater accuracy within scour protection selection and design processes.

6.1.6 Structural Design Charts and Method

To achieve a flood-resilient design, the concrete floodway superstructure needs to be designed to sustain ultimate flood loading determined from the peak flow analysis and as experienced during extreme flood events. As determined in the gap analysis within the literature review, current design guidelines neglect the significant forces present during extreme flood events, and rather focus on hydraulic design principals and mitigating hydraulic related failure due to erosion and scour. The empirical investigations within this research have significantly contributed to knowledge and allowing frameworks and methodologies to be determined, thus enabling floodways to be structurally designed in a matter that increases structural resilience and immunity against extreme flood events. The design guideline presents a structural design methodology that utilises the design charts deduced through the research, which are based on the critical section to determine steel reinforcement requirements which satisfy strength, serviceability, and durability requirements. The critical section was determined based on the experimental investigations and the numerical analysis presented in this research, highlighting the worst case loading scenario and the presence of significant bending moments and shear forces within floodway structures.

The design guideline details the structural design method required to select the cross-sectional area of steel and concrete that satisfies requirements for strength, serviceability, durability and fatigue in accordance with the requirements of AS 5100.5. This includes detailed commentary and explanation of calculations based on the applicable clauses in AS 5100.5.

6.1.7 Design Drawing Set Considerations

The design guideline details the requirements for the final civil and structural drawing sets. This is based on the outcomes deduced within each section of the design process. It also provides the typical format of the drawing set.

6.1.8 Examples

The following comprehensive worked examples are provided within the design guideline to guide users through the various design stages of a floodway.

1. Stage-Discharge Curve - Provides steps to develop a stage-discharge curve including the velocity-discharge relationship for a watercourse.
2. Hydraulic modelling using HEC-RAS - Provides steps to complete hydraulic computational modelling of a trapezoidal creek profile with a 1 m raised floodway structure incorporated.
3. Structural Design - provides steps to determine the cross-sectional area of steel and concrete that is required to satisfy strength, serviceability and durability criteria.
4. Design of a Floodway - provides complete design steps for a raised floodway structure type.

CHAPTER 7

CONCLUSIONS

Through the course of research it was discovered that Australia relies heavily upon small road structures such as floodways to service its vast rural road network. After undertaking a review of literature it was discovered that research into floodway design was limited, focused primarily on hydraulic design considerations and subsequently, repeat structural damage and consistent failure mechanisms had been observed.

This research sought to provide a detailed investigation into the current application of floodways, and deduced a design methodology in the form of a guideline to improve structural resilience against extreme flood events. To develop the floodway design guideline the field of knowledge was investigated and expanded through a survey instrument, finite element analysis investigations and an experimental program. These investigations have led to a thorough methodology for the analysis of floodway structures, through the use of finite element models and structural design procedures.

7.1 Project Outcomes

Outlined below are specific summaries pertaining to the research outcomes achieved.

Industry Survey and Experimental Investigations (Chapter 3)

The responses received within the industry survey supported the findings of the review of literature, provided general industry consensus and also further defined the issues being experienced in practice enabling overall research outcomes to be targeted. Through this survey, areas of design concern were exposed in the context of extreme flood events, which included downstream floodway components (rock protection, cut-off wall and batter), raised floodway structures, soils that lack cohesion or that are dispersive and load cases associated with debris impact, such as boulders. The results from this survey and the subsequent published article also provided a significant contribution to the current knowledge base pertaining to experiences, observations and challenges faced by subject matter experts in the Australian context.

An experimental program was also developed using a scaled model test specimen, a soil box and a centrally placed horizontal load. Results were then compared to those obtained from a numerical model and the industry survey. Qualitatively, it was discovered that the crack propagation and displacement results correlated closely to the strain concentrations and displacements identified within the numerical model. It was also discovered that several different failure modes exist, including structural concrete failure, yielding of adjoining soil material causing displacement and hydraulic failure through scour and erosion in the downstream vicinity.

Numerical Simulation of a Floodway (Chapter 4)

An initial methodology to numerically model a commonly used standard engineering floodway type with a load case equivalent to that experienced during extreme flood events was deduced. Investigation using the derived finite element model enabled vulnerabilities and geometric features to be analysed and the worst case loading scenario determined. The worst case loading scenario was discovered to occur under a 4-tonne boulder impact loading case, no downstream rock protection and with a 900 mm cut-off wall depth. Structural design charts relating to the bending moment and shear force under a range of flow conditions was also developed, allowing steel and concrete to be designed accordingly.

Applying Analysis Techniques to Several Floodway Types (Chapter 5)

Chapter 5 applied the finite element modeling techniques from Chapter 4 to an additional four commonly used standard engineering floodway types with structural and hydraulic variation. The primary outcome was the refinement of the numerical analysis methodology for floodways enabling an increased level of design input and a complete set of commonly used standard engineering floodway types with subsequent design charts. This set of commonly used standard engineering floodway types forms the basis of the Floodway Design Guideline.

Floodway Design Guideline (Chapter 6)

The Floodway Design Guideline formed the practical end-user outcome for the research and collated the outcomes from the research project and subsequent investigations. The guideline utilises a structural model to predict failure during a worst case flow and an associated boulder impact load. The floodway design guideline steps the designer through all stages of floodway design including preparatory works, floodway selection and design for hydraulic, protection and structural.

7.2 Contribution to Knowledge

The outcomes of this research have provided significant empirical investigation into floodways from a flood resilience perspective. The survey and experimental investigations have collated qualitative and quantitative findings to enable areas of floodway vulnerability, failure mechanisms and the experiences and observations of industry experts to be defined. This enabling verification of modelling techniques and criterion's applied within the subsequent numerical simulation to be made, but also provides initial academic studies for validation of future research.

Outcomes from the extensive numerical investigations deduced the worst case loading scenario for floodways enabling structural design efforts to be concentrated, and frameworks to mitigate floodway vulnerability developed. Based on these investigations structural design charts relating to the bending moment and shear force values of floodways under a range of flow conditions were developed, thus allowing steel reinforcement and concrete to be designed in a flood resilient manner.

The most significant contribution to knowledge pertaining to the outcomes of this research was the incorporation of the deduced structural design method and outcomes into the current hydraulic design framework from the current design guidelines. Through the implementation of the design process, floodway structural resilience and immunity against extreme flood loadings will improve. As floodways serve a critical function in the connection of rural communities, it is expected that time of closure and disruption post extreme flood events will be reduced. This will also have positive impacts for road authorities such as, reduced maintenance requirements and costs, and ensuring compliance with required service levels are achieved.

7.3 Further Research Work

Further research work in context of the floodway design process includes:

1. Further studies into an appropriate experimental test program to achieve

quantitative agreement and model convergence with numerical analysis results. Although qualitative agreement was achieved within this research, further work into the revision of scaled dimensions used within the test specimens is required to ensure accurate casting and demoulding can be achieved. Further, the location of strain gauges needs to be carefully considered to alleviate the sensitivity and complexity in accurately recording strain.

2. Continuous improvement and revision to the floodway design guideline is required to incorporate further advancements in the field of knowledge. This may include further research work into the design of several other floodway types, or the construction of a full-scale floodway, which complies with the specified design procedure along with relevant performance monitoring.

3. A study into the application of the design methodology for other small road structures such as culverts. Culverts are road structures that experience very similar loading to floodways during extreme flood events, and therefore the design methodology explored within this research may be relevant to culvert structures also.

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APPENDIX A

CONFERENCE PRESENTATIONS & POSTERS

A.1 Conference Presentation

Greene I 2021, "Floodway Behaviour During Extreme Flood Events", *Queensland Disaster Management Research Forum* 2021, Brisbane, Presented July 2021.

Abstract: Floodways, due to being in a frequently submerged state are extremely sensitive to flood events with downstream components often sustaining damage. This research focuses on obtaining improvements to floodway structural design to lower asset damage after a flood event, thus directly increasing the accessibility and serviceability of rural communities. To derive these outcomes, a survey seeking floodway asset owners' practical experiences was conducted, an experimental laboratory program was investigated and computational numerical modelling and simulation was performed. A floodway design guideline was deduced and forms the practical outcome of this research. This design guideline will enable increased floodway structural resilience to be achieved through structural design charts and an associated design methodology for a range of watercourse applications.

A.2 Conference Poster

Conference poster submission relating to the BNHCRC project "Enhancing resilience of critical road infrastructure: bridges, culverts and flood ways under natural hazards", *Australasian Fire Authorities Council 2019* (AFAC19), Melbourne - Displayed 27-30 August 2019

NUMERICAL INVESTIGATION INTO THE BEHAVIOUR OF FLOODWAYS DURING EXTREME FLOOD EVENTS

Isaac Greene, Weena Lokuge & Karu Karunasena

Centre for Future Materials, University of Southern Queensland, QLD | Contact: Isaac.greene@usq.edu.au Phone: 0439 863 740

Australian floodway design guidelines exclusively consider hydraulic principles. Comprehensive analysis of floodway failure mechanisms has exposed deficiencies demanding the need to investigate the structural adequacy of floodways to enhance resilience during extreme flood events.

RESEARCH INTEREST

The severity of flood events in the Lockyer Valley Region has become more prevalent in recent years causing catastrophic failure to floodway superstructures. A recent event which had devastating consequences on the built environment causing loss of life was the Lockyer Valley floods of January 2011.

RESEARCH AIMS

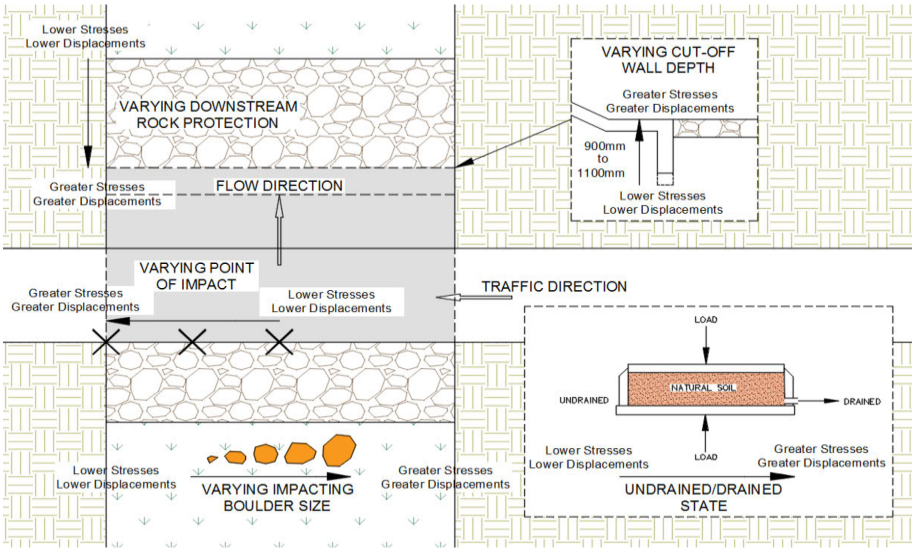
- Review the geometric and loading factors contributing to floodway failure.
- Develop a design process that satisfies strength and serviceability requirements for floodways under extreme flood loading.
- Integrate the proposed structural analysis outcomes into the current hydraulic design methods recommended by Australian design guidelines.

METHODOLOGY

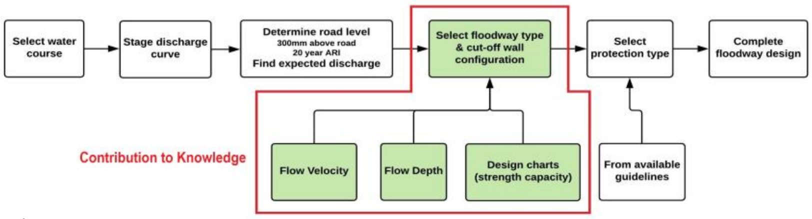
1. Select a range of standard engineering floodway types.
2. Construct detailed 3-dimensional finite element models of the floodway types.
3. Perform parametric finite element analysis to determine loading combination and geometric factors that cause the floodway types to be most susceptible to damage.
4. Develop design charts for design bending moment (M*) and design shear force (V*).

RESEARCH FINDINGS & OUTCOMES

Impact loading applied in reference to AS 5100.2:2017 consistently yielded the highest stress and displacement results for all loading cases.

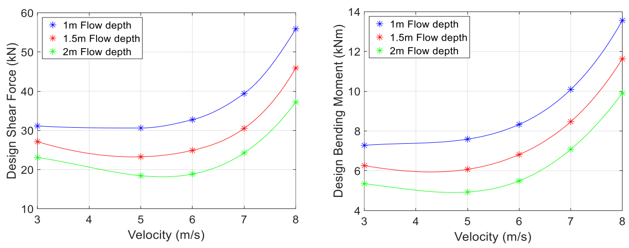


Integrated Design Procedure



Design Charts

Strength capacity charts assembled based on the worst case loading combination. M* and V* values can be referenced for different soil types, flow velocities and depths enabling accurate design of structural elements in reference to AS 3600.



CONCLUSION

Current floodway design presents structural vulnerability for a number of AS 5100.2:2017 flood loadings.

This research, which remains ongoing seeks to present a finite element modelling approach to improve floodway resilience through a simplified structural design method and provide a procedure to integrate structural analysis outcomes into current hydraulic design processes.

APPENDIX B

COPYRIGHT INFORMATION

Publication I - Copyright Information:

Greene I, Lokuge W & Karunasena W 2020, “Floodways and Flood-Related Experiences: A Survey of Industry Experts and Asset Owners”, *Engineering for Public Works*, Vol. 19, pp. 69-75. ISSN 2652-6050. Published - September 2020.

Re: Request to Include Article in Thesis

Isaac Greene <Isaac.Greene@usq.edu.au>
Mon 12/6/2021 10:45 AM
To: Isaac Greene <Isaac.Greene@usq.edu.au>

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Lexy Rohde
Business Support Manager
07 3632 6825
0422 871 625
Level 1, 6 Eagleview Place
Eagle Farm QLD 4009
Lexy.Rohde@ipweaq.com
www.ipweaq.com
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From: Isaac Greene <Isaac.Greene@usq.edu.au>
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To: Lexy Rohde <Lexy.Rohde@ipweaq.com>
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Dear Lexy,

My name is Isaac Greene and I am studying a Master of Engineering Research at the University of Southern Queensland (USQ).

I am the lead author of the following article: "Floodways and Flood Related Experiences: A Survey of Industry

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0439 863 740

Publication III - Copyright Information:

Greene I, Lokuge W & Karunasena W 2020, “Structural Design of Floodways Under Extreme Flood Loading”, *International Journal of Disaster Resilience in the Built Environment*, Vol. 11 No. 4, pp. 535-555. https://doi.org/10.1108/IJDRBE-10-2019-0072. Published - 9 May 2020 (Q1).

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APPENDIX C

SURVEY INSTRUMENT & ETHICS APPROVAL

Questionnaire to Investigate the Behaviour of Floodways Under Extreme Flood Loadings

Project Details

Title of Project: Questionnaire to investigate the behaviour of floodways under extreme flood loadings

Human Research Ethics Approval Number: H20REA145

Description

This research aims to improve the resilience of floodway structures during extreme flood events. The research involves three-dimensional modelling of floodway structures under different extreme loading conditions, with outcomes contributing to the development of a structural design method to compliment the hydraulic design approaches utilised in the current Australian design guidelines.

The researcher has requested your assistance because you are an individual with current or previous experience in floodway design, maintenance and construction. Your input into this survey will assist in validating model results and in aligning research outcomes with industry requirements.

The overarching research project, titled “Investigation on the Behaviour of Small Road Structures Under Extreme Flood Loadings” is being undertaken as part of a Master of Engineering (Research) Project in partnership with the Bushfire and Natural Hazard Cooperative Research Centre (BNHCRC).

Participation

Your participation will involve the completion of an online questionnaire that will take approximately 15 minutes of your time. You may choose any time to complete the questionnaire.

Theme of questions

A sample of the theme of questions is as follows:

- Which floodway components are most likely to sustain damage during extreme flood events? and
- From your experience, what failure mechanisms typically cause floodways to fail.

Your participation in this project is entirely voluntary and your response will not be identifiable. If you do not wish to take part, you are not obliged to. If you decide to take part and later change your mind, you are free to withdraw, close the survey (do not complete it) and your responses will not be included in the research. However, once you have completed the questionnaire, the Principal Investigator will be unable to remove your data from the dataset because responses are completely anonymous.

Expected Benefits

You may not derive direct benefit from completing this questionnaire. However, you may find that reflecting on your previous experiences in floodway failures, maintenance and construction is beneficial. This research will be published in a quality journal. This research may provide valuable insights and understanding of floodway failure, its prevalence and the common failure mechanisms observed in practice, which will be a valued contribution to the limited body of knowledge which exists on floodway structures. It is expected that industry, such as local and state government will benefit from the insights generated through this questionnaire in order to establish ways to improve the structural resilience of floodways during extreme flood events.

Risks

There are no anticipated risks beyond normal day-to-day living associated with your participation in this research. The questionnaire does not contain any distressing content.

Privacy and Confidentiality

All comments and responses will be treated confidentially and you will not be identifiable in the data. Names are not required in any of the responses. Outcomes of the survey will be disseminated to participants via a concise report containing a summary of results in a non-identifiable form. All data will be securely stored as per University of Southern Queensland's Research Data Management policy. Only the Research Team will have access to the data.

Funding

This project is funded by the Bushfire and Natural Hazards Cooperative Research Center (BNHCRC).

Consent to Participate

Clicking on the 'Submit' button at the conclusion of the questionnaire is accepted as an indication of your consent to participate in this research.

Questions or Further Information about the Project

Please contact the Principal Investigator (details at the bottom of this form) if you have any questions in relation to this research.

Concerns or Complaints Regarding the Conduct of the Project

If you have any concerns or complaints about the ethical conduct of this research, you may contact the University of Southern Queensland Ethics Coordinator on +61 7 4631 2690 or email ethics@usq.edu.au (mailto:ethics@usq.edu.au). The Ethics Coordinator is not connected with this research and can facilitate a resolution to your concern in an unbiased manner.

Contact Details

Principal Investigator

Mr. Isaac Greene

Email: Isaac.Greene@usq.edu.au (mailto:Isaac.Greene@usq.edu.au)

Mobile: +61 439 863 740

Research Supervisors

Dr. Weena Lokuge

Email: Weena.Lokuge@usq.edu.au (mailto:Weena.Lokuge@usq.edu.au)

Telephone: +61 7 3470 4477

Prof. Karu Karunasena

Email: Karu.Karunasena@usq.edu.au (mailto:Karu.Karunasena@usq.edu.au)

Telephone: +61 7 3470 4315

There are 12 questions in this survey.

Floodways Under Extreme Flood Loading

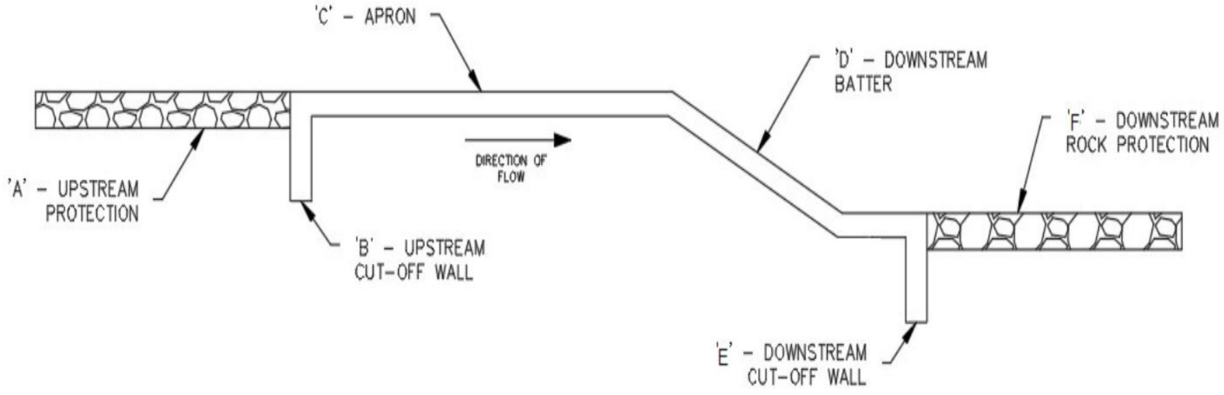
In your experience what is the likelihood that a floodway, inclusive of protection, will sustain damage during extreme flood events? *

Choose one of the following answers

Please choose **only one** of the following:

- Highly likely
- Likely
- Neither likely nor unlikely
- Unlikely
- Very unlikely

In your experience which floodway component is most susceptible to damage during an extreme flood event? What is the likely cause of this damage?



*

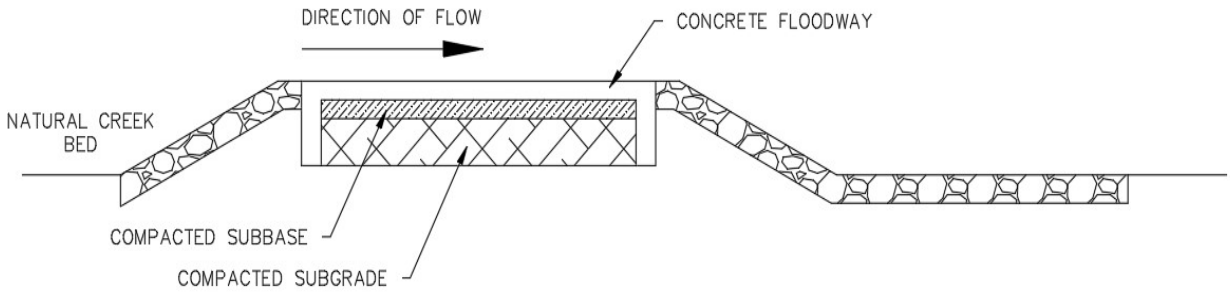
Choose one of the following answers
Please choose **only one** of the following:

- ◯ A - Upstream rock protection
- ◯ B - Upstream cut-off wall
- ◯ C - Apron
- ◯ D - Downstream batter
- ◯ E - Downstream cut-off wall
- ◯ F - Downstream rock protection

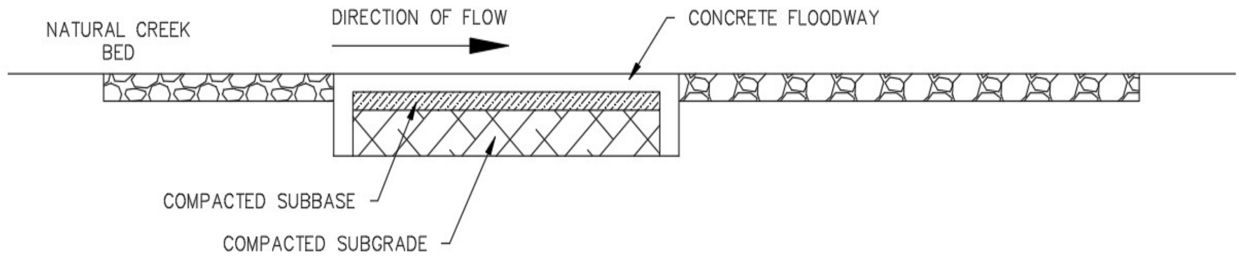
Make a comment on your choice here:

In your experience is floodway failure more common in raised floodway structures or floodway structures situated level with the creek bed?

Typical raised floodway structure:



Typical level floodway structure:



*

❗ Choose one of the following answers
Please choose **only one** of the following:

◯ Raised floodway structures
◯ Level floodway structures

Have you found floodway failure to be more common in certain soil types?

*

❗ Choose one of the following answers
Please choose **only one** of the following:

◯ Yes
◯ No

Which soil type have you found floodway failure to be most common in? *

Only answer this question if the following conditions are met:
Answer was 'Yes' at question '4 [Q4]' (Have you found floodway failure to be more common in certain soil types?)

Choose one of the following answers
Please choose **only one** of the following:

- ◯ Sandy soils
- ◯ Gravel Soils
- ◯ Silty soils
- ◯ Clay soils
- ◯ Other ______

During extreme flood events, have you found that increased sediment load, such as organic debris (logs) and boulders from landslides, bank erosion and other processes, contributed to floodway failure as a result of being conveyed by floodwaters and impacting the floodway structure?

*

Choose one of the following answers
Please choose **only one** of the following:

- ◯ Yes
- ◯ No

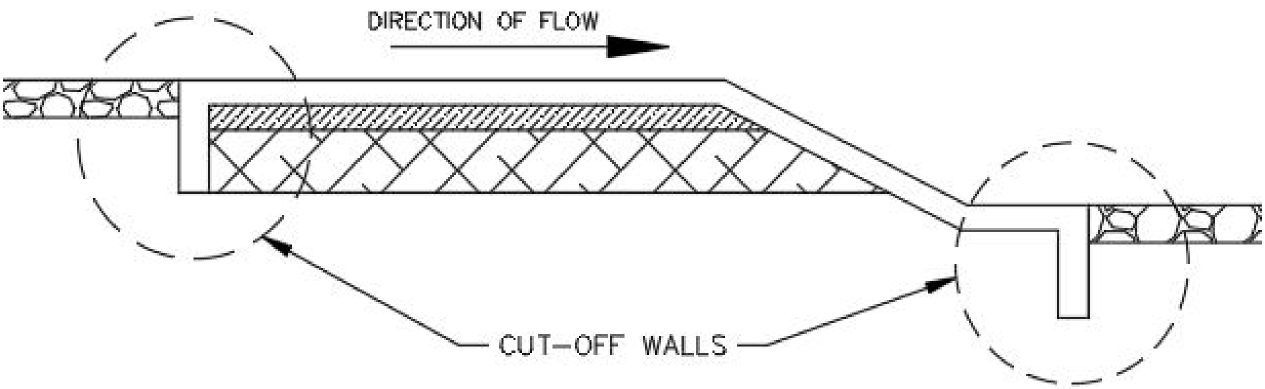
Has impact from boulders (large rocks) specifically contributed to these failures experienced? *

Only answer this question if the following conditions are met:
Answer was 'Yes' at question '6 [Q6]' (During extreme flood events, have you found that increased sediment load, such as organic debris (logs) and boulders from landslides, bank erosion and other processes, contributed to floodway failure as a result of being conveyed by floodwaters and impacting the floodway structure?)

❶ Choose one of the following answers
Please choose **only one** of the following:

- ◯ Yes
- ◯ No

Has your organisation completed any investigation into different concrete cut-off wall configurations (depth, width etc.)?

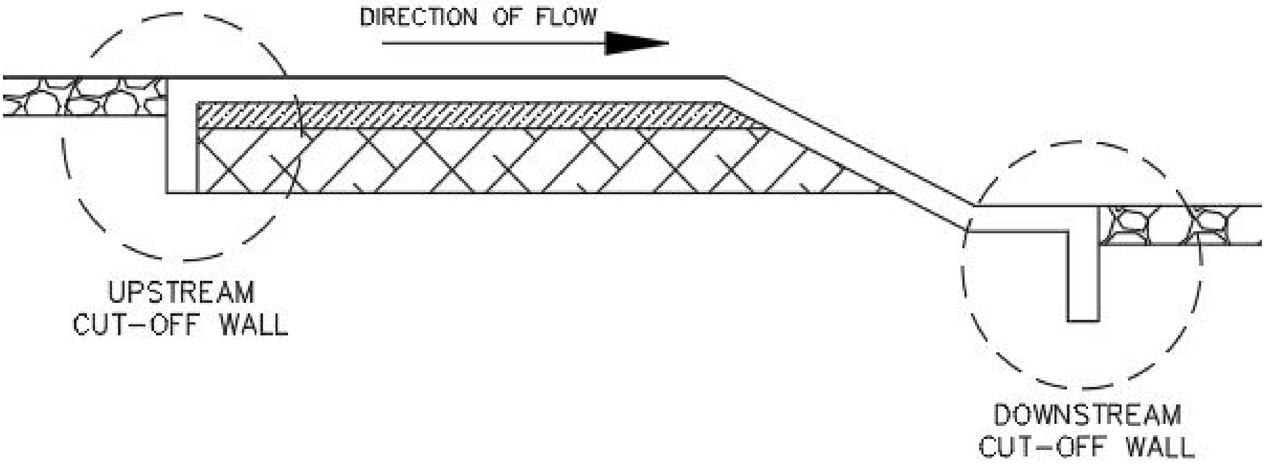


*

❶ Choose one of the following answers
Please choose **only one** of the following:

- ◯ Yes
- ◯ No

Which cut-off wall did these investigations specifically apply to?



*

Only answer this question if the following conditions are met:
Answer was 'Yes' at question '8 [Q8]' (Has your organisation completed any investigation into different concrete cut-off wall configurations (depth, width etc.)?)

Choose one of the following answers
Please choose **only one** of the following:

- ◯ Upstream cut-off wall
- ◯ Downstream cut-off wall
- ◯ Both upstream and downstream cut-off walls

What were the investigations carried out? and did these investigations lead to an increase in structural resilience?

Only answer this question if the following conditions are met:
Answer was 'Yes' at question '8 [Q8]' (Has your organisation completed any investigation into different concrete cut-off wall configurations (depth, width etc.)?)

Please write your answer here:

Has your organisation trialled any other improvements or amendments to floodway design to increase structural resilience against flood events? If so, please explain. *

Choose one of the following answers
Please choose **only one** of the following:

- Yes
- No

Make a comment on your choice here:

Is there any other feedback you wish to provide based on your experience in floodway construction and maintenance?

Please write your answer here:

Thank you for taking the time to complete this survey.
11.07.2021 – 15:40

Submit your survey.
Thank you for completing this survey.

[RIMS] USQ HRE Application - H20REA145 - Expedited review outcome -Approved

human.Ethics@usq.edu.au <human.Ethics@usq.edu.au>

Thu 7/23/2020 3:10 PM

To: Isaac Greene <Isaac.Greene@usq.edu.au>

Cc: Weena Lokuge <Weena.Lokuge@usq.edu.au>

Dear Isaac

I am pleased to confirm your Human Research Ethics (HRE) application has now been reviewed by the University's Expedited Review process. As your research proposal has been deemed to meet the requirements of the National Statement on Ethical Conduct in Human Research (2007), ethical approval is granted as follows:

USQ HREC ID: H20REA145
Project title: Questionnaire to investigate the behaviour of floodways under flood loadings
Approval date: 23/07/2020
Expiry date: 23/07/2023
USQ HREC status: Approved

The standard conditions of this approval are:

a) responsibly conduct the project strictly in accordance with the proposal submitted and granted ethics approval, including any amendments made to the proposal;.

(b) advise the University (email:ResearchIntegrity@usq.edu.au) immediately of any complaint pertaining to the conduct of the research or any other issues in relation to the project which may warrant review of the ethical approval of the project;

(c) promptly report any adverse events or unexpected outcomes to the University (email: ResearchIntegrity@usq.edu.au) and take prompt action to deal with any unexpected risks;

(d) make submission for any amendments to the project and obtain approval prior to implementing such changes;

(e) provide a progress 'milestone report' when requested and at least for every year of approval.

(f) provide a final 'milestone report' when the project is complete;

(g) promptly advise the University if the project has been discontinued, using a final 'milestone report'.

The additional conditionals of approval for this project are:

(a) Nil.

Please note that failure to comply with the conditions of this approval or requirements of the Australian Code for the Responsible Conduct of Research, 2018, and the National Statement on Ethical Conduct in Human Research, 2007 may result in withdrawal of approval for the project. Congratulations on your ethical approval! Wishing you all the best for success!

If you have any questions or concerns, please don't hesitate to make contact with an Ethics Officer.

Kind regards

Human Research Ethics

University of Southern Queensland
Toowoomba – Queensland – 4350 – Australia
Phone: (07) 4631 2690
Email: human.ethics@usq.edu.au

APPENDIX D

FLOODWAY DESIGN GUIDELINE

FLOODWAY DESIGN GUIDELINE

I. Greene, W. Lokuge* and W. Karunasena

Centre for Future Materials, School of Civil Engineering and Surveying, University of Southern Queensland, QLD 4300, Australia

Emails: Isaac.Greene@usq.edu.au, *Weena.Lokuge@usq.edu.au, Karu.Karunasena@usq.edu.au
*Corresponding author

FOREWARD

Following a series of extreme natural disasters within Australia, enhanced community resilience has become the forefront of many infrastructure related projects. In July 2013 funding from the Australian Government was announced to support national research through the Bushfire and Natural Hazard Cooperative Research Centre (BNHCRC). This effort has been supported by local, state and federal government bodies and universities.

Researchers from the University of Southern Queensland in partnership with RMIT University have investigated the behavior of floodways under extreme flood loading. Floodways are defined as road structures that improve the trafficable surface of waterways to facilitate safe vehicular movement. Floodways are often implemented in lower order roads where periods of unserviceability can be accepted. Through the undertaking of research contributing to the development of this design guideline it was discovered that the current floodway design process focusses primarily on hydraulic design criteria omitting structural analysis considerations. This floodway design guideline has an increased focus on ensuring floodways are structurally adequate to withstand flood-related loadings whilst in a submerged state. The floodway design guideline steps the user through all stages of floodway design, including preparatory works, floodway selection and hydraulic, protection and structural design. It is intended that a designer can use this manual inclusive of design charts and comprehensive worked examples as a guide to rural floodway design.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the Lockyer Valley Regional Council for providing information relating to floodways and the support of the Commonwealth of Australia through the Cooperative Research Centre program; *Bushfire and Natural Hazard CRC* and the *Research Training Program (RTP) Scholarship*.

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1. INTRODUCTION

Following a series of extreme natural disasters within Australia, enhanced community resilience has become the forefront of many infrastructure related projects. In July 2013 funding from the Australian Government was announced to support national research through the Bushfire and Natural Hazard Cooperative Research Centre (BNHCRC). The development of this design guideline was commissioned to increase the resilience of rural communities by providing a method to design concrete floodway structures with greater structural integrity, consequently reducing the likelihood of asset damage post extreme flood events. It is anticipated by adopting these practices, community disruption and local government expenditure will be reduced and access restrictions to rural communities will be less frequently imposed.

Floodways are traditionally designed to convey water in an efficient and safe manner while facilitating safe vehicular movements. The hydraulic design of floodways is well documented in design guidelines at both a national and international level [Queensland Department of Transport and Main Roads, 2010; Main Roads Western Australia, 2006; Austroads, 2013; US Army Corps of Engineers Afghanistan Engineer District, 2009; Lohnes et al., 2001]. The hydraulic design utilised within these design guidelines focuses on implementing traditional hydraulic design practises such as the Manning's formula and the Empirical Broad Crested Weir formula and assumes that the primary mechanism of failure occurs when design discharge exceeds the hydraulic capacity of the floodway or adjacent rock protection.

Significant research and investigation have been undertaken into the principal failure mechanisms of floodways during peak flow events in recent years [BNHCRC, 2015; Wahalathantri et al., 2015; Furniss et al., 2002; GHD 2012]. The research conducted by BNHCRC (2015), Wahalathantri et al. (2015) and GHD (2012) specifically investigated the factors contributing to the many floodway failures reported after the significant and widespread flooding within Australia. These studies have recorded consistent failure mechanisms and have attributed the primary cause of floodway failure to increased sediment loads, organic debris (logs) and boulders impacting the floodway causing the superstructure to experience significant loading while in a submerged state. Furniss et al. (2002) explains that this type of failure is often complex and is a factor of the location and frequency of landslides, watercourse bank erosion, treefall and other processes occurring upstream of the concrete floodway structure.

This design guideline considers a structural model to predict failure during a worst-case peak flow and an associated boulder impact load. This is achieved by undertaking numerical finite element modelling and simulation to develop a design methodology based on structural design considerations for concrete floodways. This design methodology still relies upon and incorporates the hydraulic practices from traditional design guidelines for floodway flow characteristics, however, it does not consider hydraulic principles as the primary predictor of floodway failure. The incorporated structural design methodology within this design guideline provides designers with an expeditious method to calculate the ultimate design forces using design charts. By implementing this design process, structural resilience is improved through satisfying durability, serviceability and strength criteria. It is intended that a designer can use this manual inclusive of structural design charts and worked examples as a complete guide to floodway design.

Further details on the numerical finite element modelling and simulation methods, which were used to derive the structural design charts and methodologies within this guideline are available in the journal article, *"Structural design of floodways under extreme flood loading"* (Greene et al. 2020a, pp. 535-555).

Industry input and experience was also received through the undertaking of an Australian focused survey, along with the review of previous international survey work and national floodway grant funded projects. This provided site-specific intuition from engineers enabling an understanding of floodway failure, including its prevalence and the common failure mechanisms observed in practise. Sixty-four responses were received with participation from Queensland, New South Wales, Victoria and South Australia, providing a significant cross-section of floodway experience throughout Australia. Furthermore, the failures noted by the survey respondents provided qualitative agreement with the finite element model results used to derive the structural design procedure detailed within this design guideline.

The results of this survey were published by the Institute of Public Works Engineering Australasia, Queensland branch (IPWEAQ) in the Engineering for Public Works e-journal, *"Floodways and Flood-Related Experiences: Survey of Industry and Asset Owners"* (Greene, Lokuge & Karunasena 2020b, pp. 69-75). These results provide interesting insights into floodway related issues and highlights additional items for consideration in floodway design and implementation.

1.1 SCOPE, ASSUMPTIONS AND LIMITATIONS

This floodway design guideline provides preliminary guidance on hydrological, hydraulic, structural, and civil design considerations of simple floodway structures to achieve maximum structural resilience during extreme flood events. This document is not prescriptive, rather it is designed to assist an experienced designer who shall make their own independent assessment on suitability and if other design considerations are required.

Structural design criteria within this guideline are based upon an accidental loading combination resulting from the impact of a boulder during an extreme flood event. As a result, design considerations are based on achieving resilience against flooding and a high serviceability level, which may not necessarily derive the most economical solution.

2. FLOODWAYS

Floodways are often used within ephemeral (short lasting) water courses, water courses with shallow continual flow and in large flow scenarios with the addition of culverts (Gautam and Bhattarai 2018). The application of floodways is suited to relatively consistent creek bed profiles and not deeply incised creek beds (Clarkin et al. 2006). The floodway structure is designed to permit vehicular crossing during low flow events through improvements to the stability and predictability of the trafficable surface (Figure 2.1).

Figure 2.1. Typical floodway implemented on a rural road.

Floodways are a solution often implemented when a reduced or interrupted level of service during rainfall events can be accepted. This often includes lower order roads such as minor roads, formed tracks and access roads with a relatively low vehicle count per day (less than 50). Floodways, due to their simple anatomy provide capital savings compared to other road asset classes such as bridges and culverts. However, due to frequently being in a submerged state floodways often require greater maintenance.

Floodways can also be of an unsealed, sealed or concrete construction depending on the level of service required. Many rural roads in outback and regional Australia are typically of an unsealed (gravel) or sealed construction, as opposed to concrete floodways which are frequently implemented in semi-rural areas. Due to the regular upkeep required of unsealed floodways, preference has been to replace these structures with concrete floodways to reduce the need for significant reoccurring operational expenditure for councils. Floodways can also be situated level with the channel bed or raised. They can also be unvented or vented through the incorporation of culverts. Flow over the floodway structure is generally dispersed more widely than that of culvert applications, therefore, aiding in the reduction of flow concentrations and erosion downstream of the structure (Queensland Department of Transport and Main Roads, 2010).

GHD (2012) and Wahalathantri et al. (2015) have categorised the principal failure mechanisms of floodways into three main categories:

1. Erosion – Erosion occurs when the velocity of flow is high enough to overcome the scour resistance of rock protection and the natural channel strata. Scouring, if not rectified, undermines the floodway structure causing failure.
2. Deposition - Deposition is a result of localised watercourse widening or from reducing the original watercourse grade to make allowance for the floodway structure. This subsequently slows the velocity of flow and allows sediments and silts to settle on the structure. This mechanism rarely results in complete failure.
3. Failure of the structure – Floodways are exposed to significant loadings throughout their serviceable life as a result of being in a frequently submerged state, including; drag, debris, impact, hydrostatic pressure, uplift and vehicular loading (although not necessarily concurrently). If these loadings exceed the design strength then failure will occur, sometimes catastrophically.

A typical concrete floodway has seven major components (Figure 2.2) as follows:

A. Upstream Rock Protection - provides protection against scour in the upstream zone.
B. Upstream cut-off wall - prevents undermining of the structure and ground water movement through the underlying granular road pavement.
C. Apron - this is the trafficable road surface.
D. Downstream batter - acts as a spillway for the structure and accounts for any difference in upstream and downstream elevation. The batter is often armoured to prevent scouring.
E. Downstream cut-off wall - prevents undermining of the structure and ground water from flowing through the underlying granular road pavement.
F. Downstream Rock Protection - provides protection against scour in the downstream zone.

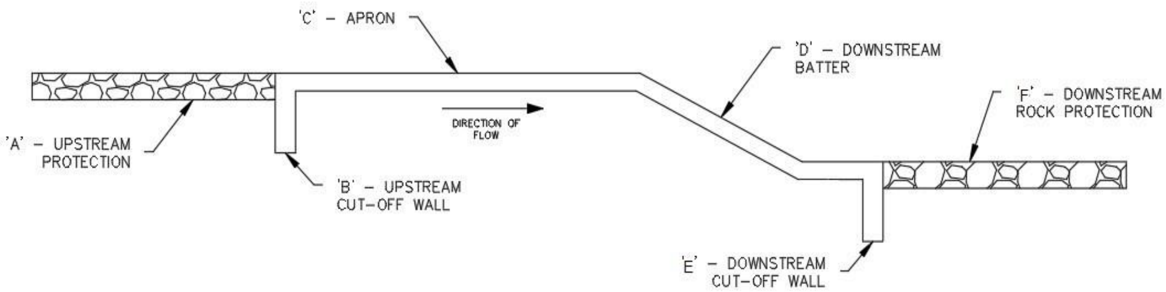


Figure 2.2. Major components of a typical floodway.

3. CONCRETE FLOODWAY DESIGN

3.1 GENERAL

Floodways incorporate several design aspects including road design, pavement design, hydraulic design, and structural design. It is important that each aspect is carefully considered and designed to reduce the likelihood of asset damage and community disruption post extreme flood event. An overview of the design procedure adopted within this design guideline is provided below (Figure 3.1).

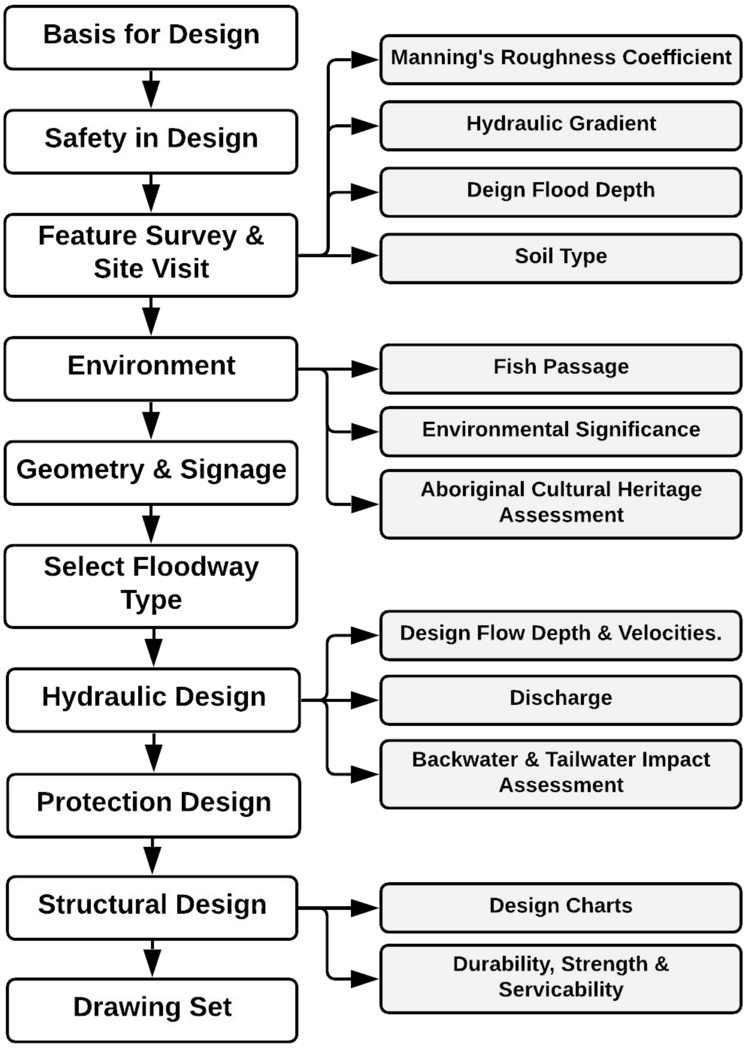


Figure 3.1. Floodway design process flowchart.

3.1.1 Design References

Several design references are referred to within this design guideline which complement and provide further details on specific design components relating to floodways. A summary of the primary design references is detailed below:

a. Australian Standards:

i. Australian Standard 5100:2017 series - *Bridge Design – Concrete* [Standards Australia 2017A; Standards Australia 2017B].

ii. Australian Standard 1742.2:2009 - *Manual of Uniform Traffic Control Devices, Part 2 – Traffic Control Devices for General Use* (Standards Australia 2009).

b. Floodway design guidelines provided by state and national bodies:

i. *Guide to Road Design Part 5B: Drainage – Open Channels, Culverts and Floodway,* Austroads (2013).

ii. *Road Drainage Manual Chapter 10 – Floodway Design*, Department of Transport and Main Roads (2010).

iii. *Floodway Design Guide,* Main Roads Western Australia (2006).

c. Guidelines referred to which provide input into the construction of floodways:

i. *Survey and Mapping Standard 67-08-42 - Waterways Investigation Surveys*, Main Roads Western Australia (2014)

ii. *Queensland Urban Drainage Manual* (QUDM), IPWEAQ (2016B).

iii. *Reinforcement Detailing Handbook*, Concrete Institute of Australia (2010).

iv. *Culvert Fishway Planning and Design Guidelines*, James Cooke University (Kapitzke, 2010).

v. *Erosion and Sediment Control – Field Guides,* Catchment and Creeks (2021).

vi. *Guidelines for Watercourse Crossings on Waterfront Land*, NSW Office of Water (2012).

3.2 BASIS FOR DESIGN

The decision to adopt a floodway structure is based on the intended level of service for the road being designed and, on the organisations, individual serviceability standards. Floodway design is often a trade-off between initial construction costs and ongoing maintenance requirements. A floodway that is correctly designed to a specific crossing environment will provide increased resilience, reduce maintenance requirements and extend asset life. The 'basis of design' summarises the owners project requirements and is the foundation for good decision making throughout the design process. The 'Basis for Design' shall form a summary of the primary assumptions, rationale, design criteria and considerations required for the project. For floodways this shall include acceptable closure periods, maintenance periods, design rainfall events and the required level of service.

In floodway design the road asset classification is the primary input for the 'basis for design' as it defines the level of service that the floodway design must achieve. To assist in determining floodway suitability the IPWEAQ (2015A) provides a table which details road asset classifications and floodway suitability based on required level of service (Table 3.1).

Table 3.1. Determining asset classification and floodway suitability (IPWEA 2015).

Asset Classification Characteristic	Major Road	Minor Road	Local Access	Formed Track
Vehicles per day	> 150	50 - 150	10 - 50	0 - 10

Design speed (km/hr)	≥ 100	100 - 60	70 - 40	50 - 30
Road type	Sealed	Sealed or unsealed	Sealed or unsealed	Unsealed
Number of properties servicing	N/A	> 25	4 - 25	< 4
Floodway suitability	Not suitable	Suitable – two lane width	Suitable – two or single lane width	Suitable – single lane width

For the application of a single lane floodway the following minimum width shall be implemented:

1. 4.5 m (3.5 m lane and 2 x 0.5 m shoulders).

For the application of a double lane floodway the following minimum width shall be implemented:

2. 8.0 m (3.5 m lane and 2 x 0.5 m shoulders).

If the specific asset owner requires a higher level of service, floodway widths and shoulders can be extended for a lower road asset classification.

3.3 SAFETY IN DESIGN

Safety risks and hazards need to be identified and mitigated in the design process of floodways. These aspects include planning, design, construction, operation, and maintenance. To identify and mitigate potential risks and hazards the safe design of floodways shall comply with the following legislative requirements and guidelines:

- Work Health and Safety Act (2011).
- Work Health and Safety Regulations.
- Safe Design of Structures, Code of Practice (Safe Work Australia, 2012).

3.4 FEATURE SURVEY AND SITE VISIT

Survey data in the form of a feature survey is used to develop a detailed understanding of the topography and geometrical characteristics of the unrestricted waterway. This data is utilised in hydraulic analysis to calculate the channel cross-sectional area, hydraulic gradient, geometric alignment, peak flow analysis and for the preparation of design drawings. Main Roads Western Australia (2006) provides a detailed technical guide to surveying waterways, namely, *'Survey and Mapping Standard 67-08-42 - Waterways Investigation Surveys'.* The minimum data required to understand the topography of a watercourse which is detailed within this guideline is summarised as follows:

- Cross-sections of the watercourse both upstream and downstream of the proposed structure that extend beyond the water level for the design flow being considered.
- A long section of the streambed, including the water surface profile.
- A long section of the existing or proposed road centerline.
- Contour plan.
- Site photographs.
- Flood level indications.

The feature survey shall also pick-up the existing surface contours, other structures and significant landmarks relevant to the design, such as, buildings, fences, trees, adjoining roadways, drainage, the existing channel, and any utility services which may be impacted during construction.

During the feature survey relevant field data shall also be collected for input into the hydraulic design process as follows:

1. Manning's coefficient of roughness (n): The Manning's Coefficient of Roughness for the channel needs to be determined for the channel. Table A.1 in Appendix A outlines general Manning's roughness coefficients for different typical channel types. It must be acknowledged that although this table provides estimates for Manning's roughness coefficient, there are many variables in a natural channel that can significantly influence this value. These variables include flow depth, discharge, localised obstructions, channel irregularities, alignment, and silting/scouring. It is therefore prudent that the proper perspective is established when assigning roughness values for a channel.
2. Hydraulic gradient (S): Hydraulic gradient can be approximated by the bed slope of the channel, which is determined by dividing the rise of the channel bed by the run. The most accurate method to achieve this is by undertaking a detailed survey of the channel bed, both upstream and downstream of the proposed floodway crossing site. Consideration must be taken when selecting the run length to ensure undesirable averaging does not occur due to localised undulations within the channel. Main Roads Western Australia (2014) specifies that the run length should extend to a minimum 200 metres upstream and 200 metres downstream of the proposed structure.

   hydraulic gradient can also be calculated from the water surface profile by determining the gradient during a rainfall event or between reliable debris marks left on the immediate riverbank after a rainfall event. This reasonable and convenient prediction of slope provides an accurate estimation, since slope does not vary significantly over a range of flows, or rate of change of flow at a point (Fenton & Keller, 2001).

3. Design flood depth (h): the depth of flow at the proposed floodway site can be calculated through a range of different methods as listed below with accuracy decreasing in descending order:
   - Theoretical estimates through validated and verified[1] hydrological modelling of design flood events.
   - Gauging stations (Figure 3.2) capture reliable stream data enabling the depth and flow rate of historical rainfall and flood events to be determined.
   - By reference to historical photography of recorded flood events, particularly aerial photography.
   - Historical flood references or observing reliable debris marks left on the riverbank immediately after a flood event of the design size.

[1] Stream data from gauging stations, historical flood data, photography and recorded observations of known flood events can be used to validate and verify hydrological models. This enables a greater level of confidence to be achieved.

Figure 3.2. A typical gauging station (Reedy Creek).

4. Soil type categorisation – The mechanical properties of soil within the channel have a significant impact on the resilience of concrete floodway structures against extreme flood loadings and is a requirement for the design of the structure in Section 3.10. Soil type is based upon the outcomes of an appropriate level of geotechnical investigation as recommended by a NATA accredited geotechnical consultancy. The mechanical properties of the in-situ soil type shall be categorised into one of three commonly encountered soil types (APPENDIX A, Table A.2), if alignment of local conditions to the tabulated soil types is not achieved, then a specific design solution is required. Soil aggressivity shall also be categorised for use in concrete durability design.

3.5 ENVIRONMENT

The implementation of a floodway crossing within a waterway has several environmental sensitivities that need to be well managed during design to avoid environmental degradation. The NSW Office of Water (2012) has produced a guideline named, *'Guidelines for Watercourse Crossings on Waterfront Land'*, this document provides a detailed list of considerations which shall be considered during the design and construction of a water crossing structure. By implementing these considerations hydrologic, hydraulic, and geomorphic degradation can be mitigated and environmental qualities preserved. In addition, a Review of Environmental Factors (REF) shall be undertaken, which includes a flora and fauna biodiversity assessment and aboriginal cultural heritage assessment. This REF then needs to be assessed and approved by the relevant local or state government entity who will provide conditions relating to the construction. Fish passage, biodiversity and cultural heritage due diligence investigations are discussed further below.

- Fish passage - structures such as floodways have a significant negative impact on fish passage. Kapitzke (2010) explains that structures, such as floodways can cause significant hydraulic barriers precluding fish passage due to the potential of high velocities, turbulence, insufficient water depth and significant batter gradients. Fish passage provisions at floodway crossings are of environmental significance and an issue that should be addressed during design. The James Cook University, School of Engineering in Physical Sciences has undertaken detailed research and development studies into fish passage provisions for road structures and has created a best practice design guideline series called *'Culvert Fishway Planning and Design Guidelines'* (Kapitzke, 2010). This design guideline series consists of five parts as follows:

- Part A - About these guidelines.
- Part B - Fish Migration and Fish Species Movement Behaviour.
- Part C - Fish passage barriers and options.
- Part D - Fish Passage Design: Road Corridor Scale.
- Part E - Fish Passage Design: Site Scale.

Floodway designers should consult this design guideline series to determine the full scope of requirements and to ensure appropriate fish passage provisions are implemented.

- Environmental assessment - a review of environmental factors shall be undertaken to investigate the current site conditions, determine environmental impacts and to assess the risk level of the impact. In a floodway context these impacts may be due to channel modifications, erosion potential, vegetation removal and waterway pollution.
- Aboriginal cultural heritage assessment – the Department of Environment Climate Change & Water (2010) explains that landscape features such as waterways have a high probability of Aboriginal cultural significance. A due diligence assessment that satisfies the relevant state authority shall be undertaken prior to carrying out any construction activities to avoid harming any objects of significance. If an area of significance is discovered, then the required permits to impact the site and potentially harm or destroy an object shall be obtained.

A Construction Environmental Management Plan (CEMP) addressing the conditions of the determined REF shall be prepared and submitted for approval prior to construction commencing. Catchment and Creeks (2021) provides erosion and sediment control field guides specific to instream works to assist in the formation of the CEMP.

3.6 GEOMETRY AND SIGNAGE

3.6.1 Geometric Alignment

Floodways often exist in complex surroundings that include horizontal bends, undulating channel depths and water courses that do not consist of uniform nor prismatic channels. For new floodway constructions or where flexibility of geometric alignment exists the designer shall attempt to align the floodway orientation and locality in accordance with the below considerations. These considerations act to reduce the likelihood of undue deterioration of the floodway superstructure and to increase vehicle crossing safety when navigating the floodway structure.

Horizontal Alignment:

Floodways were possible shall not be located on horizontal curves. If a horizontal curve is required by design, then a minimum horizontal radius of 1,000 m shall be adopted. Straight alignments perpendicular to the watercourse are preferential as they allow motorists to define the pavement edge when the floodway is in a submerged state. Straight alignments also reduce the potential for downstream batter erosion and reduce protection requirements as flows are typically more uniform.

Vertical alignment:

To ensure predictability of the floodway crossing, approaches should be limited to less than 15% grade and the mid-section shall have minimum vertical elevation variance to ensure uniform water depth exists throughout the floodway extent. The crossfall at the floodway structure shall also be limited to a maximum of 2% in the direction of flow (drainage requirement). The vertical alignment of the road leading up to the floodway shall provide motorists with appropriate sight distance so that they can react safely to adverse conditions (Main Roads Western Australia, 2006).

It is also important to sufficiently extend the floodway and associated protection up the approach to prevent scouring due to afflux at the interface between the concrete floodway structure and the adjoining road, particularly when an adjoining gravel road construction is being used.

3.6.2 Signage

Floodway signage needs to comply with Australian Standard 1742.2:2009, *"Manual of Uniform Traffic Control Devices, Part 2 – Traffic Control Devices for General Use"* (Standards Australia, 2009).

An isolated diamond warning sign with the words *"FLOODWAY"*, W5-7 (Figure 3.3) provides drivers with advance warning and enables them to stop prior to entering the floodway crossing. This sign is best positioned at the highest available point and at distance 'A' from the sign *"ROAD SUBJECT TO FLOODING INDICATORS SHOW DEPTH"* (G9-21), where distance 'A' is determined in reference to Table 3.4 and Figure 3.6.

Figure 3.3. A typical W5-7 *"Floodway"* advance warning sign.

A sign with the words *"ROAD SUBJECT TO FLOODING INDICATORS SHOW DEPTH"*, G9-21 (Figure 3.4) shall be provided at distance 'B' from the floodway depth indicators, G9-22, where distance 'B' is determined from Table 3.4 and Figure 3.6. This sign is used to alert motorist of a potential water over road condition and to use the available depth indicators for judgement prior to crossing.

Figure 3.4. A typical G9-21 *"Road Subject to Flooding, Indicators Show Depth*" sign.

Depth indicators, G9-22 (Figure 3.5) are provided at the floodway crossing and shall be clearly visible by motorist. The zero mark needs to indicate the absolute lowest point of the floodway crossing.

Figure 3.5. A typical G9-22 *"Depth Indicator"* sign.

Table 3.2. Sign distances (Standards Australia, 2009)

Dimension	Description	V_{85}, km/h[2]			Units
		<75	75 – 90	>90	
Dimension A	Distance of W5-7 sign from G9-21 sign	80 – 120	120 - 180	180 – 250	meters (m)
Dimension B	Distance of G9-21 sign from G9-22 depth indicators	50	60	70	meters (m)

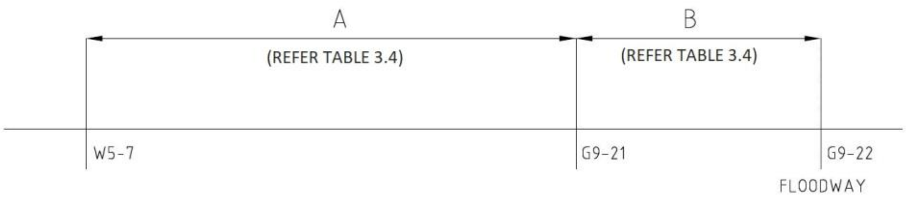


Figure 3.6. Floodway sign required spacing (Standards Australia, 2009).

During flood events temporary *"WATER OVER ROAD"* or *"ROAD CLOSED"* signs shall also be placed based on the present crossing condition (Figures 3.7 and 3.8). With the use of these signs, road authorities must develop robust policies, procedures, and systems that detail the level of assessment required to enable floodway condition to be assessed and to prioritise maintenance post flood events, thus ensuring these signs can be safely removed and the road reopened once the event ceases to exist.

[2] V_{85} is the 85th percentile speed, which is the speed at or below which 85% percent of vehicles are observed to travel at under uncongested conditions. AS 1742.2 (Standards Australia, 2009) provides a guide for the determination of the 85th percentile speed.

Figure 3.7. A typical "Water Over Road" sign.

Figure 3.8. A typical "Road Closed" sign.

Road edge guide posts (Figure 3.9) shall also be placed at a minimum of 25 metre spacings to indicate the road edge and to assist motorists in safely navigating the floodway.

Figure 3.9. A typical road edge guide post.

3.6.3 Road Level

Road level is dependent on the type of floodway being implemented. Floodways can be aligned level or raised relative to the channel bed. level floodways do not affect the hydraulic control of the waterway, that is, the road level is equal to the channel invert level at the crossing location. For raised floodways which affect the hydraulic control of the waterway, the road level is to be carefully considered by the designer to ensure that the time of closure is minimised and that the backwater levels do not cause flooding and subsequent damage to upstream assets. This is normally constrained by geometrical requirements and to ensure high velocities do not exist at critical erosion and scouring zones such as the downstream batter. Examples of raised and level floodways are provided in Section

3.7, *"Select Floodway Type"*. Selected road level is further refined in the hydraulic design process through computational modelling.

3.7 SELECT FLOODWAY TYPE

Five concrete floodway types are provided for use as standard engineering designs based on different water course geometries and flow characteristics (LVRC 2008a, 2008b, 2008c, 2008d & 2008e). These standard floodway types provide designs which affect the hydraulic control of the waterway (raised floodway types) and designs which do not affect the hydraulic control of the waterway (constructed level with the channel).

3.7.1 Design Considerations

Specific construction considerations common to the five specified standard floodway structures are as follows:

Cut-off walls:

The cut-off walls shall extend around the entire perimeter of the concrete floodway inclusive of the approaches at each end. The cut-off wall provides several important functions as follows:

- Increase the stabilising moment of the floodway structure by providing a greater distribution area to the adjoining soil. This significantly increases the structures ability to resist horizontal loading in the direction of flow from loadings such as debris, drag and impact.
- Prevent groundwater from flowing through the underlying foundation material.
- Protects the structures foundation material against scouring and undermining, both upstream, downstream and at the transitions from the concrete floodway back to the road surface at the approaches.

Structural considerations within this design guideline are based on a minimum 900mm cut-off wall depth to achieve the above desired functions.

Foundation:

Foundation adequacy shall be investigated thoroughly through appropriate geotechnical investigation and subsequent pavement profile design. In addition, the long-term effects of pavement durability shall be considered, including the constant variability in moisture content related to fluctuation in the watercourse water level. Stabilisation of the foundation material is recommended to enable retention of pavement strength while in a saturated state. Alternative foundation materials such as lean mix concrete and foam bitumen pavement have also been successfully adopted as suitable foundation materials in floodway crossing applications.

Concrete Structure:

A minimum compressive strength of 40 MPa shall be utilised to satisfy the durability requirements of AS 5100.5 (Standards Australia, 2017A). Furthermore, all slabs shall be cast in-situ and be monolithic in construction (cast in one continuous pour).

3.7.2 Raised Versus Level Floodway Construction

There is an emerging trend of floodway asset owners vertically realigning damaged floodway structures level with the channel bed after observing significantly less exposure to damage during extreme flood events for this construction type.

Floodway structures situated level with the channel bed (Figure 3.10) are extremely effective structures due to not affecting the hydraulic control of the waterway. This type of construction therefore reduces lateral loadings which increase proportionally with velocity, such as drag, debris and impact. The potential of scour and undermining of the downstream cut-off wall is also significantly reduced as flow is not accelerated to supercritical levels as is the case in raised floodway structures at the downstream batter. Level floodways are also a significantly simpler structure and are comparatively cost effective to implement when compared to other crossing structure types. However, depth of flow is uncontrolled, causing traffic movements to be precluded during anything other than minor rainfall events reducing the level of service that can be achieved (increased time of closure).

Figure 3.10. A typical level floodway construction.

Raised floodway structures (Figure 3.11) are critical in applications where control of the flow depth and velocity over the structure needs to be achieved. These structures reduce time of closure and facilitate the incorporation of culvert structures allowing the road surface to remain unaffected during minor flood events.

Figure 3.11. A typical raised floodway construction.

3.7.3 Standard Floodway Types

A set of standard engineering floodway drawings, referred here in as Type 1 to Type 5 have been provided. These standard drawings provide a typical detail for generalised situations only. Where a specific detail is to be implemented in a project, the suitability of its application needs to be confirmed by the user. Note: all dimensions are in millimeters (mm) unless stated otherwise.

Floodway Type 1

The Type 1 floodway (Figure 3.12) is designed to be implemented in a waterway:

- Of relatively flat grade.
- Where no hydraulic control is required to facilitate safe vehicular crossing conditions. That is creek bed level is the same as road surface.
- Where a reduced level of service can be accepted (greater time of closure).
- Where the channel bed is of similar grade upstream and downstream of the floodway structure.

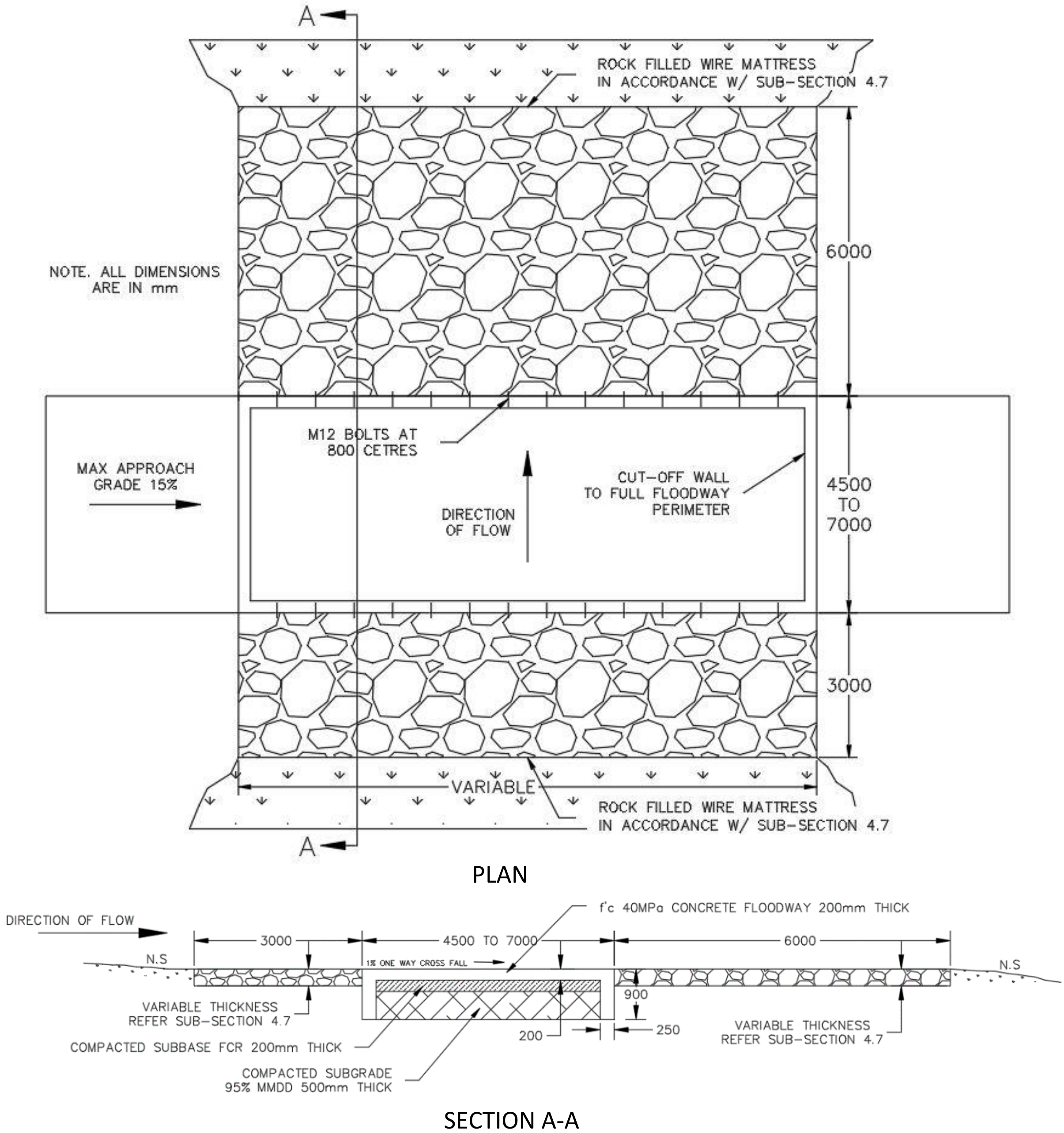
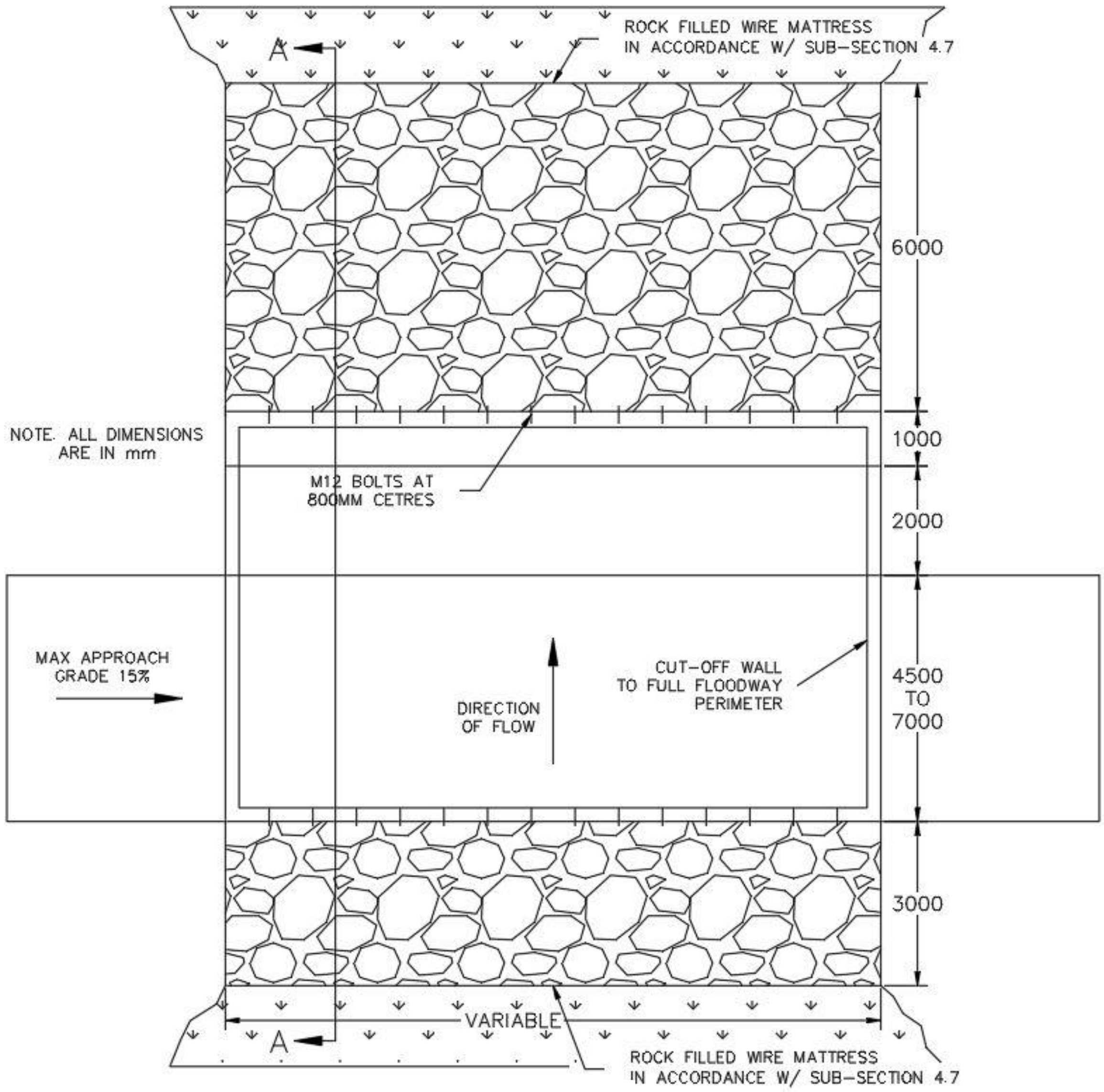


Figure 3.12. Floodway Type 1 details (LVRC 2008a).

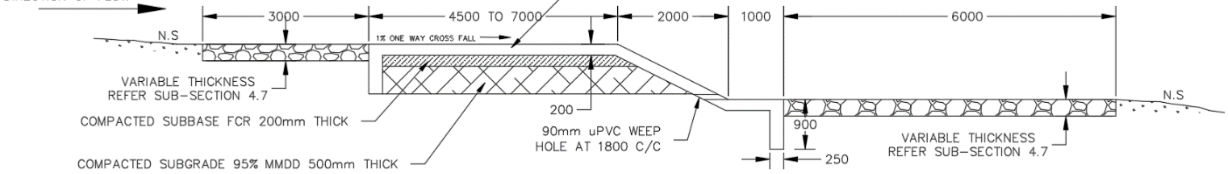
Floodway Type 2

The Type 2 floodway (Figure 3.13) is designed to be implemented in waterways:

- Where a concrete batter of variable grade is required downstream.
- Where no hydraulic control is required to facilitate safe vehicular crossing conditions.
- Where a reduced level of service can be accepted (greater time of closure).



PLAN VIEW



SECTION A-A

Figure 3.13. Floodway Type 2 details (LVRC 2008b).

Floodway Type 3

The Type 3 floodway (Figure 3.14) is designed to be implemented in waterways:

- With significant catchment areas which contribute high discharges.
- Where a hydraulic control is required to facilitate safe vehicular crossing.
- To achieve a higher level of service since the height of the floodway can be adjusted to reduce time of closure, increasing passability.

Note. A drainage culvert should be incorporated to reduce the effect of backwater and associated flooding upstream, prevent stagnation of upstream water and to prevent water flowing over the floodway in low flow conditions.

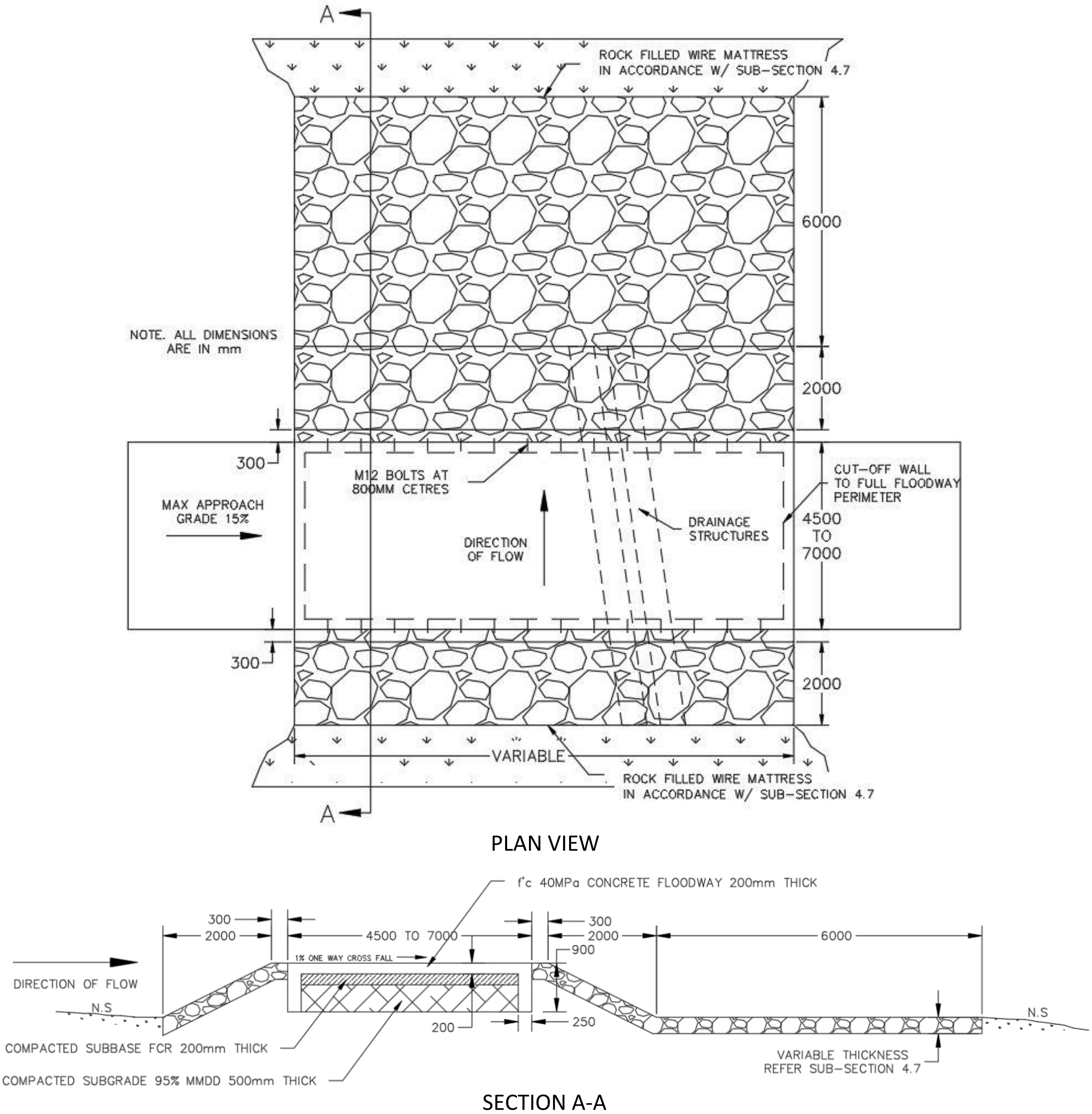


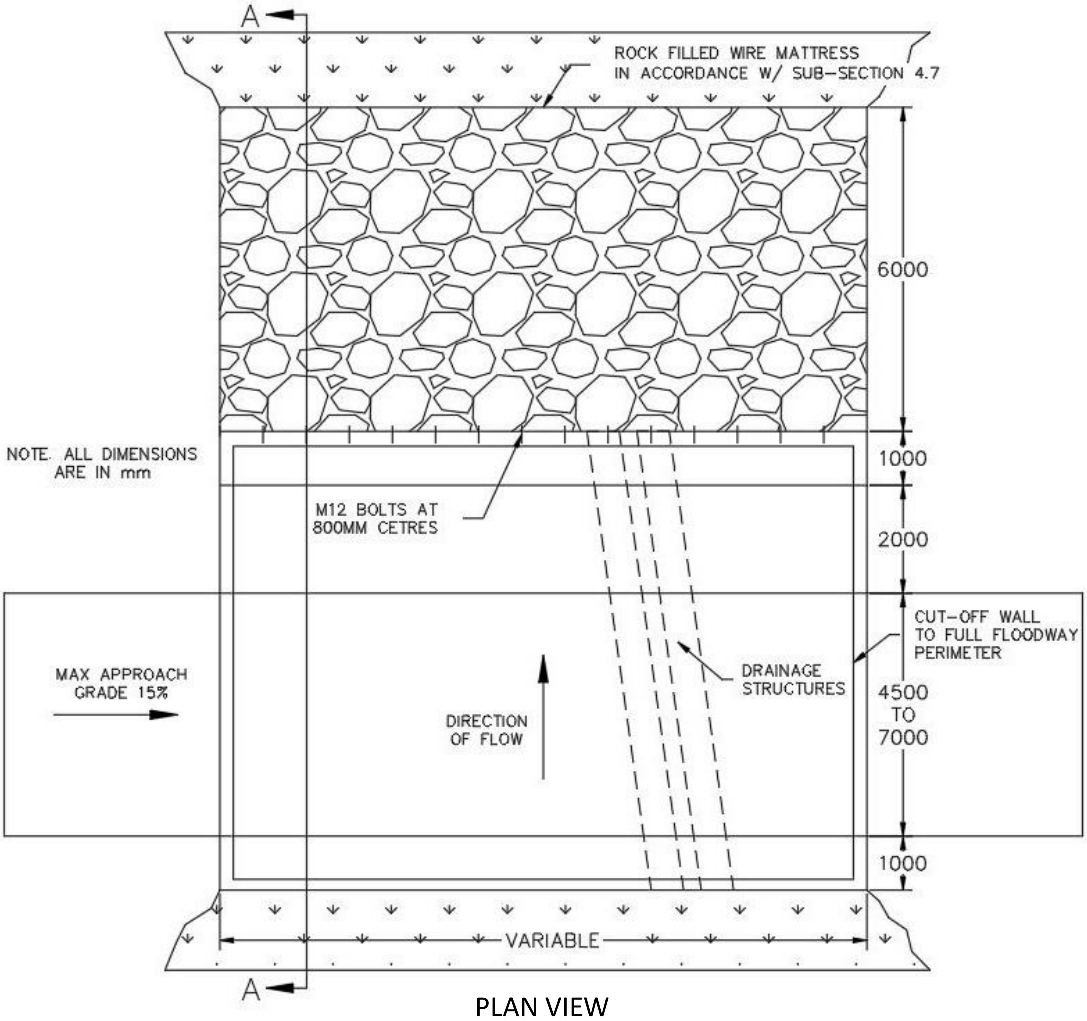
Figure 3.14. Floodway Type 3 details (LVRC 2008c).

Floodway Type 4

The Type 4 floodway (Figure 3.15) is designed to be implemented in waterways:

- With significant catchment areas which contribute high discharges.
- Where a hydraulic control is required to facilitate safe vehicular crossing conditions.
- To achieve a higher level of service since the height of the floodway can be adjusted to reduce time of closure, increasing passability.

Note. A drainage culvert should be incorporated to reduce the effect of backwater and associated flooding upstream, prevent stagnation of upstream water and to prevent water flowing over the floodway in low flow conditions.



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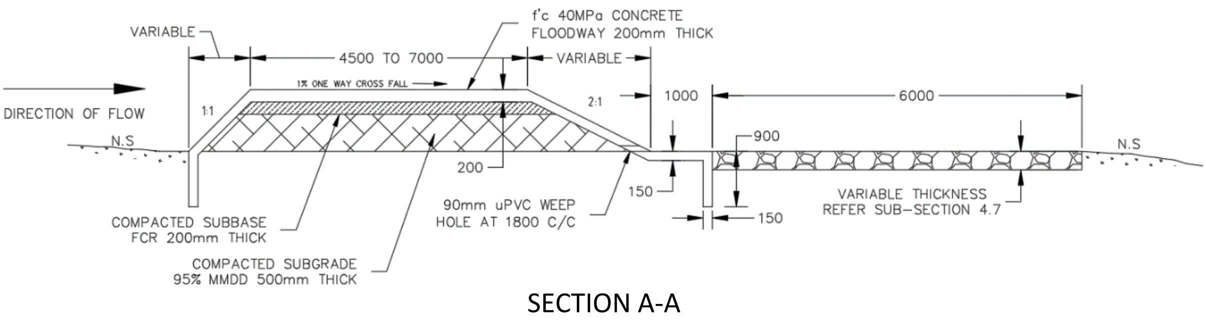


SECTION A-A

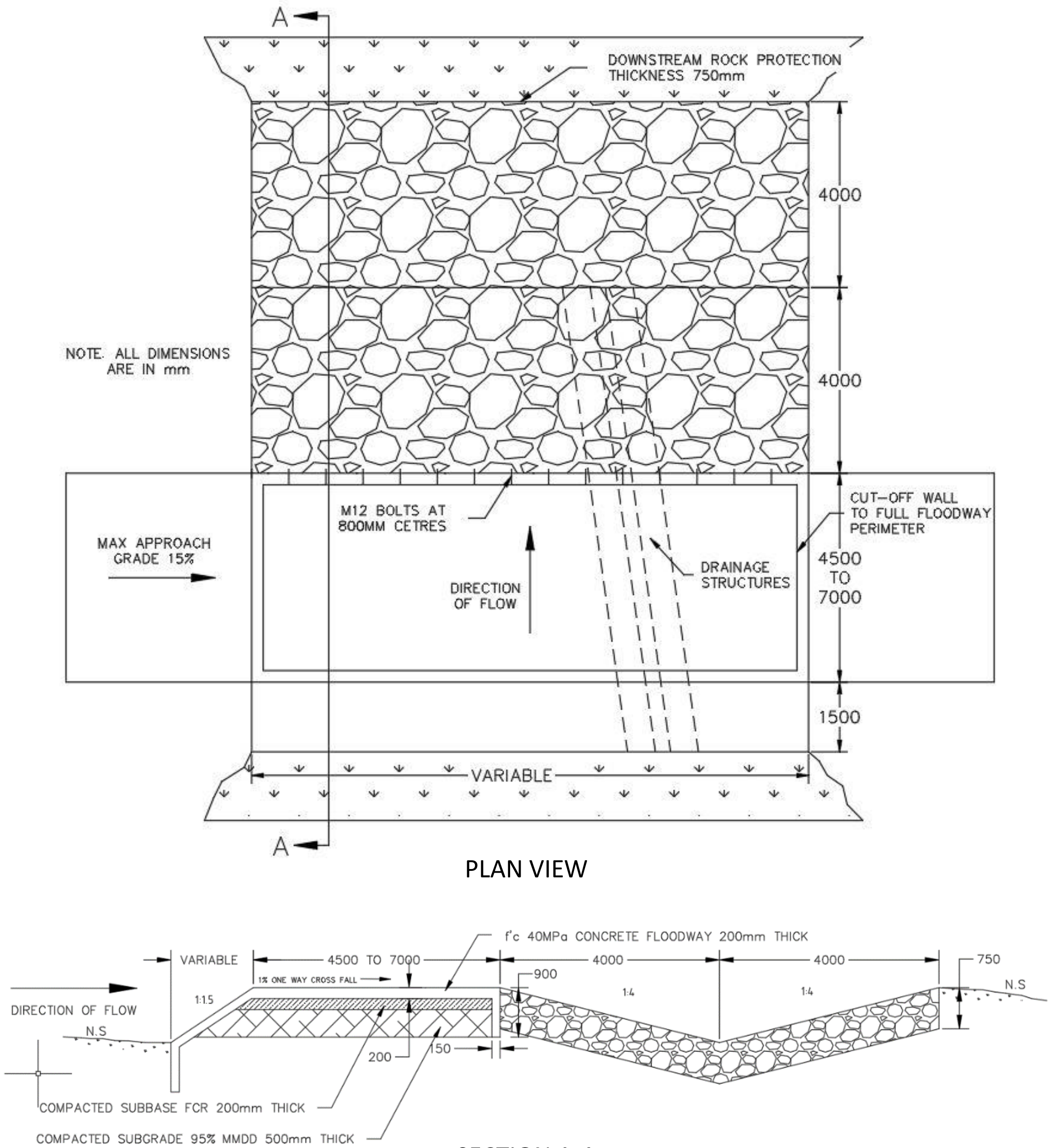
Figure 3.15. Floodway Type 4 details (LVRC 2008d).

<u>Floodway Type 5</u>

The Type 5 floodway (Figure 3.16) is designed to be implemented in waterways:

- With significant catchment areas which contribute high discharges.
- Where a hydraulic control is required to facilitate safe vehicular crossing conditions.
- Incorporates a v-shaped downstream rock protection arrangement. This deepening of the waterway channel dissipates flow energy and provides protection against increased bed shear stress.

Note. A drainage culvert should be incorporated to reduce the effect of backwater and associated flooding upstream, prevent stagnation of upstream water and to prevent water flowing over the floodway in low flow conditions.



SECTION A-A

Figure 3.16. Floodway Type 5 details (LVRC 2008e).

Table 3.5 has been provided to assist in the selection of a standard engineering floodway type based on the design watercourse flow characteristics (depth and velocity).

Table 3.3. Selection of standard engineering floodway type.

Floodway Cross-section	Flow Depth			Flow Velocity		
	High	Medium	Low	High	Medium	Low
Floodway Type 1						
Floodway Type 2						
Floodway Type 3						
Floodway Type 4						
Floodway Type 5						

Key: Usually suitable Not ideal/doubtful Usually not-suitable

3.8 HYDRAULIC DESIGN

The hydraulic design of floodways is covered extensively in the existing design guidelines which are listed for reference as follows:

i. Austroads (2013) *"Guide to Road Design Part 5B: Drainage – Open Channels, Culverts and Floodway".*
ii. Department of Transport and Main Roads (2010) "*Road Drainage Manual Chapter 10 – Floodway Design"*.
iii. Main Roads Western Australia (2006) *"Floodway Design Guide"*.

The hydraulic design criteria presented in this section summarises the hydraulic design procedures and methodologies derived in these design guidelines, which are required to be implemented in the design of the five (5) standard engineering floodway types. It also enables design flow, depth and velocity to be determined for the structural design method outlined in Section 3.11.

3.8.1 Design Discharge

The design discharge is the flow rate which the floodway structure is exposed to for a defined average recurrence interval (ARI). The selected ARI and corresponding flow rate have a direct impact on floodway type, time of road closure and whether or not a drainage structure is required. The design discharge is determined via hydrology assessment of the contributing catchment. Queensland Urban Drainage Manual (QUDM) (IPWEAQ, 2016B) is a comprehensive manual for stormwater design and is widely recognised and used within Australia and internationally. The methods adopted within QUDM are particularly useful to determine hydrological design criteria for floodways and have been relied upon for defining the design ARI rainfall events and the corresponding flow rates within the examples of this design guideline.

For the design of a floodway, two design discharge values are required to be selected and satisfied through hydraulic design as follows:

Serviceability design discharge value - Defines the design discharge rainfall event for which the road must remain trafficable for. This design value may vary considerably between level floodway crossings (approximately 0-year ARI rainfall event) and raised floodway types, depending upon level of service and site requirements.

Structural design discharge value - Defines the design discharge rainfall event for which the road must be able to structurally withstand. This would typically be greater than the 100-year ARI rainfall event.

3.8.2 Typical Flow Regimes and Hydraulic Profiles

Supercritical flows occur in waterways due to either a steep channel gradient or a constriction. Increased erosive potential is associated with supercritical flows, therefore, steep channel gradients and constrictions tend to erode over long periods, causing channels to naturally adopt subcritical flow conditions that are stable and steady due to being in an equilibrium state. The construction of a Floodway increases the likelihood of supercritical flow conditions through the introduction of either steep bed gradients or constrictions. The development of supercritical conditions need to be controlled through engineering design to ensure adequate protection against scour exists, particularly at the point where supercritical flows revert to subcritical flows.

For a floodway situated level with the channel, such as Floodway Types 1 and 5 the hydraulic gradient between the tailwater level and the flow over the floodway is approximately equal (Figure 3.17 and 3.18). Further, the water surface profile (WS PF) is greater than the critical flow profile (Crit PF) resulting in a subcritical flow regime which behaves in a stable and predictable manner. Armouring of the channel, such as rock, is still required at the transient point of these floodway types to protect the natural waterway bed from erosion. This erosion results from the increased velocity and bed shear stress associated with the acceleration of flow over the concrete floodway surface as flows are typical more laminar and Manning's coefficient of roughness less than that of the natural channel.

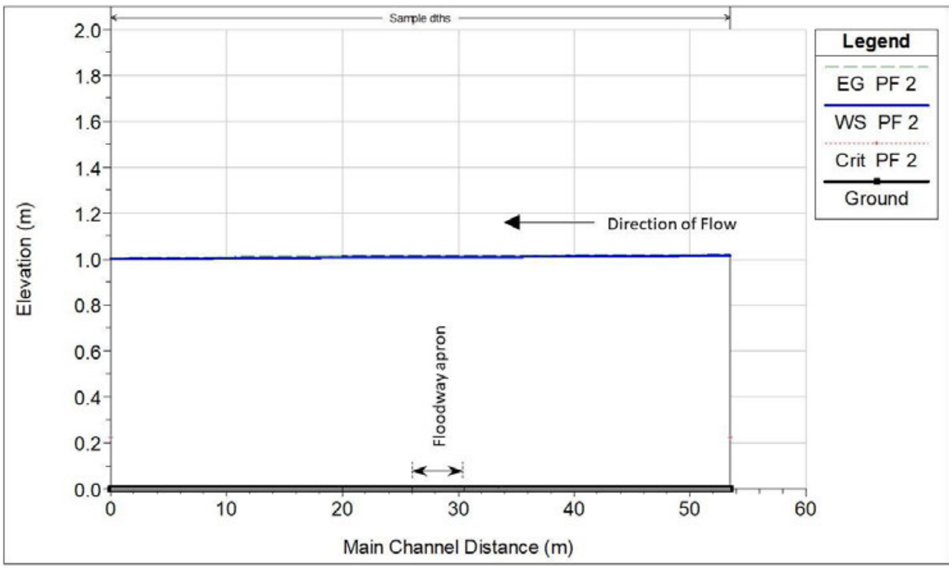


Figure 3.17. Floodway Type 1 typical hydraulic profile.

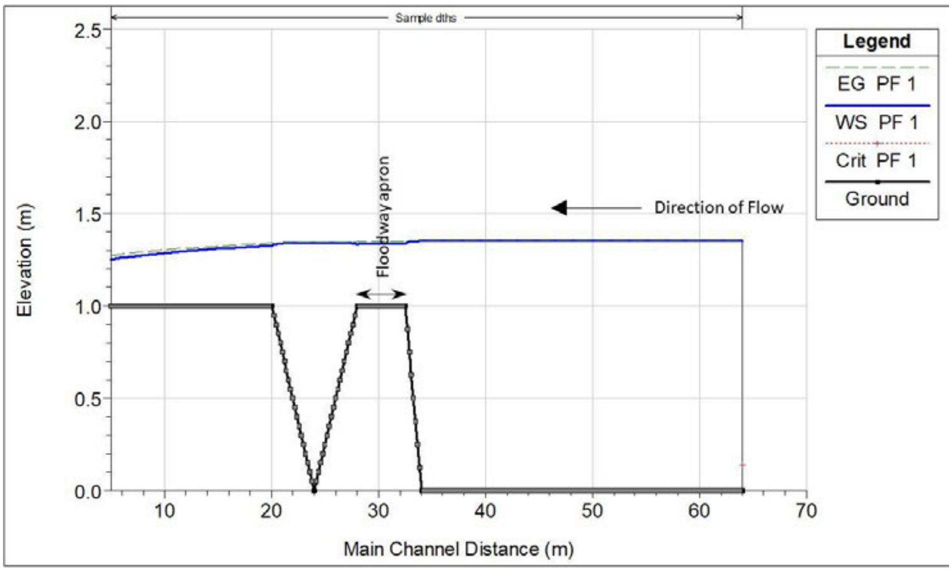


Figure 3.18. Floodway Type 5 typical hydraulic profile.

In a raised floodway, (floodway Types 2, 3 and 4), different downstream flow regimes exist based upon tailwater level. Main Road Western Australia (2006) explains that one of the following three flow regimes are typically experienced as flow is accelerated down the batter of a raised floodway structure:

1) Flow will reach its steady-state velocity corresponding to the maximum velocity while travelling down the downstream batter and prior to entering the tailwater (as described by Manning's equation). On entry into the tailwater the flow is decelerated creating turbulence and an associated hydraulic jump.
2) Flow accelerates while travelling down the batter until it penetrates the tailwater and is decelerated by the turbulence of the hydraulic jump. In this state the maximum velocity occurs immediately before entering the tailwater. As the flow does not reach steady-state velocity the use of Manning's equation will over-estimate the maximum velocity.
3) Supercritical flow reaches the tailwater and remains supercritical until a hydraulic jump occurs downstream. Main Roads Western Australia (2006) states that this condition is rarely observed in practise, furthermore this condition was not reproduced through hydraulic computational modelling for the five floodway types utilised within this guideline.

As the tailwater level rises the hydraulic jump moves upstream until it reaches the downstream batter of the floodway. At this point flow across the floodway tends to plunge into the tailwater, causing turbulence and eddy currents, until the velocity is slowed.

Analysing the hydraulic profiles of raised floodway Types 2, 3 and 4 the normal water surface profile (WS PF) is less than the critical flow profile (Crit PF) when travelling down the downstream floodway batter. The flow in this region is therefore supercritical, which is described as rapid and unstable flow. Supercritical flow reverts to subcritical flow through a hydraulic jump (on contact with the tailwater), representing high energy loss and erosive potential (increased bed shear stress). Protection is required at this point to protect the bed from erosion. The typical downstream protection extent of six (6) meters adopted in Floodway Types 2, 3 and 4 is provided for this purpose, however, the exact extent should be confirmed through modelling (refer to Example 2).

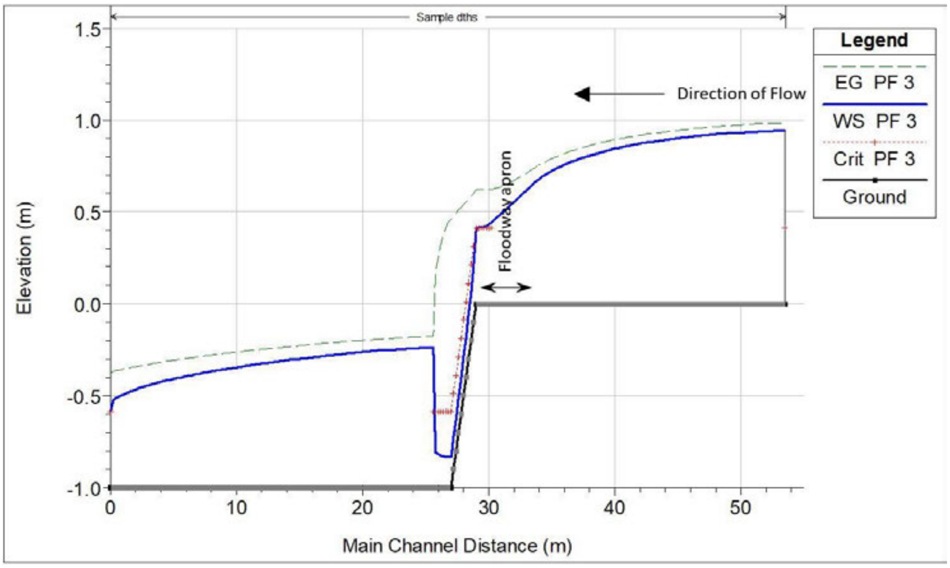


Figure 3.19. Floodway Type 2 typical hydraulic profile.

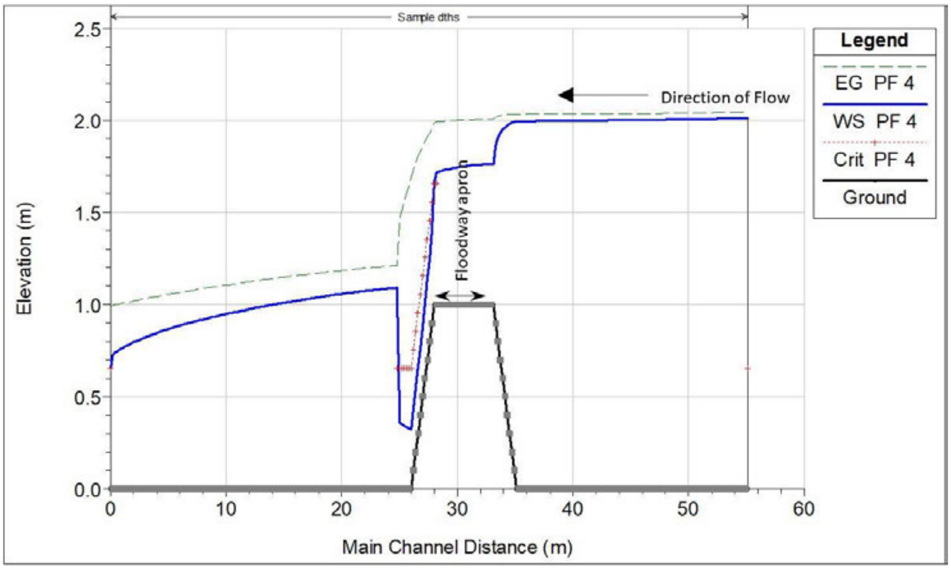


Figure 3.20. Floodway Type 3 typical hydraulic profile.

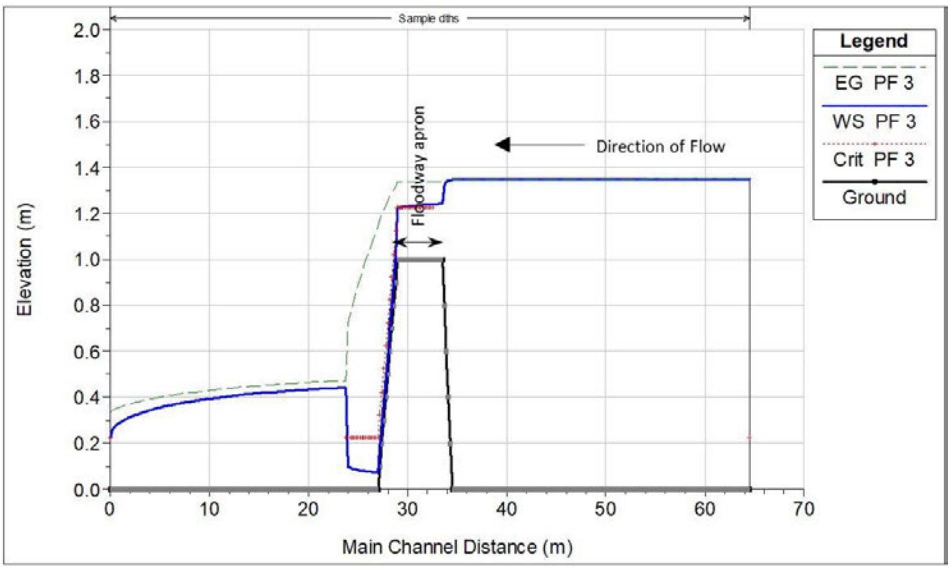


Figure 3.21. Floodway Type 4 typical hydraulic profile.

Figures 3.17 to 3.21 are extracts of hydraulic computational models from Hydrological Engineering Centre River Analysis System (HEC-RAS) (U.S. Army Corps of Engineers Hydrologic Engineering Center, 2019). HEC-RAS is an open-source software package that enables designers to model the hydraulics of water flow through channels and is a crucial tool for the hydraulic analysis of floodway structures.

3.8.3 Open Channel Stage-Discharge Curve

A stage discharge curve plots the relationship of flow versus depth at a particular cross-section within the unrestricted waterway. This curve enables the following basic flow characteristics to be determined:

- Maximum flow capacity of the channel.
- Velocity-discharge relationship.
- Depth and velocity of flow for a specific discharge.

To calculate the velocity of an open channel for a select flow depth Manning's equation (Equation 1) is used. Variables within Manning's equation are based upon a representative cross section of the channel and includes cross-sectional area, wetted perimeter, hydraulic radius, hydraulic grade, and Manning's coefficient of roughness.

$$Q = V \cdot A = \frac{1}{n} \cdot A \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}} \qquad (1)$$

Where: Q = channel discharge (m^3/s).
A = flow area (m^2).
R = hydraulic radius area/wetted perimeter (m).
S = slope of hydraulic grade line.
n = Manning's roughness coefficient.
V = Velocity (m/s).

A typical floodway channel cross-section has three distinct areas (Figure 3.22); the main channel, each immediate riverbank, and the extended flood plain area. In this situation the composite Manning's method should be adopted to determine flow (Equation 2).

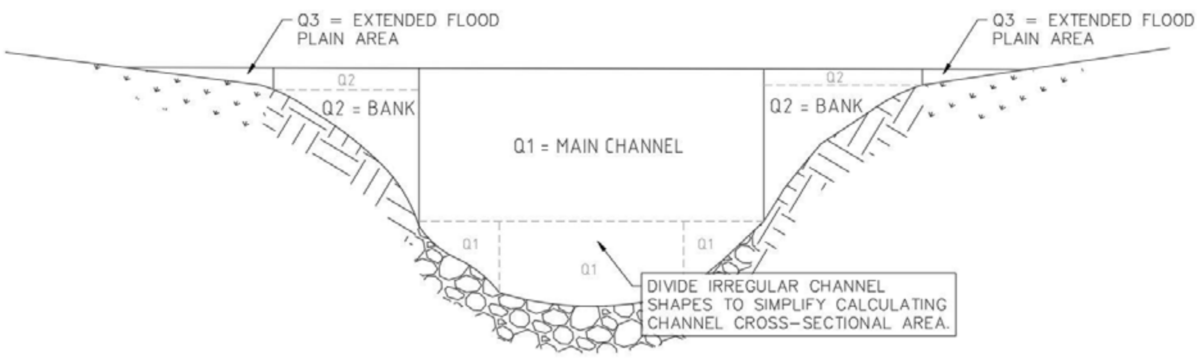


Figure 3.22. Typical watercourse cross-section.

$$Q_T = Q_1 + 2(Q_2) + 2(Q_3) \tag{2}$$

Where: $Q_1,\ Q_2,\ Q_3 = V \cdot A = \frac{1}{n} \cdot A \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$

Design flow velocity can then be calculated by dividing total flow by total cross-sectional area (Equation 3).

$$V_T = Q_T/A_T \tag{3}$$

Where: V_T = Total velocity (m/s).
Q_T = Total flow (m^3/s).
A_T = Total area (m^2).

To determine the maximum flow capacity and the corresponding velocity of the channel the maximum channel depth and width needs to be determined via surveying the channel (Section 3.4). Solving Equation 1 based on these values will provide the velocity for the maximum channel capacity, which can then be solved for the maximum channel discharge. To plot the stage-discharge curve a series of iterations using Equation 1 are required for select depths of 0 metres to the depth corresponding to the maximum channel capacity (Table 3.6). The velocity-discharge relationship of the channel can also be determined by solving each iteration for the corresponding discharge value.

Table 3.4. Stage-discharge curve calculation table.

Depth (m)	**Area, A (m2)**	**Wetted perimeter, p (m)**	**Hydraulic radius, (m)** $R = A \cdot P$	**Velocity, m/s** $V = \frac{1}{n} \cdot A \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$	**Discharge, m³/s** $Q = V \cdot A$
0					
Iteration 1					
Iteration 2					
Iteration 3					
Iteration 4					
Maximum Capacity					

An example of a stage-discharge curve is provided in Figure 3.23.

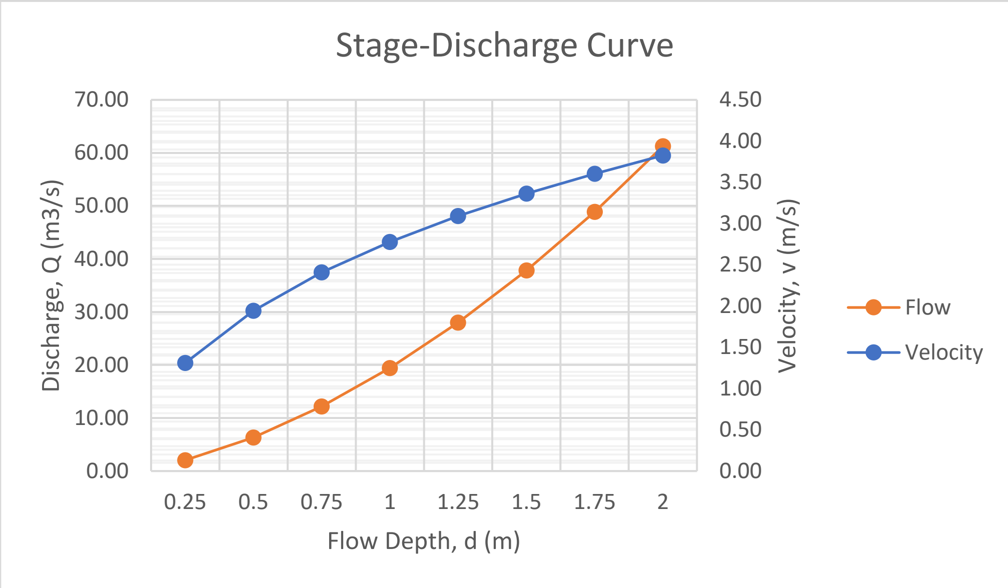


Figure 3.23. A typical stage-discharge curve.

3.8.4 Peak Flow Analysis

The stage-discharge curve is used to determine the design depths and velocities of the unrestricted channel corresponding to the chosen serviceability and structural design discharge values. In the case of an unvented floodway situated level with the channel these flow velocities and depths represent the flow conditions at the floodway as no effect on hydraulic control is present.

3.8.5 Tailwater and Backwater Analysis

Tailwater level is the water level that exists immediately downstream of a drainage structure. The tailwater level provides valuable input into determining the flow regime over a raised floodway structure as it can have a significant influence on rate and height of discharge. Generally, the tailwater level can be determined using Manning's equation (Equation 1) or the stage-discharge curve for uniform flow conditions within regular shaped channels. If the channel is more complex, not regular or flows are not uniform (supercritical) then the tailwater profile and elevation shall be calculated using hydraulic channel modelling software, such as HEC-RAS (U.S. Army Corps of Engineers Hydrologic Engineering Center, 2019) or similar. Example 2, *Hydraulic Modelling Using HEC-RAS*, provides tuition on the use of HEC-RAS to model floodway structures.

Backwater or afflux level is the increase in water level due to the accumulation of water immediately upstream of a raised floodway structure. The utilisation of hydraulic channel modelling software to determine water surface profiles, such as, HEC-RAS is the simplest and most reliable means to calculate backwater level and enables a backwater versus discharge curve to be defined based on a user specified range of design rainfall events. Main Roads Western Australia (2006) details an iterative approach based on hand calculations to calculate backwater level (based on Manning's equation); however, this method is relatively lengthy due to its reliance on a trial-and-error based approach. Refer to Main Roads Western Australia (2006) '*Floodway Design Guideline'* for further details.

Increases in backwater level, if not managed can have a significant effect on upstream assets, land use and infrastructure. The significance of the impact is site specific and is determined via consulting the backwater versus discharge curve obtained through hydraulic modelling to determine the backwater level for a specified design rainfall event. If an upstream asset falls within the area corresponding to the backwater elevation and cannot withstand the proposed effects of floodwaters, then a floodway structure that enables a higher flow capacity should be implemented. This can also be achieved by reducing the specified road level or increasing the waterway area at the location of the floodway.

3.8.6 Hydraulic Design - Trafficability Requirement

Based on the selected serviceability design discharge value, which the floodway is to remain trafficable for, the depth of flow, h (Figure 3.24) needs to be checked to ensure this condition is satisfied. Main Roads Western Australia (2006) details that the height (h) must be less than or equal to the maximum permittable crossing depth of 300 mm for a floodway to remain open and in a serviceable and trafficable state. If the water depth is greater than the maximum permittable crossing depth, then the crossing shall be closed as significant risk to public safety exists.

For this analysis the five standard engineering floodway types outlined within this design guideline are classified into two categories as follows:

Category A floodways - Floodways constructed level with the waterway bed (Figure 3.10) and therefore do not affect the hydraulic control of the waterway, that is, no energy loss across the structure.

- Type 1
- Type 2

Category B floodways - floodways which are raised above the waterway bed (Figure 3.11) and affect the hydraulic control of the channel, that is, an energy loss across the structure is observed.

- Type 3
- Type 4
- Type 5

Category A floodways

For Category A floodways, Manning's formula (Equation 1) can be used to calculate the height (h) of flow over the floodway (Figure 3.24) since the natural channel remains unrestricted. This is determined from the stage-discharge curve of the unrestricted channel for the serviceability design discharge value.

Design discharge and the corresponding ARI rainfall event for which a Category A floodway can remain in service for is typically very low. This is due to Category A floodway's not having any affect over the control of the waterway and culvert structures unable to be incorporated.

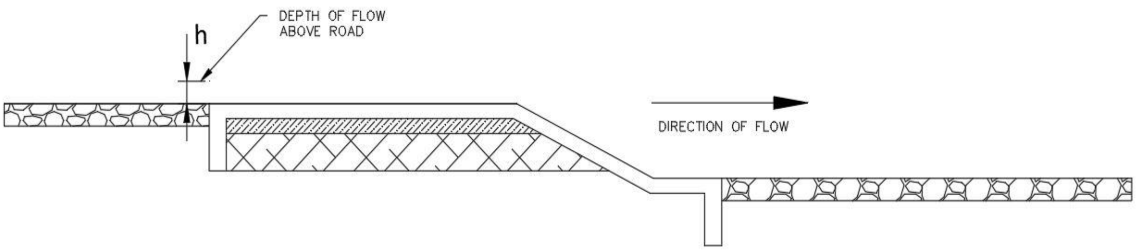


Figure 3.24. Depth of flow, h above the road surface.

Category B floodways

For Category B floodways the Empirical-Broad Crested Weir formula (Equation 4) has been noted in literature as applicable for determining the flow over a raised floodway structure [Rossmiller et al. 1983; IPWEA 2015; Main Roads Western Australia, 2006; Queensland Department of Transport and Main Roads, 2010; Austroads Ltd, 2013; US Army Corps of Engineers Afghanistan Engineer District, 2009].

Initially, a floodway crest level (p) and length of floodway (L) are defined (Figure 3.25).

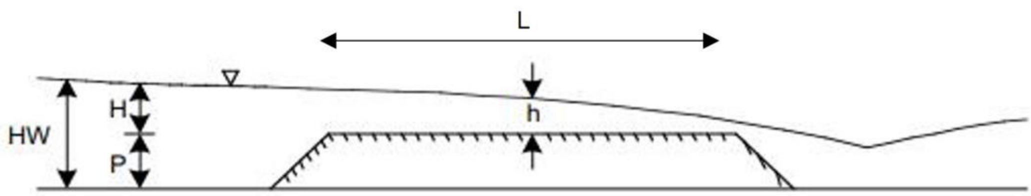


Figure 3.25. Flow profile of a raised floodway (Lohnes et al. 2001).

Where:

HW = Depth of floodway headwater.
P = Height of floodway.
H = Total Upstream head.
L = Length of floodway.
h = Head on roadway.

The assumed allowable head on a roadway (h) is to equal 300 mm or less to allow the safe passage of vehicles (Main Roads Western Australia, 2006).

Discharge (m^3/s) is calculated based on the design of the floodway profile. If the floodway is in an unsubmerged state then the design discharge, Q can be calculated by the Broad-Crested Weir formula with a submergence factor applied (C_s/C_f) (McEnroe et al., 2017). The Broad-Crested Weir formula takes the general form presented in Equation 4 (Department of Transport and Main Roads, 2010).

$$Q = C_f L H^{\frac{3}{2}} \left(\frac{C_s}{C_f}\right) \quad (4)$$

Where

Q = discharge over floodway (m^3/s).
C_f = Coefficient of discharge free flow.
C_s = Coefficient of discharge ow with submergence.
L = Length of floodway (m).

Discharge coefficients are determined by the charts provided in Figure 3.26.

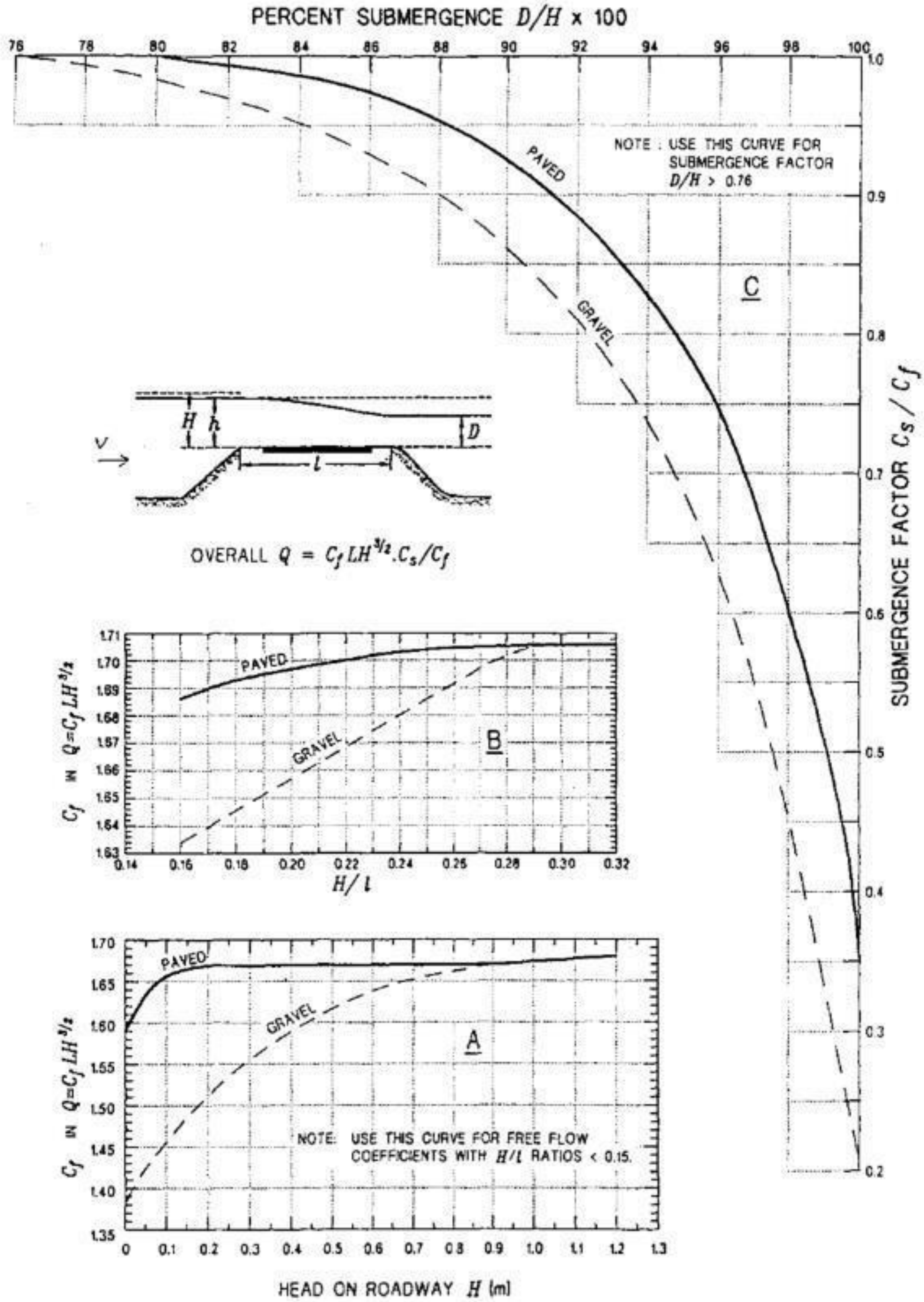


Figure 3.26. Discharge coefficient graph for flows over raised floodways (Department of Transport and Mainroads, 2010).

Where (in reference to Figure 3.26):

h = height between crown and the upstream water surface (m).
l = top width of road formation (m).

H/l, which is determined by:

$$H/l = \frac{h + \frac{v^2}{2g}}{l} \quad (5)$$

Where:

g = acceleration due to gravity (9.81 m/s^2).

Curve B (Figure 3.26) is used with H/l to obtain the free flow coefficient, C_f. If H/l is less than 0.15 then C_f should be obtained by using curve A (Figure 3.26). In addition, if submergence is present, that is, D/H is greater than 0.76 then the percent submergence should be calculated by D/H x 100 and corresponding submergence factor C_s/C_f applied to the Broad-Crested Weir formula.

Once the floodway is in a submerged state it can be analysed as an open channel with no hydraulic control.

Main Roads Western Australia (2006) explains that when the floodway is in an unsubmerged state, the variation of C_f with H/l remains constant and a value of 1.69 can be assumed for C_f. This state only occurs when D/H ratio is less than 0.76 and simplifies Equation 4 to Equation 6.

$$Q = 1.69LH^{\frac{3}{2}} \quad (6)$$

3.8.7 Incorporation of Drainage Structures

Drainage structures such as culverts perform the following functions in floodway design:

- They enable a higher-capacity floodway in terms of discharge to be designed, thus reducing the effect of backwater and associated flooding upstream;
- They prevent the need for flow over the floodway during minor and frequent rainfall events; and
- They prevent stagnation due to standing water upstream of the floodway.

If a drainage structure is incorporated into the floodway structure, then the flow over and through the structure must be apportioned (Figure 3.27) assuming that the backwater level remains equal to the natural channel. That is, the total flow, Q_T is equal to the summation of the flow over the floodway, Q_{top} and the flow through the drainage culverts, Q_c (Equation 8).

$$Q_T = Q_c + Q_{top} \quad (8)$$

Where: Q_T = Total flow in the waterway calculated by Equation 1 (m^3/s).

Q_{top} = flow over top of the floodway calculated by Equation 4 or 6 (m^3/s).

Q_c = flow through the culvert structure (m^3/s).

Once the flow through the proposed culvert is calculated, the culvert can be hydraulically designed through traditional drainage methods outlined in culvert design guidelines. Austroads (2013) provides a specific guide for culvert design, namely, 'Chapter 3 - *Guide to Road Design Part 5B: Drainage - Open Channels, Culverts and Floodway*'.

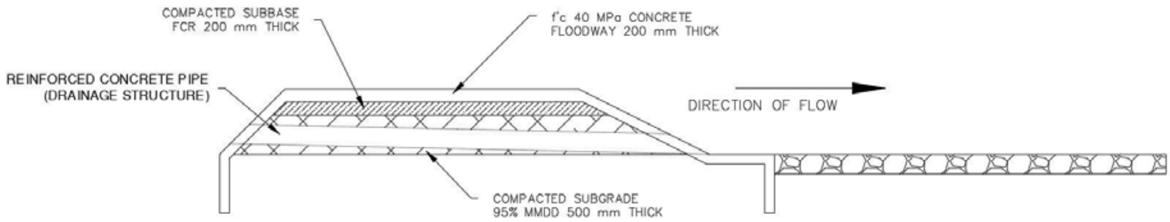


Figure 3.27. Floodway with a significant drainage structure incorporated.

If a drainage culvert only takes a small portion of the total flow (Figure 3.28), then the contribution of that culvert structure can be omitted. This assumption provides a more conservative outcome for design (Main Roads Western Australia, 2006).

Figure 3.28. Floodway with an insignificant drainage structure incorporated.

3.9 SCOUR PROTECTION DESIGN

Permissible velocity based on strata characteristics of the channel bed must be considered to ensure a non-erodible channel exists in the vicinity of the floodway. The adoption of materials based on maximum permissible velocity is a well-established practice within channel design. Several authors such as the U.S. Army Corps of Engineers (1991) have derived tables relating velocity to different channel bed material types (Table 3.7). If a channel bed material differs from this list, then field experience, modelling or laboratory experiments should be conducted to gain reasonable confidence that the channel bed will resist erosion, within the bounds of acceptable maintenance intervals for the floodways serviceable life.

Table 3.5. Maximum permissible mean channel velocities (U.S. Army Corps of Engineers, 1991).

Channel Strata Type	Mean Channel Velocity, m/s
Fine Sand	0.61
Coarse Sand	1.22
Fine Gravel	1.83
Earth	
Sandy Silt	0.61
Silty Clay	1.07
Clay	1.83
Grass-lined channels (slope less than 5%)	
Sandy Silt	1.5
Silt Clay	2.1
Sedimentary or Poor-Quality Rock	3.0
Soft Sandstone	2.44
Soft Shale	1.05
Igneous/metamorphic or Good-Quality Rock	6.0

3.9.1 Scour Analysis

Main Roads Western Australia (2006) provides tables for different rock protection types including their limiting velocities (Table 3.8, 3.9 and 3.10). Based on the flow velocities calculated for the sections of the floodway, rock protection can be assigned by selecting a rock protection type with a limiting velocity greater than the calculated velocity at that section in the design. The limiting velocity, is the velocity at which the critical shear stress is exceeded for the selected protection type, resulting in particle motion i.e. failure of the rock protection.

Table 3.6. Dumped rock riprap protection design values (Main Roads Western Australia, 2006).

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 3.7. Standard classes of rock riprap protection (Main Roads Western Australia, 2006).

Rock Class	Rock Size (m)	Rock Mass (kg)	Minimum Percentage of Rock Larger Than
Facing	0.40 0.30 0.15	100 35 2.5	0 50 90
Light	0.55 0.40 0.20	250 100 10	0 50 90
¼ tonne	0.75 0.55 0.30	500 250 35	0 50 90
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

Table 3.8. Rock mattress protection design (Main Roads Western Australia, 2006).

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

Where, D_{50} refers to 50% of rock in a sample must be less than the specified diameter provided in Table 3.10.

3.9.2 Scour Protection Types

Rock armouring, in almost all instances is required to provide a non-erodible channel bed. Rock protection comes in many different forms such as dumped rock riprap, gabion rock baskets/mattresses and rock pitching with concrete/mortar grouting. Observations suggest that the downstream rock protection followed by the downstream batter are the most susceptible floodway components to be damaged during extreme flood events (refer *"Floodways and Flood-Related Experiences: Survey of Industry and Asset Owners"* (Greene, Lokuge & Karunasena 2020b, pp. 69-75). This is a result of the supercritical flow penetrating the subcritical tailwater causing a hydraulic jump and associated turbulence and eddy currents. This turbulence and eddy currents cause scour, erosion and "popping" of loose rock, unless the channel is appropriately armoured.

Dumped rock riprap

Dumped rock riprap (Figure 3.29) is graded rock placed on a pre-prepared surface with geotextile fabric (440 GSM or greater) typically using an excavator bucket. End dumping directly from a truck is not recommended, particularly on sloped surfaces as segregation occurs. The benefit of dumped rock riprap is that the protection remains flexible and can be easily repaired by placing further rock. It is also a simple construction technique, relatively inexpensive to implement and vegetation will establish as silt and sediment accumulates between the rock voids. Dumped rock riprap, due to its loose nature does have a tendency to "pop" and be transported by floodwaters when velocities are high, particularly at the downstream zone in raised floodway types where sub-critical flows and associated turbulence is present.

Figure 3.29. Typical dumped rock riprap protection.

Gabion rock baskets and mattresses

Gabion rock baskets and mattresses are wire cages filled with graded rock (Figure 3.30). Gabions shall be placed on geotextile fabric with a high GSM rating (440 GSM or greater) to prevent erosion of the founding surface. Gabions are a superior protection choice, compared to dumped rock riprap as the wire baskets provide anchorage and prevent scouring when flow velocities are high. As a result of the increased anchorage, gabions permit the use of lower quality and less dense rock, reducing quarry costs. Gabions are, however, more labour intensive, expensive to implement, less flexible than dumped rock riprap and are more expensive to repair. The wire can also be susceptible to corrosion and abrasion over time, causing premature failure.

Figure 3.30. Typical gabion rock basket protection.

The gabion baskets should be anchored to the founding surface and connected to the concrete floodway cut-off wall via a galvanised steel angle with 200 mm long, M12 galvanised bolts chemical set at 800 mm centres (Figure 3.31).

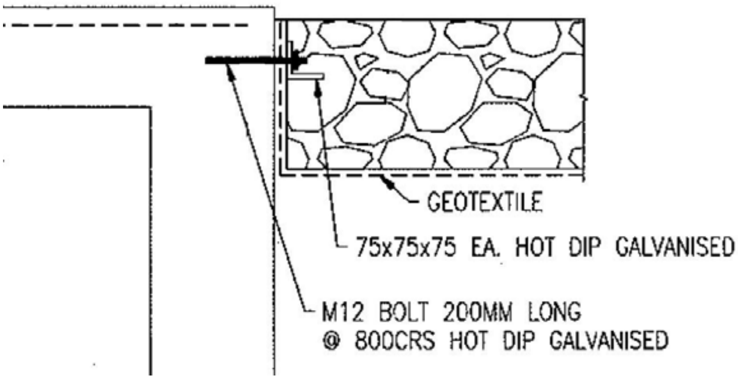


Figure 3.31. Rock filled wire mattress connection detail (LVRC 2008a).

Rock pitching

Rock pitching is rock protection, however, the voids are filled with concrete (Figure 3.32). This is a rigid monolithic form of rock protection and is the most expensive to implement. Rock pitching provides the best anchorage for rock in high flow applications. It is also useful if the natural channel consists of dispersive soils or if significant quantities of large quality rock is not available within the local area. Due to its rigid nature it is susceptible to undermining, which can cause premature failure. This option is also expensive to rectify in comparison to the other protection types.

Figure 3.32. Typical rock pitching with concrete protection.

Natural Vegetation

Natural vegetation lined channels are not recommended as adequate protection against scouring during extreme flood events.

3.10 STRUCTURAL DESIGN CHARTS

To achieve a flood-resilient design, the concrete floodway superstructure needs to be designed to sustain ultimate flood loading determined from the peak flow analysis and as experienced during extreme flood events. This section presents a simplified structural design method that utilises design charts based on the critical section to determine steel reinforcement requirements which satisfy strength, serviceability, and durability during extreme flood loading. For further details on the numerical finite element modelling, parametric analysis and simulation methods used to derive the structural design charts refer to *"Structural design of floodways under extreme flood loading"* (Greene et al. 2020a, pp. 535-555).

3.10.1 Worst Case Loading Scenario

Loading Condition

Floodway loadings from pressures such as hydrostatic, debris, vehicular, drag, impact and lift were investigated through a parametric analysis. From this study impact from a four-tonne boulder was determined as the most significant source of loading and therefore the primary predictor of floodway failure during extreme flood conditions (worst-case scenario). Boulder impact is a phenomenon which occurs during flooding where large rocks (boulders) are introduced into the flow through landslides, bank and channel erosion. These boulders progressively move downstream in a rolling motion for as long as the velocity of the watercourse exceeds the critical velocity of the boulder (Furniss et al., 2002). This loading case also had qualitative agreement with the survey respondents (refer *"Floodways and Flood-Related Experiences: Survey of Industry and Asset Owners"* (Greene, Lokuge & Karunasena 2020b, pp. 69-75) and with various sources of literature summarising the critical conditions for the movement of boulders based on field observations [Van Rijn (2019); Inbar and Schick (1979); Turowski et al. (2009) and Fahnestock (1963)].

AS 5100.2, "*Bridge Design, Design Loads"* (Standards Australia, 2017B) provides a design force calculation for a floating log impacting a bridge superstructure. This formula is an equation of work with force equal to the kinetic energy of the object impacting the structure. This equation assumes that an object such as a log is buoyant and therefore moving at the same velocity as flow. That is the velocity of flow is equal to the velocity of the object and no net acceleration is present. The movement of boulders due to floodwaters is a much more complex phenomenon as they are submerged and roll, slide and saltate along the waterway channel in both steady and unsteady state flow conditions.

The movement of boulders occurs once drag force exceeds the frictional force exerted by the channel bed. During the arrival of floodwaters, large fluctuating accelerations in flow occur. These accelerations exert a very large impulsive force on stationary objects, such as boulders because of loss of fluid momentum. It is these velocity fluctuations in conjunction with drag force from the floodwater that contribute to the movement of boulders during flooding (Alexander & Cooker, 2016). Furthermore, this is exacerbated by the considerably high bulk density of sediment laden floodwaters that increase buoyancy, decreasing the force required to move an object. Boulders move slower than the velocity of the water column due to their intermediate contact with the channel, shear mass and their spherical geometry (Fondriest Environmental, 2014). As the head of the floodwaters advances past the boulder a lower net contribution from impulsive force results as flow velocity is generally more consistent (Alexander & Cooker, 2016). A factor of 0.5, aligning with the coefficient of drag for a near-spherical boulder was applied to the impact force equation within AS 5100.2, "*Bridge Design, Design Loads"* (Standards Australia, 2017B). This providing a worst-case estimation for the velocity of a submerged boulder which is either in intermediate contact with the channel (with friction force) or fully mobile in the floodwaters (without friction force) and with movement a resultant of drag force, impulsive force, buoyancy, and mass.

This approach also coincides with the maximum impact value considered in AS 5100.2 (Standards Australia, 2017B), which is limited to the impact from a maximum 2-tonne log (0.5 factor multiplied by the boulder weight of 4-tonnes equals 2-tonnes). This, therefore, provides a relevant worst case loading scenario for all waterways even ones which boulders are not present in.

Floodway geometry

The geometric conditions adopted in the design charts are outlined below and were found to have the most significant contribution to the worst-case loading scenario for each of the five standard floodway types.

- A cut-off wall depth of 900 mm. This was the smallest depth trialled and as a result provided the least distribution area to the adjoining soil. This significantly reduces the structure's ability to resist horizontal loading (such as impact) in the direction of flow.
- No rock protection or soil adjacent the downstream cut-off wall. This simulating a downstream head cut contacting the floodway structure, or a floodway left unmaintained after several flood events. In the case of level floodway types, no upstream rock protection was present to allow a surface perpendicular to flow to be present for the impact loading.
- A flow depth of 2 m. This depth provided the greatest contribution to hydrostatic pressure.
- An impact loading from a 4-tonne boulder.
- Soil type was discovered to have a large variable influence on the displacement and stress results. Therefore, three different soil type categories were selected for the structural design procedure. This allows the structural design method to align with a range of in-situ soil conditions specific to the floodway site locality.

3.10.2 Bending Moment and Shear Force Locations

Bending Moment

The location of the critical cross-section relating to the maximum design bending moment was found to occur near the midpoint of the downstream cut-off wall (Figure 3.33) and originates because of the horizontal loading from the boulder impact considered in the worst-case loading scenario.

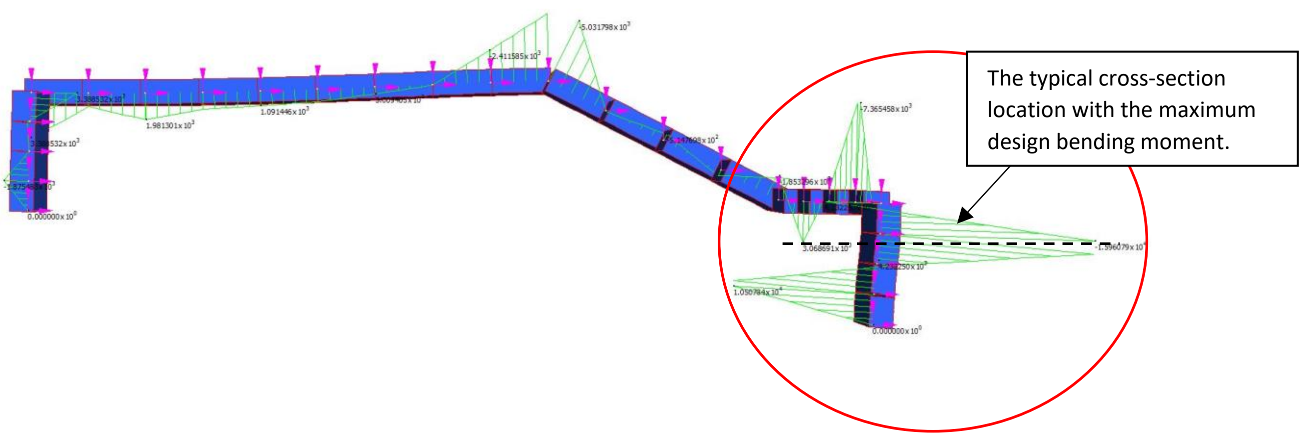


Figure 3.33. Typical cross-section location for design bending moment, M*.

Shear Force

The location of the critical cross-section relating to the maximum design shear force was also found to occur near the midpoint of the downstream cut-off wall (Figure 3.34) and originates because of the horizontal boulder impact loading considered within the worst-case loading scenario.

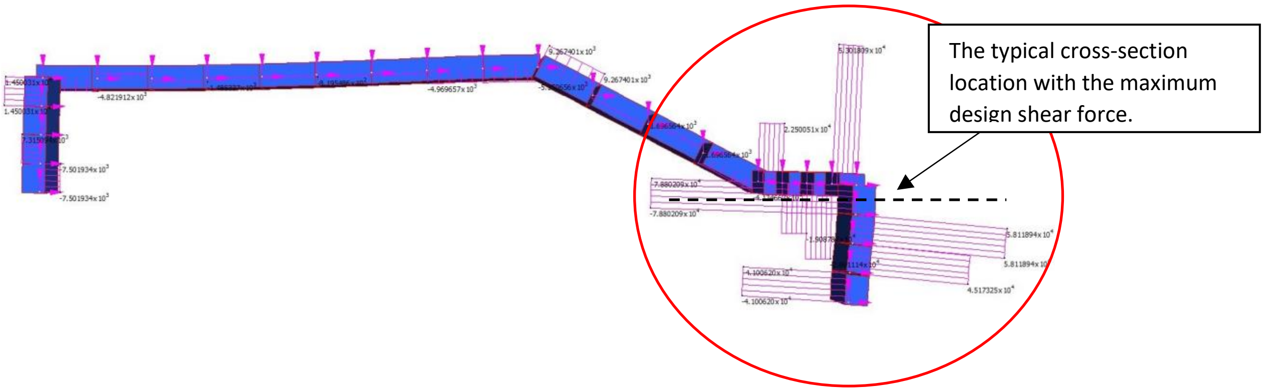


Figure 3.34. Typical cross-section location for design shear force, V*.

3.10.3 Use of Structural Design Charts

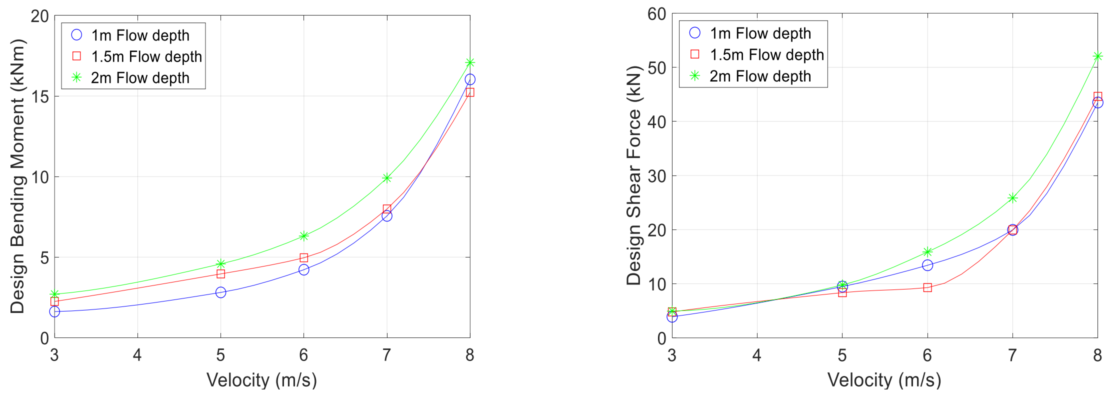
The design bending moment and shear force values within the structural design graphs represent the ultimate limit state values for the floodway structure during extreme flooding and are based on three commonly encountered soil types for a range of different flow velocities and depths. If an intermediate value is required linear interpolation can be utilised. Design of the floodway structural elements can then be conducted by referencing these charts and in conjunction with AS 5100.5 (Standards Australia, 2017A). The mechanical properties of soil are determined at the time of field inspection (Section 3.4). Table 3.3 has been reproduced as Table 3.11 to assist in selecting the appropriate soil type within the structural design graphs.

Table 3.9. Mechanical properties of the water courses in-situ soil.

Material Type	**E (MPa)** [3]	ν [4]	**ρ (kg/m^3)** [5]	**c' (MPa)** [6]	**ϕ (°)** [7]	**K0** [8]	**e** [9]
Soil 1: Silty Sand (Silts and silty sands)	40	0.3	1,700	0.01	25	0.426	0.4
Soil 2: Sandy Soil (Poorly graded sands with fines and gravely-sand-silt mixtures)	30	0.25	1,800	0.075	34	0.44	0.3
Soil 3: Clay Soil (clays of varying plasticity and clayey fine sands and silts)	100	0.3	1,900	0.01	20	0.658	0.15

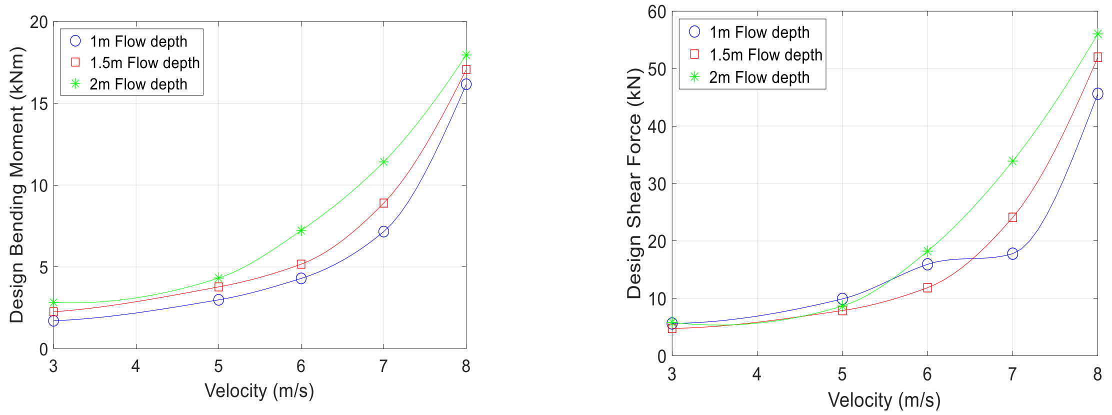
[3] Modulus of Elasticity (MPa)
[4] Poisson Ratio
[5] Density (kg/m^3)
[6] Cohesion (MPa)
[7] Angle of friction (degrees)
[8] Horizontal stress ratio
[9] Void ratio

3.10.4 Floodway Type 1



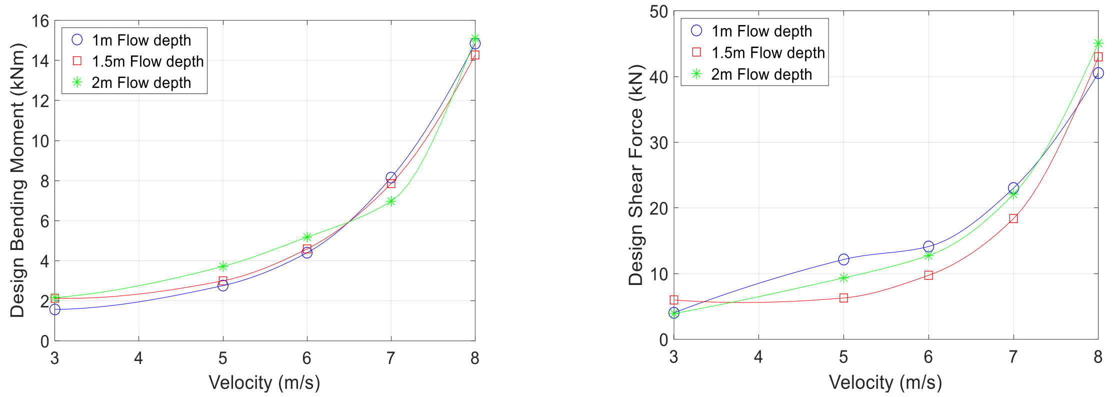
(a) (b)

Figure 3.35. Type 1, Soil Type 1 (a) Bending moment and (b) Shear force design charts.



(a) (b)

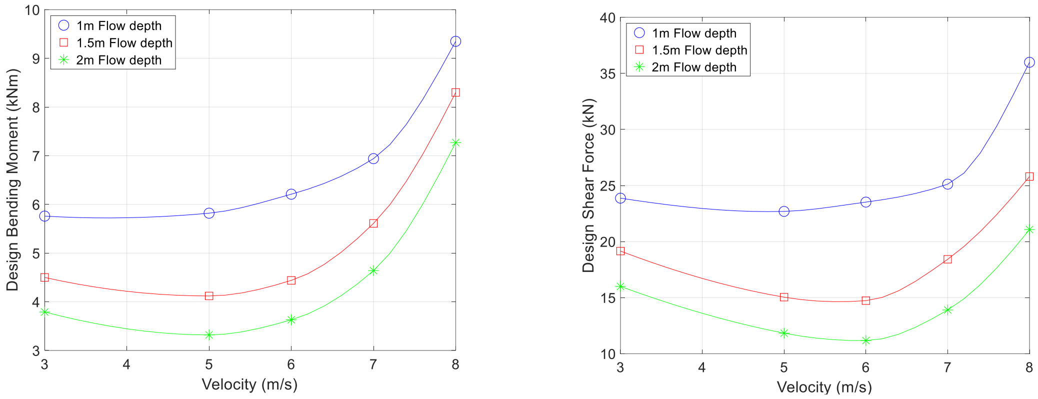
Figure 3.36. Type 1, Soil Type 2 (a) Bending moment and (b) Shear force design charts.



(a) (b)

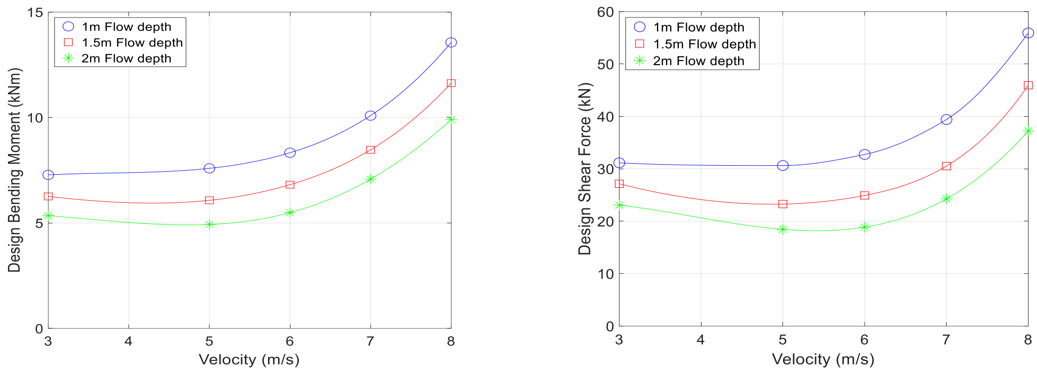
Figure 3.37. Type 1, Soil Type 3 (a) Bending moment and (b) Shear force design charts.

3.10.5 Floodway Type 2



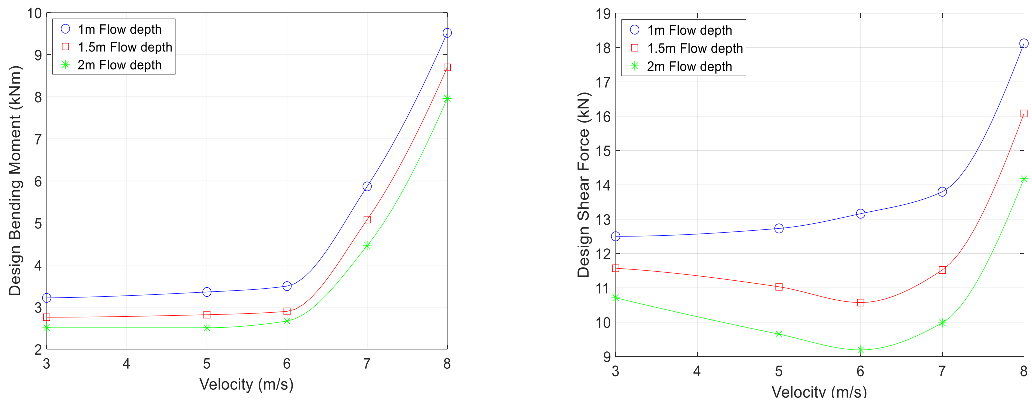
(a) **(b)**

Figure 3.38. Type 2, Soil Type 1 (a) Bending moment and (b) Shear force design charts.



(a) **(b)**

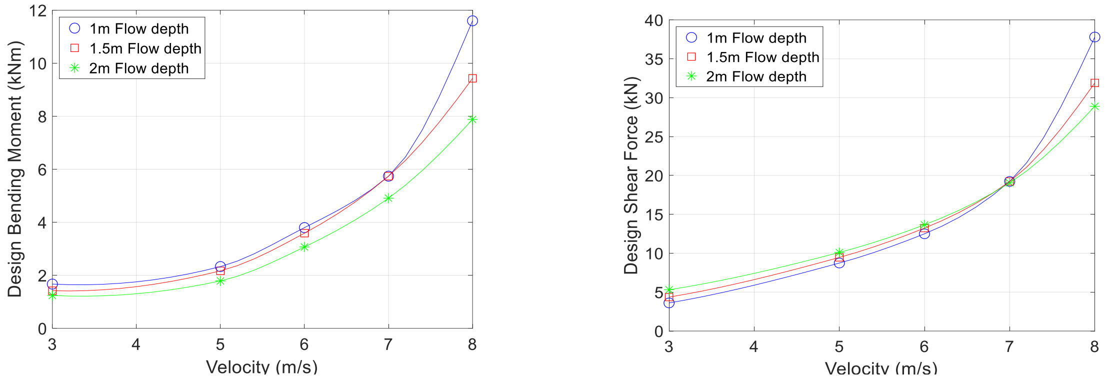
Figure 3.39. Type 2, Soil Type 2 (a) Bending moment and (b) Shear force design charts.



(a) **(b)**

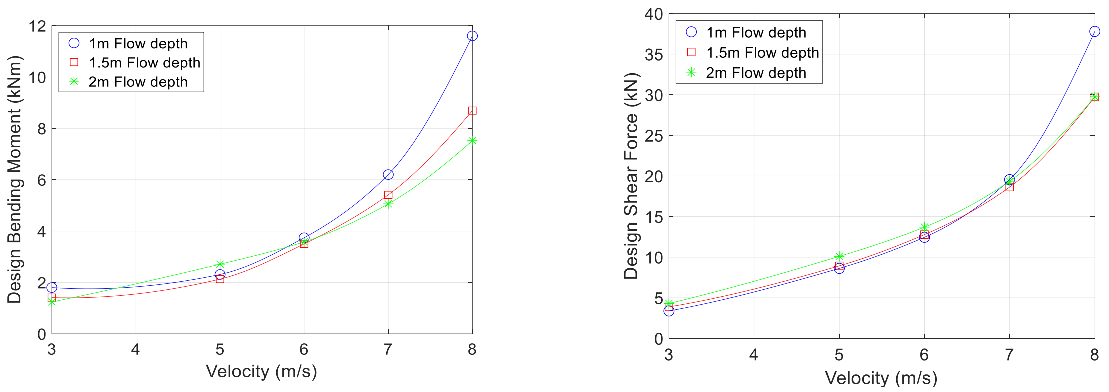
Figure 3.40. Type 2, Soil Type 3 (a) Bending moment and (b) Shear force design charts.

3.10.6 Floodway Type 3



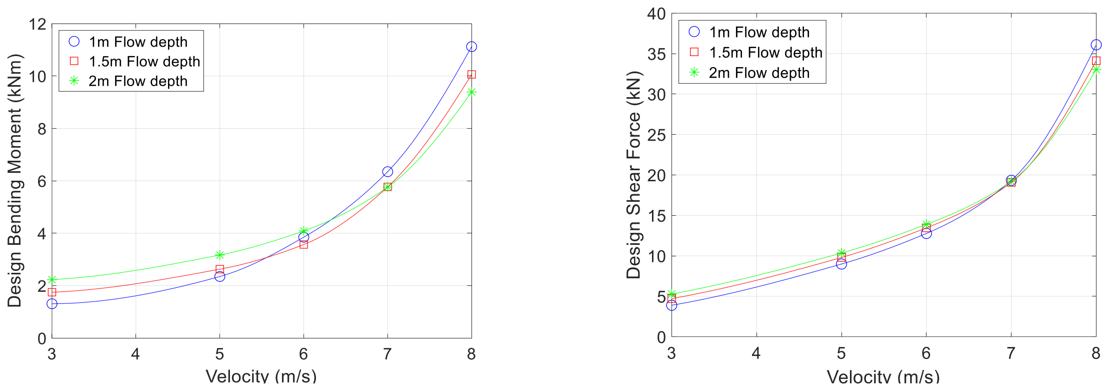
(a) (b)

Figure 3.41. Type 3, Soil Type 1 (a) Bending moment and (b) Shear force design charts.



(a) (b)

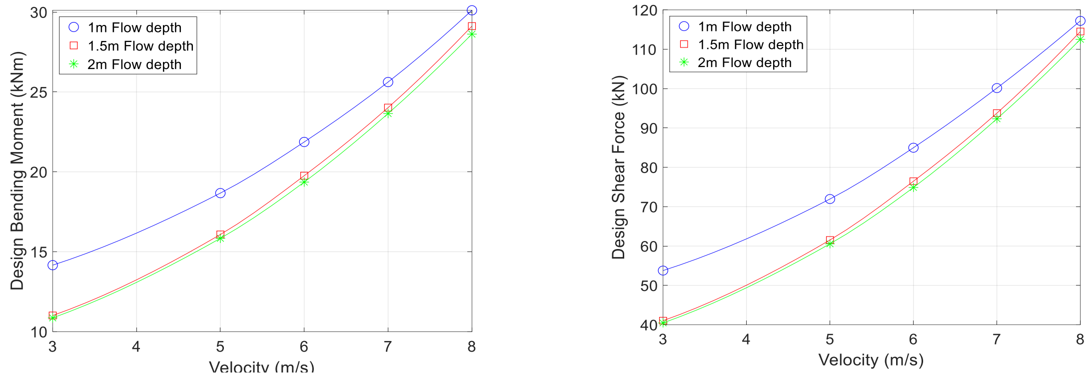
Figure 3.42. Type 3, Soil Type 2 (a) Bending moment and (b) Shear force design charts.



(a) (b)

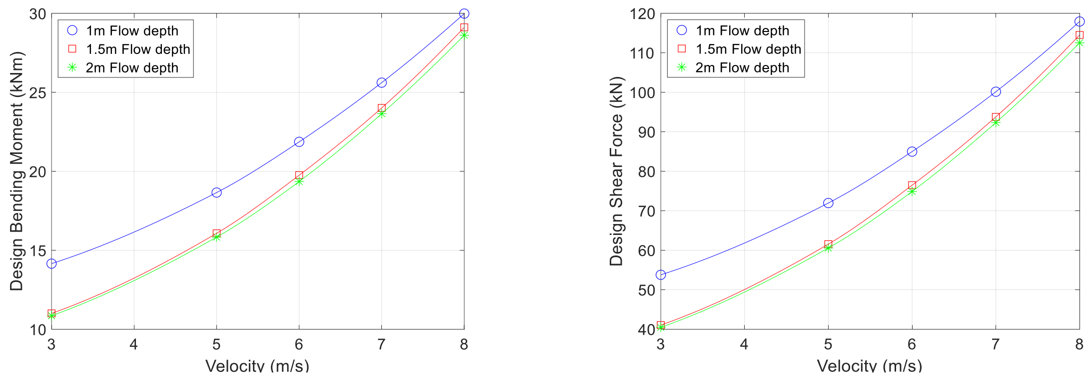
Figure 3.43. Type 3, Soil Type 3 (a) Bending moment and (b) Shear force design charts.

3.10.7 Floodway Type 4



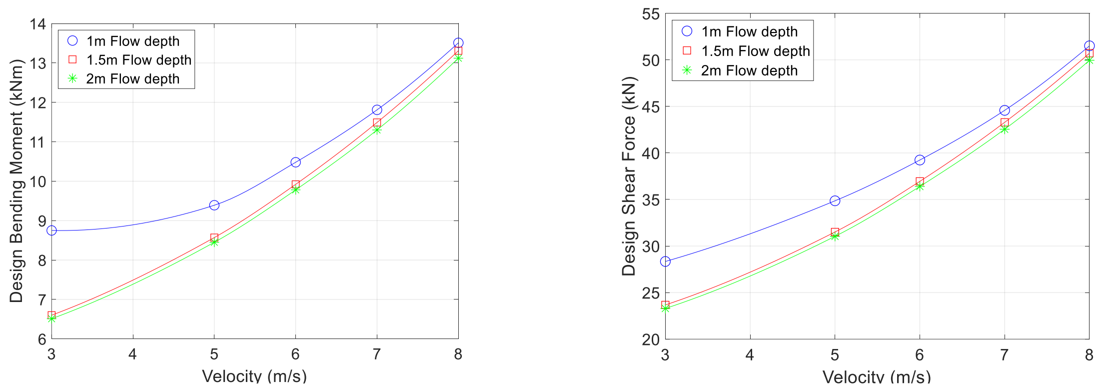
(a) **(b)**

Figure 3.44. Type 4, Soil Type 1 (a) Bending moment and (b) Shear force design charts.



(a) **(b)**

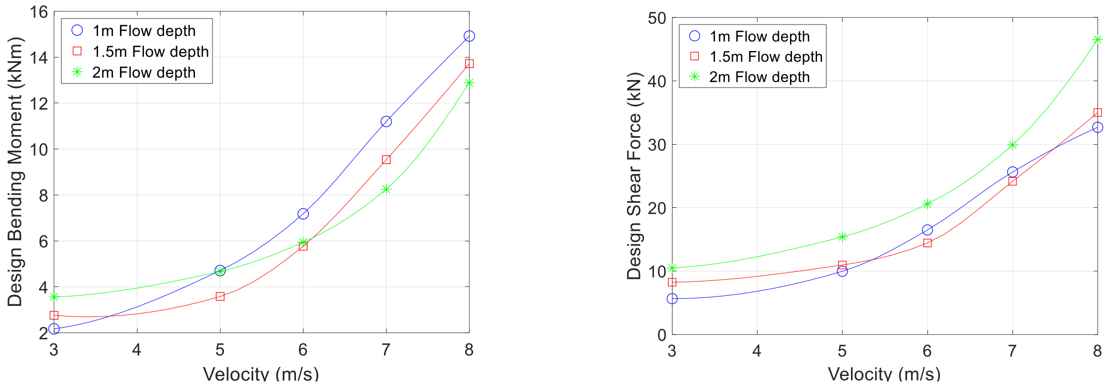
Figure 3.45. Type 4, Soil Type 2 (a) Bending moment and (b) Shear force design charts.



(a) **(b)**

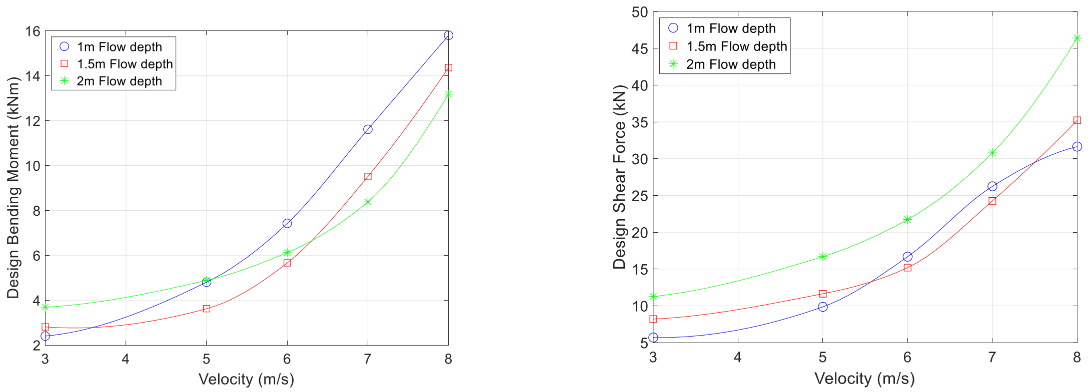
Figure 3.46. Type 4, Soil Type 3 (a) Bending moment and (b) Shear force design charts.

3.10.8 Floodway Type 5



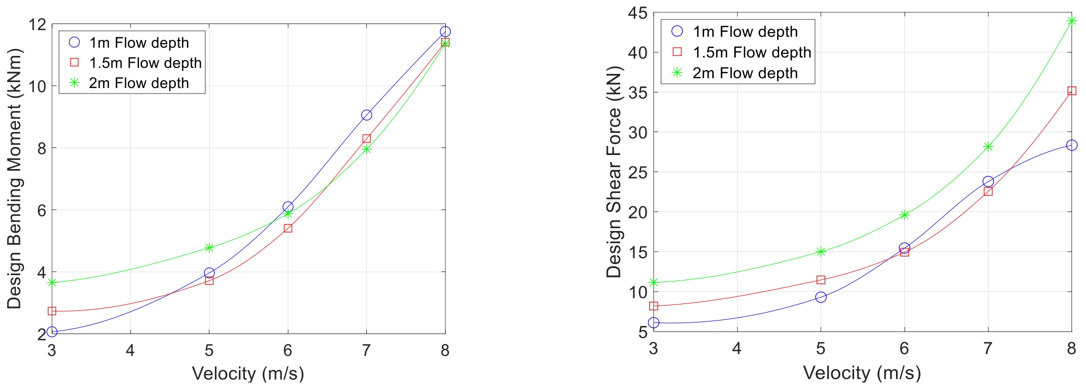
(a) **(b)**

Figure 3.47. Type 5, Soil Type 1 (a) Bending moment and (b) Shear force design charts.



(a) **(b)**

Figure 3.48. Type 5, Soil Type 2 (a) Bending moment and (b) Shear force design charts.



(a) **(b)**

Figure 3.49. Type 5, Soil Type 3 (a) Bending moment and (b) Shear force design charts.

3.11 STRUCTURAL DESIGN METHOD

The structural design method aims to select the cross-sectional area of steel and concrete that satisfies requirements for strength, serviceability, durability and fatigue in accordance with the requirements of AS 5100.5 (Standards Australia, 2017A).

Design parameters utilised in this section include:

1. Maximum bending moment and shear force
    - Soil type.
    - Maximum design flow velocity.
    - Maximum design flow depth.
2. Strength parameters of concrete.
3. Strength parameters of steel.

3.11.1 Durability for Concrete

Concrete quality requirements for durability shall be determined in accordance with Section 4 of AS 5100.5 (Standards Australia, 2017A) and is based on the most severe exposure classification of any of the floodway concrete surfaces.

The exposure classification must be determined in accordance with Section 4.3 of AS 5100.5 (Standards Australia, 2017). The minimum exposure classification of any floodway design shall be B1. This exposure classification is a result of floodway members been subjected to constant wetting and drying from being in contact with fresh water (chloride content <300 ppm). If members are in occasional contact with brackish , then the exposure classification shall be increased to B2 (chloride content 300-6000 ppm).

The minimum compressive strength of concrete shall be 40 MPa as the pavement is subjected to pneumatic tyre traffic (Table 4.6, AS 5100.5, 2017). A higher compressive strength may be required based on the specific floodway surface and exposure environment.

The minimum cover for 40 MPa concrete for corrosion protection, assuming standard formwork and compaction techniques shall be applied in accordance with Table 4.14.3.2 of AS 5100.5 (Standards Australia, 2017A) as follows:

- 45mm for all exposure class B2 applications.
- 60mm for all exposure class C applications.

All surfaces that are in contact with the ground shall be increased by the following in accordance with Table 4.14.3.5 of AS 5100.5 (Standards Australia, 2017A):

- 10 mm if damp-proof membrane is used; or
- 30mm if not.

During placement, the concrete shall be adequately protected from moisture loss. AS 5100.5 (Standards Australia, 2017A), clause 4.4.2 describes several adequate curing techniques including moist curing, membrane curing, polyethylene sheet curing and through retaining the formwork in place. One of these methods shall be adopted to ensure moisture is obtained until the commencement of curing, enabling target compressive strength to be achieved.

If the concrete floodway structure is to be situated in an aggressive soil as defined in clause 4.8 of AS 5100.5 (Standards Australia, 2017A) then revision of exposure classification shall be determined by referencing Table 4.8, AS 5100.5 (Standards Australia, 2017A).

Concrete mix design shall be in accordance with AS 5100.5 Table 4.4.1(A)/(B) (Standards Australia, 2017A).

3.11.2 Design for Strength and Serviceability in Bending

The strength and serviceability in bending for the cross-section subjected to the greatest bending moment under extreme flood conditions shall be determined in accordance with Section 8 of AS5100.5 (Standards Australia 2017A). Design for strength and serviceability in bending is an iterative process and requires the designer to make an initial estimate of tensile steel reinforcement required. If a significantly large difference exists between moment capacity (M_u) and design moment (M^*) for the initial estimated steel reinforcement, further refinement based on an iterative process is required.

Both positive and negative bending moments exist within the floodway structure (Figure 3.33) and reinforcement should be provided at both the inner and outer faces. The presence of other bending moments originating from loadings such as vehicular movement also exist, albeit at a smaller magnitude (not illustrated in Figure 3.33). Reinforcement requirements should, therefore, be adopted uniformly throughout the structure and designed in accordance with the worst-case design bending moment value, M* which corresponds to the critical case and is obtained from the design graphs.

The design procedure for bending reinforcement can be summarised as follows and is schematically represented in Figure 3.50.

- Determine the design bending moment value, M^* in accordance with the design chart.
- Check if moment capacity (M_u) based on an assumed tensile reinforcement is satisfactory to resist the design bending moment (M^*).
- Check minimum area of steel reinforcement required to satisfy minimum ultimate strength in bending requirements.
- Check minimum area of steel reinforcement required to satisfy crack control against shrinkage and temperature effects.

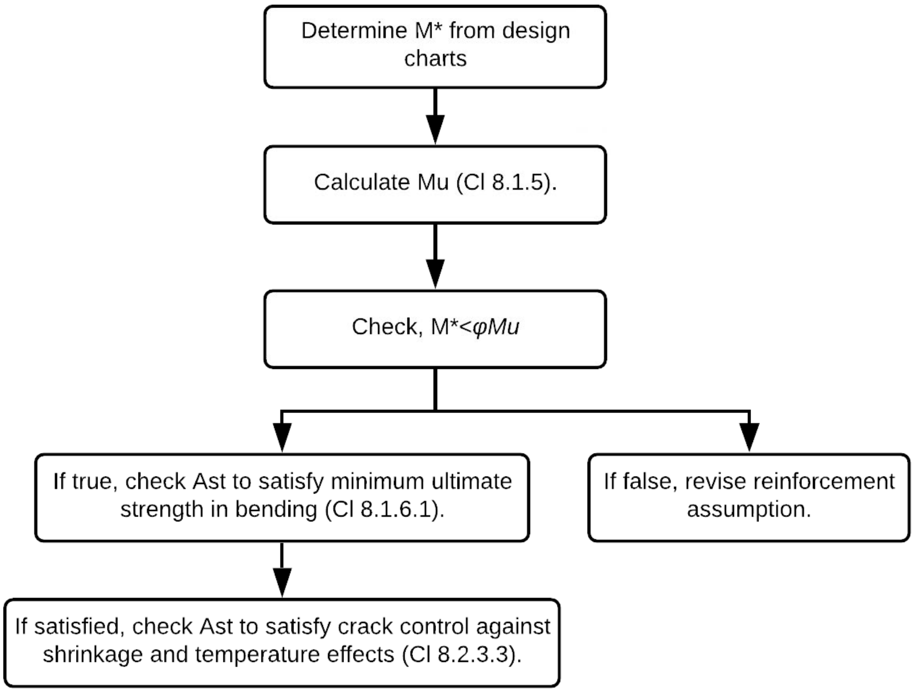


Figure 3.50. Bending reinforcement design schematic.

A rectangular stress block of the beam cross-section (Figure 3.51) is utilised to simplify the stress-strain profile of concrete in bending and incorporates equilibrium considerations. The rectangular stress block utilises the neutral axis as the datum to determine the assumed equivalent stress distribution.

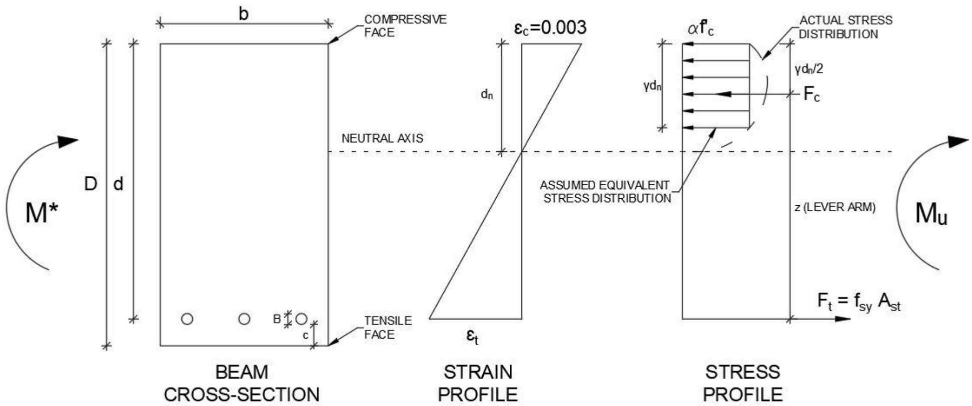


Figure 3.51. Equivalent concrete stress block.

*M** shall be determined in accordance with the design charts, where *M** is the critical design bending moment within the floodway structure under extreme flood loading conditions.

Step 1: Check if moment capacity (M_u) based on an assumed tensile reinforcement is satisfactory to resist the design bending moment (*M*)*. Note, compression reinforcement is ignored in the design of the doubly reinforced section.

Calculate the compressive force in concrete, F_c i.e. the volume of the assumed equivalent stress distribution:

$$F_c = (\alpha f'_c)\ (\gamma d_n)\ b \qquad (9)$$

Where:

α = ratio of concrete compressive strength (equation 8.1.3(1), AS 5100.5 (Standards Australia, 2017A).
γ = ratio of stress block depth (equation 8.1.3(2), AS 5100.5 (Standards Australia, 2017A)).
f'_c = concrete compressive strength (MPa).
d_n = depth to neutral axis (mm).
b = beam width (mm).

Calculate tensile forces in steel (assuming steel yields), F_t:

$$F_t = A_{st} f_{sy} \qquad (10)$$

Where:

A_{st} = cross sectional area of longitudinal tensile reinforcement (mm^2).
f_{sy} = characteristic yield strength of steel (MPa).

Equate F_t and F_c and solve for d_n to determine the depth to neutral axis:

$$F_t = F_c \qquad (11)$$

With d_n evaluated the lever arm can be calculated:

$$z = D - c - B/2 - \gamma \cdot d_n/2 \qquad (12)$$

Where:

z = lever arm (mm).
D = Beam depth (mm)
c = minimum cover for corrosion protection (mm).
B = diameter of the reinforcement (mm).

Calculate the moment capacity, M_u:

$$M_u = F_t z = F_c z \qquad (13)$$

Check if moment capacity (M_u) based on an assumed tensile reinforcement is satisfactory to resist the design bending moment (M^*) from design graphs for the given cross-section:

$$\varphi M_u \geq M^* \qquad (14)$$

Step 2: Check minimum area of steel reinforcement required to satisfy minimum ultimate strength in bending requirements at the critical section in accordance with clause 8.1.6.1.

$$A_{st} \geq [\alpha_b\, (D/d)^2 f'_{ct.f}/f_{sy}] b_w\, d \qquad (15)$$

Where:

α_b = 0.20 for rectangular sections.

Step 3: Check minimum area of steel reinforcement required to satisfy crack control against shrinkage and temperature effects. AS5100.5, Clause 9.4.3 only provides guidance for fully restrained slabs (Standards Australia, 2017A). Since a floodway structure is free to expand and contract clause 9.5.3.3, *reinforcement in the secondary direction in unrestrained slabs* in AS3600:2018 is more appropriate to the application of floodways and shall be satisfied.

$$A_{st} \geq 0.00175bD \qquad (16)$$

3.11.3 Design for Shear

The strength and serviceability in shear of the cross-section subjected to the greatest shear force when acting under extreme flood conditions shall be determined in accordance with Section 8.2 of AS5100.5 (Standards Australia 2017A). The design procedure for shear reinforcement can be summarised as follows and schematically represented in Figure 3.52.

- Determine the design shear force value, V^* in accordance with the design charts.
- Calculate the contribution of concrete to shear strength, V_{uc}. AS 5100.5:2017, clause 8.2.1.6 does not mandate shear reinforcement for sections under 300 mm (Standards Australia 2017A). For sections greater than 300mm minimum shear reinforcement is required if $V^* > 0.5\varphi V_{uc} < V_{u.max}$.
- If $V^* > V_{u.max}$, shear strength is limited by web crushing and beam dimensions should be increased to eliminate web crushing failure.
- If reinforcement is required, determine the required spacing and check if the calculated spacing is more than the minimum spacing.

In most floodway design cases the shear strength of concrete alone (V_{uc}) provides adequate resistance against the design shear forces (V^*) as determined within the shear force design graphs. In most other occurrences $V^* < 0.5\varphi V_{uc}$ or the section is less than 300 mm deep and therefore shear reinforcement is not required in accordance with AS 5100.5, clause 8.2.1.6 (Standards Australia, 2017A). Confirmation of this should be undertaken by the designer for the specific design shear force (V^*) value obtained from the design graphs.

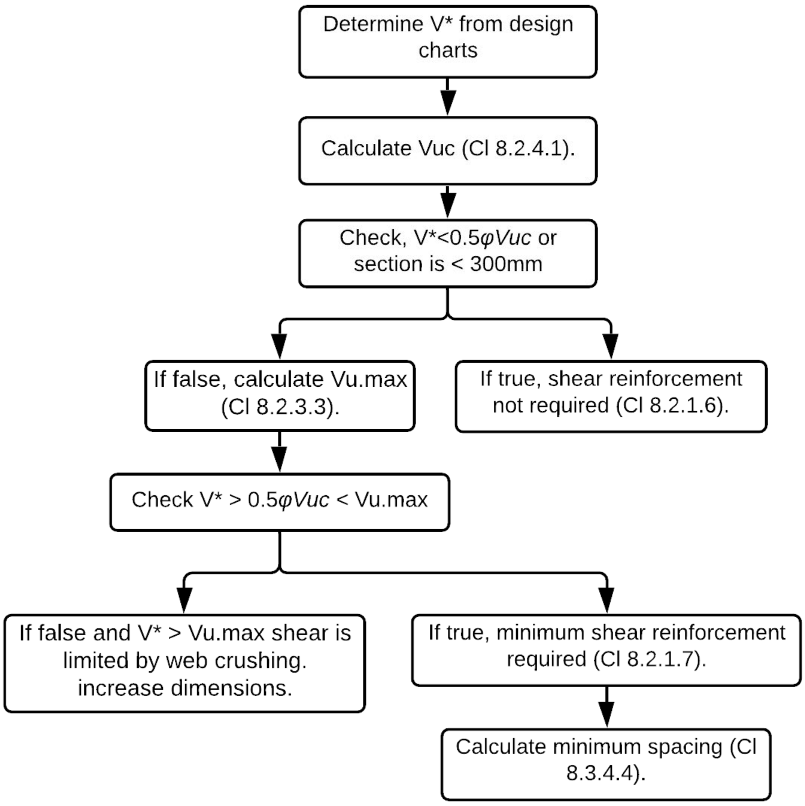


Figure 3.52. Shear reinforcement design schematic.

*V** shall be determined in accordance with the design charts, where V* is the critical design shear force within the floodway structure under extreme flood loading conditions.

Step 1: Calculate the contribution of concrete to shear strength, V_{uc}.

$V_{uc} = k_v b_v d_v (f'_c)^{1/2}$ (eq. 8.2.4.1, AS 5100.5 (Standards Australia, 2017A)) (17)

Where:

k_v = using the simplified method[10] (clause 8.2.4.6, AS 5100.5 (Standards Australia, 2017A)).
b_v = effective width of a web for shear (mm).
d_v = effective shear depth, greater of 0.72D or 0.9d (mm) (clause 8.2.1.9, AS 5100.5 (Standards Australia, 2017A)).

Step 2: Check if section is < 300 mm in depth or that $V^* < \varphi V_{uc}$ (18)

If true, then the shear capacity of concrete alone is adequate.

Step 3: Check if $V^* > 0.5\varphi V_{uc} < V_{u.max}$ (19)

Where, $V_{u.max}$ shall be calculated in accordance with clause 8.2.3.3, AS 5100.5 (Standards Australia, 2017A) to determine if shear strength is limited by web crushing.

[10] The simplified method can be used provided the yield strength of longitudinal reinforcement is less than 500 MPa, the design concrete strength does not exceed 65 MPa and the size of the maximum aggregate particle is not less than 10 mm.

$$V_{u.max} = 0.55f'_c b_v d_v \left(\frac{cot(\theta_v)+cot(\alpha_v)}{1+cot^2(\theta_v)}\right) + P_v \tag{20}$$

Where:

α_v = angle of inclination between the inclined shear reinforcement and the longitudinal tensile reinforcement ($\cot(\alpha_v) = 0$ for perpendicular shear reinforcement.

$\theta_v = 36^{\circ}$ using the simplified method in clause 8.2.4.6, AS 5100.5 (Standards Australia, 2017A).

p_v = vertical component of prestressing force.

Step 4: If true, the minimum amount of shear reinforcement shall be provided in accordance with clause 8.2.1.7, AS 5100.5 (Standards Australia, 2017A).

$$s = \frac{A_{sv.min} \cdot f_{sy.f}}{0.08 \cdot \sqrt{f'_c} \cdot b_v} \tag{21}$$

Where:

$A_{sv.min}$ = cross-sectional area of minimum shear reinforcement (mm^2).

$f_{sy.f}$ = characteristic yield strength of reinforcement used as fitments (MPa).

f'_c = characteristic compressive strength of concrete at 28 days (MPa).

b_v = effective width of a web for shear (mm) (clause 8.2.6, AS 5100.5 (Standards Australia, 2017A)).

s = required shear reinforcement spacing (mm).

If false, and $V^* > \varphi V_{u.max}$ beam dimensions should be increased to eliminate web crushing failure.

Step 5: In all instances calculated longitudinal shear reinforcement spacing shall be checked to ensure spacing is not greater than the minimum spacing requirements as per clause 8.3.4.4, AS 5100.5 (Standards Australia, 2017A) and provided in Equation 18.

For $D \leq 1.2$ m, min [s; $0.5D$; 300 mm] (22)

3.12 DESIGN DRAWING SET CONSIDERATIONS

The final civil and structural drawing sets need to communicate the final design of the floodway structure. It is important that the drawing set is well specified, conveys the design criteria and details any assumptions made. Generally, each section within the design guideline has an output which is required to be communicated. Considerations for the inclusion within the final design drawing set are as follows:

- Project requirements and asset classification (Section 3.2):

  Project requirements and asset classification defines the level of service that the floodway is required to meet and forms a primary input for the basis for design. The asset classification requirements need to be communicated and should include vehicles per day, design speed, road type and the number of properties serviced. Other requirements such as the serviceability design discharge and the structural design discharge value shall also be specified.

- Feature survey and design surface (Section 3.4):

The outputs of the feature survey and design surface shall also be conveyed within the drawing set. This shall include the design surface contours and identification of other structures or significant landmarks relevant to the design, such as, buildings, fences, trees, adjoining roadways, drainage, the existing channel, and any utility services which may be impacted during construction. A reference mark for the feature survey and design surface shall also be provided for construction accuracy.

- Environmental considerations (Section 3.5):

  An erosion and sediment control plan outlining the control measures required to mitigate undue environment degradation must also be included in the drawing set. The erosion and sediment control plan shall be the final approved plan which addresses conditions imposed on the construction by the assessing body. Catchment and Creeks (2021) provides standard erosion and sediment control detailing drawings specific to instream works for the formation of erosion and sediment control plans.

  Other environmental concerns relating to the construction including exclusion zones, due diligence requirements and prohibited areas also need to be communicated as part of the drawing set.

- Floodway type and geometry (Section 3.6 and 3.7):

  The typical detail of the standard floodway type adopted, along with any site-specific requirements to ensure suitability of application at the individual project site shall be clearly articulated within the drawing set.

  The floodway crossfall and approach gradients shall also be specified via a detailed set of long sections and cross sections. Within these sections the required cut and fill volumes between the existing and proposed design surface shall also be clearly identified along with any existing pavement interface detailing, if relevant.

- Signage (Section 3.6):

  Sign types, spacing and locations shall be provided on a plan view drawing.

- Hydraulic design (Section 3.8):

  The backwater level shall be overlayed onto a plan view drawing to indicate the extent of potential submergence within the adjoining floodplain area. The extent of turbulence within the tailwater adjoining the floodway structure will form a design input for protection and will be illustrated by the adjoining length of downstream protection provided. The parameters defining the design flood event for the structural design and the design rainfall event for which traffic access is precluded shall also be clearly articulated. This includes the design rainfall event ARI and corresponding flow depths and velocities.

- Protection design (Section 3.9):

  The protection design for the channel in the vicinity of the floodway shall be clearly specified on the drawing set. Annotations shall also exist which clearly detail the protection type, the nominal rock size, depth, and fixing requirements.

- Structural design and reinforcing detail (Section 3.10 and 3.11):

A structural drawing set conveying the structural design shall be provided. This set shall include reinforcement detailing and structural design considerations, such as, the design bending moment, shear force values and general structural commentary.

Reinforcement detailing requires the preparation of drawings that show the reinforcement requirements for the concrete floodway structure. Reinforcement detailing shall be undertaken in accordance with AS 5100.5 (Standards Australia, 2017A). The *Reinforcement Detailing Handbook* (Concrete Institute of Australia, 2010) is also an excellent resource that provides interpretation of Australian Standards and communicates the basic requirements of industry standard reinforcement detailing.

When specifying detailing it is important to consider the construction process to ensure that the design and detailing is economical, can be assembled on-site, with minimal error and can be easily handled. Due to the large area of concrete required in floodway construction, steel reinforcement mesh is the preferred reinforcement type. Reinforcement mesh, as opposed to using steel bar offers significant advantages in terms of construction speed, uniformity and satisfactorily opposes the stresses in both directions for on ground slabs. Mesh can also be easily handled and cut to shape to suit the layout of the slab on-site.

At changes in direction 'L' shaped bars shall be provided separate to the main reinforcement and must extend to the far face in all instances. These "L' shaped bars are required to lap with the main top and bottom reinforcement by the minimum development length requirement. Reinforcing mesh shall also be overlapped at the location of the change in direction by a length equal to the width of the mesh spacing.

If mesh reinforcement sheets need to be spliced, the two outermost transverse wires of one sheet must overlap the two outermost wires of the other sheet (Concrete Institute of Australia, 2010).

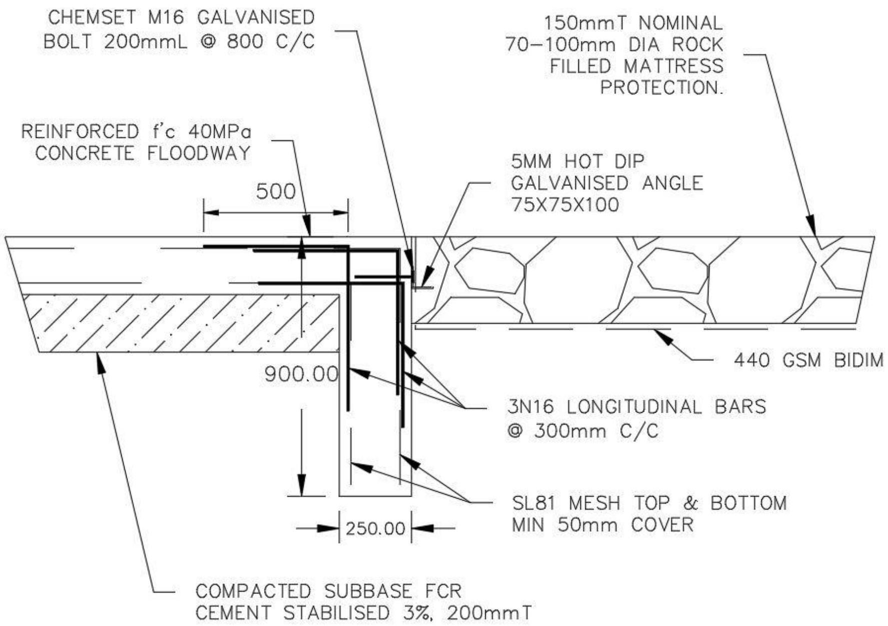


Figure 3.53. Example structural detail.

3.12.1 Typical format of the drawing set

The final drawing set shall include the following details, but not be limited to the following:

- Sections.
- Reinforcement details.
- Retaining wall details.
- Scour protection details.
- Interface details with creek and road details.
- Signage details.

The drawing set shall also detail the following information:

- Cover page:
  - List of drawings.
  - General notes including any design assumptions.
- Notes on concrete:
  - Concrete cover.
  - Reinforcement.
  - Splicing.
  - Welding.
  - Construction joints.
  - Concrete mix design.
- Hydraulics.
- Traffic loads.
- Maximum vehicular load or vehicle type.
- Scour protection information.
- Material grading.
- Geotechnical design information.
- Construction specification.
- Construction sequence.
- Curing.
- Construction tolerances.
- Proprietary fixing products (if applicable).
- Safety in Design considerations.

4. EXAMPLE 1: STAGE-DISCHARGE CURVE

Develop the stage-discharge curve including the velocity-discharge relationship for a floodway to be located at Black Duck Creek Road, Black Duck Creek, QLD 4343 (Figure 4.1).

Figure 4.1. Black Duck Creek floodway crossing (Lockyer Valley Regional Council, personal photograph, 16 June 2011).

Given:

Table 4.1. Design ARI rainfall events for Black Duck Creek.

ARI (%)	1	2	5	10	20	50	100
Discharge (m^3/s)	29.47	32.99	54.01	75.38	111.03	156.15	175.39

*Note. the discharge values were determined through a catchment analysis in reference to QUDM (IPWEAQ, 2016B) and the Design Rainfall Data System (Bureau of Meteorology, 2016).

- Hydraulic gradient 'S' = 0.0060
- Estimated Manning's roughness coefficients 'n' from Table 3. 2: Main channel consist of gravel and few boulders = 0.050
- Channel depth = 2 m
- Channel Width = 35.0 m
- Channel sides = 1 to 2 ratio

Step 1 – Determine maximum flow the natural channel can contain.

Area, A = $(33 \cdot 2) + (1)^2 = 67\ m^2$

Wetted perimeter, P = $33 + 2 \cdot \sqrt{1^2 + 2^2} = 37.47\ m$

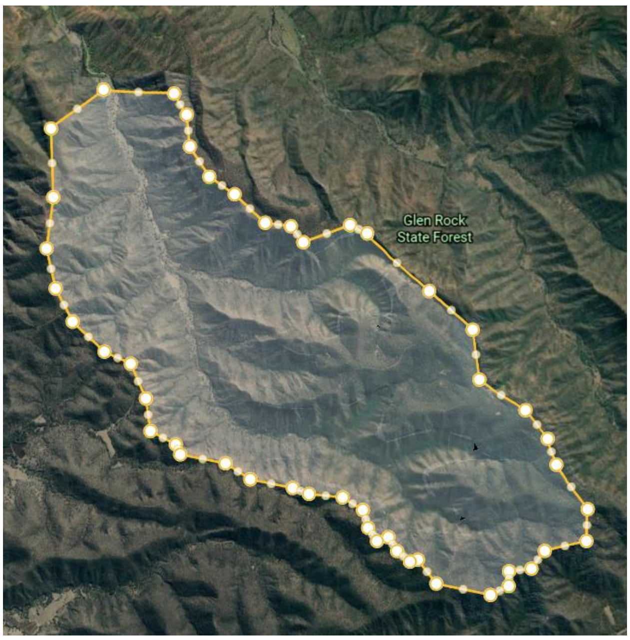


Figure 4.2. Black Duck Creek catchment area.

Hydraulic radius, R = $\frac{A}{P} = \frac{67}{37.47} = 1.79\ m$

$$Flow, Q \ = \frac{1}{n} \cdot A \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}} = \frac{1}{0.050} \cdot 67 \cdot 1.79^{\frac{2}{3}} \cdot 0.006^{\frac{1}{2}} = 152.9\ m^3$$

Therefore, the natural channel has the capacity to contain a 20-year ARI storm event and falls just short of being able to contain a 50-year ARI event prior to spilling into the floodplain area.

Next undertaking a series of iterations at 0.25 intervals for channel depth between 0 m to the maximum channel depth (Table 4.2).

Table 4.2. Iterations based on channel depth.

Depth	Area (A) (m2)	Wetted perimeter (p) (m)	Hydraulic radius (R) (m)	Velocity (V) (m/s)	(Discharge (Q) (m3/s)
0	0	0	0	0	0
0.25	9.25	39.03	0.24	0.85	7.84
0.5	18.5	39.12	0.47	1.34	24.85
0.75	27.75	39.27	0.71	1.76	48.72
1	37	39.47	0.94	2.12	78.43
1.25	46.25	39.72	1.16	2.45	113.29
1.5	55.5	40.00	1.39	2.75	152.80
1.75	64.75	40.32	1.61	3.04	196.53
2	74	40.66	1.82	3.30	244.14

Plot the stage-discharge curve with flow depth on the x-axis, discharge on primary y-axis and velocity on the secondary y-axis (Figure 4.3).

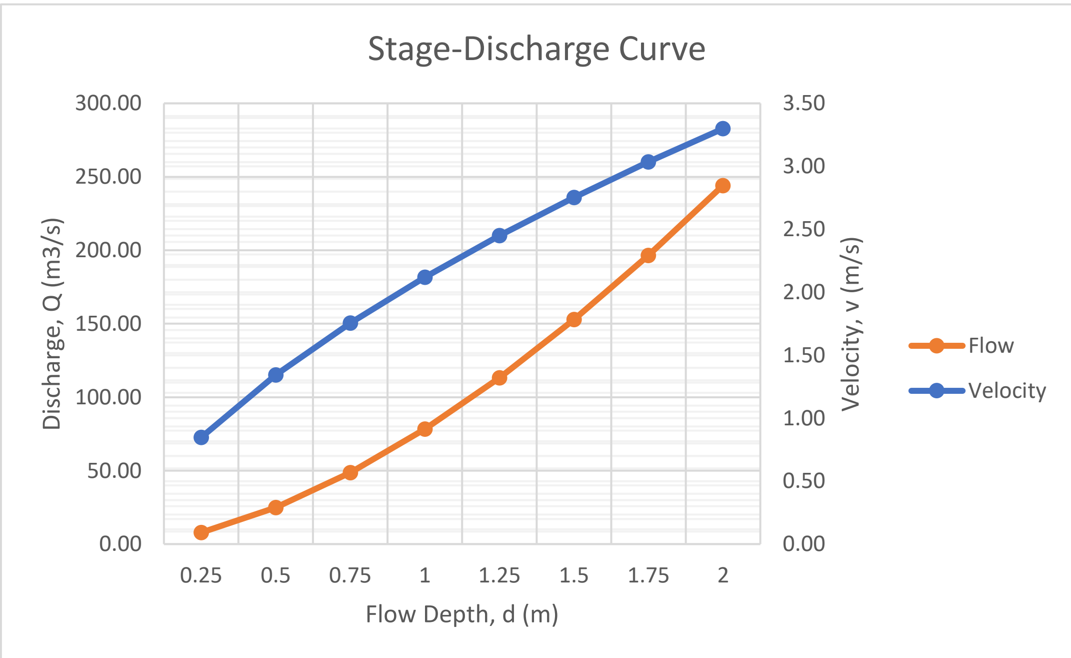


Figure 4.3. Stage-Discharge curve - Black Duck Creek.

5. EXAMPLE 2: HYDRAULIC MODELLING USING HEC-RAS

Expanding upon Example 1 *'Stage-discharge curve'*, determine the backwater depth for the 10-year ARI rainfall event discharge (75.38 m^3/s) using HEC-RAS (U.S. Army Corps of Engineers Hydrologic Engineering Center, 2019) and assuming a trapezoidal creek profile and Floodway Type 3 with a raised road height of 1 metre (Figure 3.14).

1. *Setting up a new project*

   Open HEC-RAS software and click "File" – "New Project" (Figure 5.1).

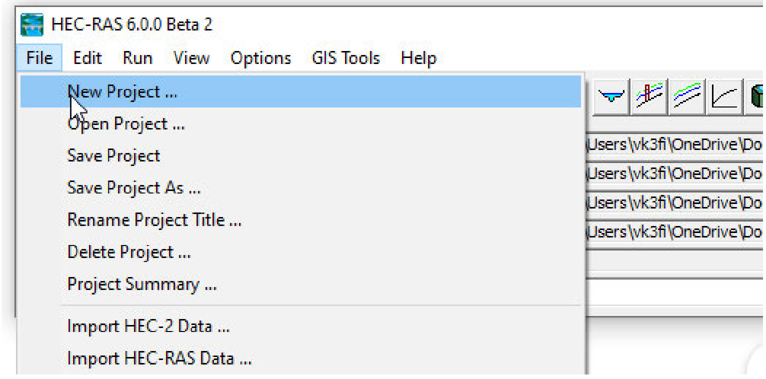


Figure 5.1. Commencing a new HEC-RAS project.

Enter the new project title, in this example the title "Black Duck Creek Floodway" was entered. Then select the desired project folder location and click "OK" (Figure 5.2).

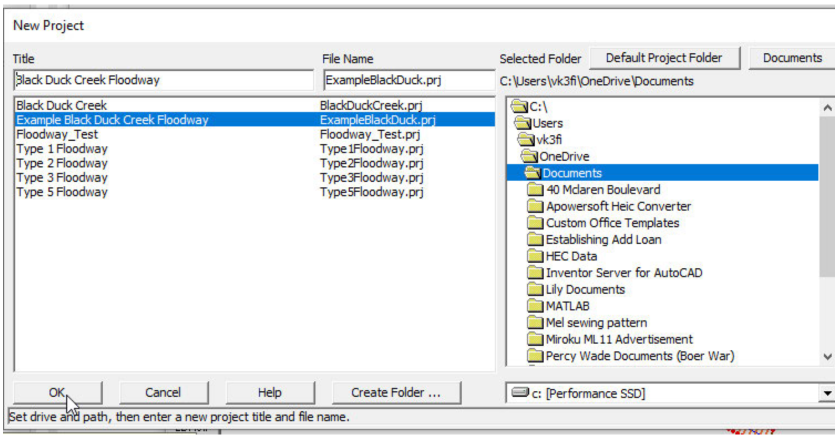


Figure 5.2. Entering project name and assigning folder location.

2. *Enter geometric data*

   Click "View/Edit geometric data". This window enables the geometric properties of the channel and the floodway to be entered and/or viewed (Figure 5.3).

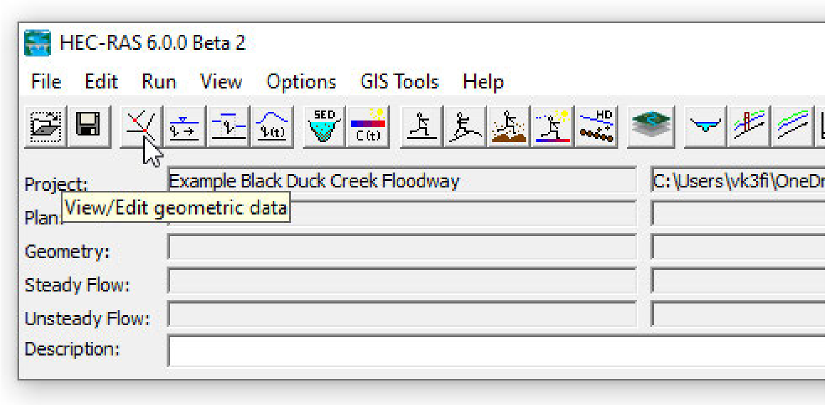


Figure 5.3. View and edit geometric data.

Click "Add new River Reach" (Figure 5.4). This enables the user to draw the river and define the reaches for which similar hydrological conditions exist within for analysis. This also enables backwater effects to be analysed for floodway crossings near creek branches.

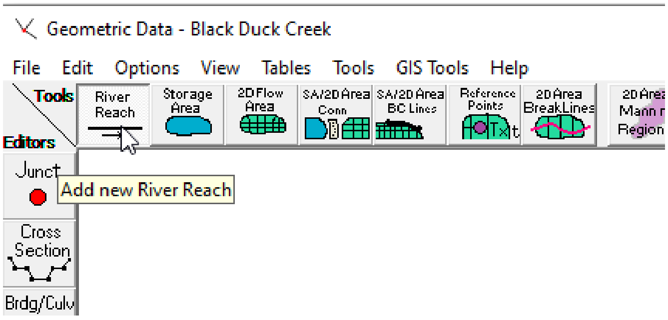


Figure 5.4. Adding a new river or reach.

Draw the reach with a diagonal line and assign a river name, "Black Duck Creek" and reach name "Ch 2200 to 2280" (Figure 5.5).

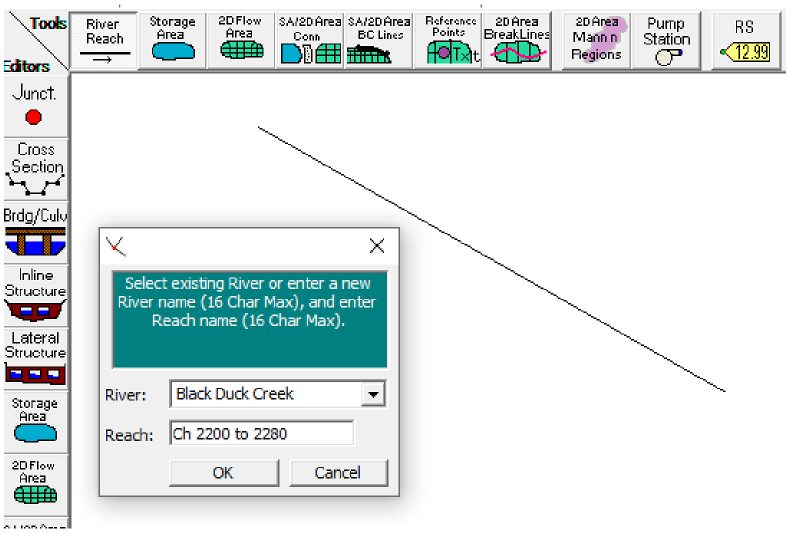


Figure 5.5. Drawing the reach and assigning names.

Next the coordinates and properties defining the channel and floodway long and cross section geometry shall be entered. A sketch of the channel long section with the cross-sections that shall to be defined is illustrated in Figure 5.6.

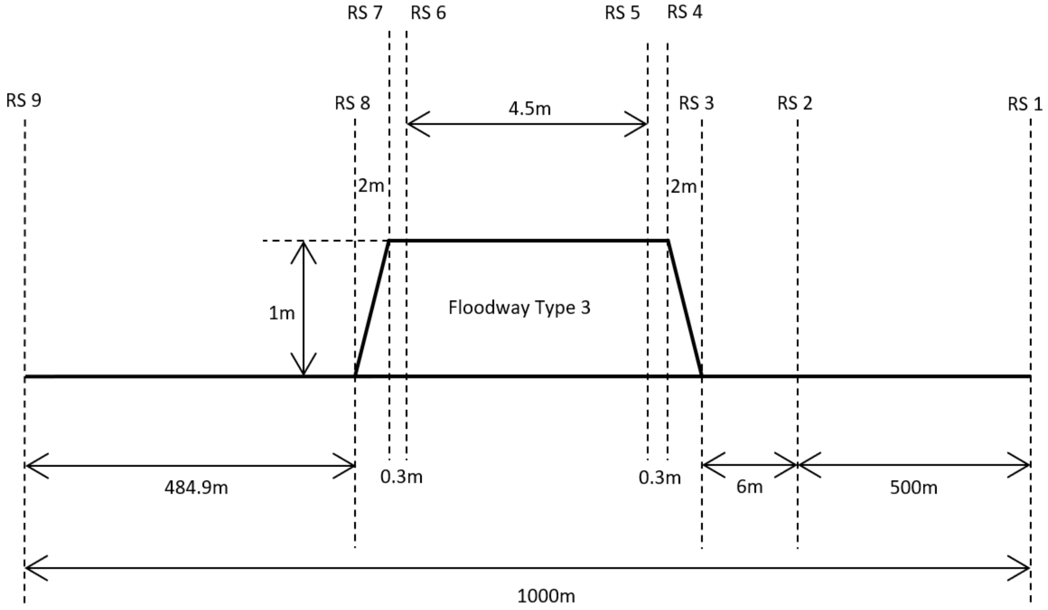


Figure 5.6. Channel and floodway long section to be modelled.

To define the long and cross section geometries, click "Edit and/or create cross section" (Figure 5.7).

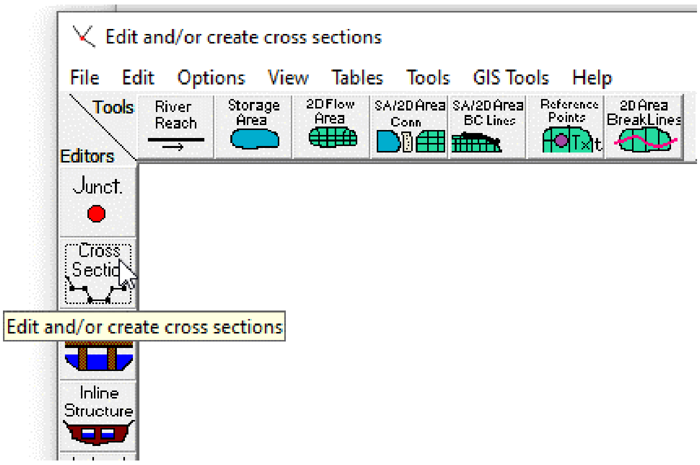


Figure 5.7. Editing and creating cross sections.

To assign a new cross section, click "Options" – "Add a New Cross Section" (Figure 5.8).

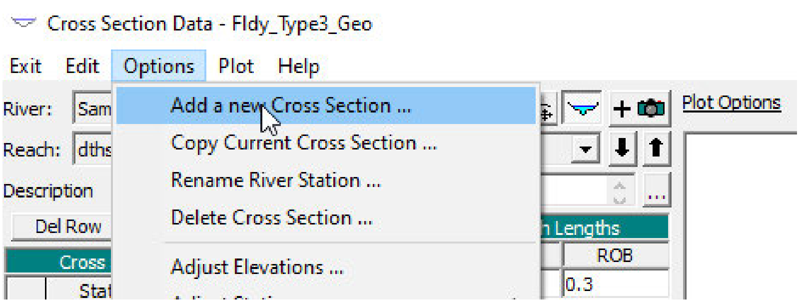


Figure 5.8. Adding new cross section to the reach.

Cross-section co-ordinates (Figure 5.9) define the boundary geometry for the analysis in terms of the ground surface profile of the channel. Table 5.1 provides the boundary co-ordinates for the Type 3 floodway situated within a trapezoidal channel and with a bed slope of 0.006, reach length of 1,000 m and raised floodway level of 1 m.

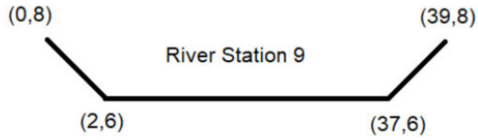


Figure 5.9. Sample cross section defining ground surface profile.

Table 5.1. Cross-section co-ordinates.

River station	Station	Elevation	DS Reach Length	River station	Station	Elevation	DS Reach Length	River station	Station	Elevation	DS Reach Length
9	0	8	484.9	**8**	0	5.0906	2	**7**	0	5.0786	0.3
" "	2	6		" "	3	3.0906		" "	3	4.0786	
" "	37	6		" "	32	3.0906		" "	32	4.0786	
" "	39	8		" "	35	5.0906		" "	35	5.0786	
6	0	5.0768	4.5	**5**	0	5.0498	0.3	**4**	0	5.048	2
" "	3	4.0768		" "	3	4.0498		" "	3	4.048	
" "	32	4.0768		" "	32	4.0498		" "	32	4.048	
" "	35	5.0768		" "	35	5.0498		" "	35	5.048	
3	0	5.036	6	**2**	0	5	500	**1**	0	2	0
" "	3	3.036		" "	3	3		" "	3	0	
" "	32	3.036		" "	32	3		" "	32	0	
" "	35	5.036		" "	35	5		" "	35	2	

Channel properties, including, Manning's coefficient, contraction and expansion coefficients and the length to the nearest downstream river station are then assigned for each new cross section added. Table 5.2 provides the channel properties for the Type 3 floodway situated within a trapezoidal channel.

Table 5.2. Cross-section channel properties.

River station	9	8	7	6	5	4	3	2	1
Downstream Reach Lengths									
LOB	484.9	2	0.3	4.5	0.3	2	6	500	0
Channel	484.9	2	0.3	4.5	0.3	2	6	500	0
ROB	484.9	2	0.3	4.5	0.3	2	6	500	0
Manning's n Values									
LOB	0.035	0.035	0.035	0.012	0.035	0.035	0.035	0.035	0.035
Channel	0.035	0.035	0.035	0.012	0.035	0.035	0.035	0.035	0.035
ROB	0.035	0.035	0.035	0.012	0.035	0.035	0.035	0.035	0.035
Main Channel Bank Stations									
Left Bank	0	0	0	0	0	0	0	0	0
Right Bank	35	35	35	35	35	35	35	35	35
Cont\Exp Coefficient									
Contraction	0	0	0.3	0	0	0	0	0	0
Expansion	0	0	0.5	0	0	0	0	0	0

An example of the cross-sectional geometry data entered for River Station 9 (Figure 5.10).

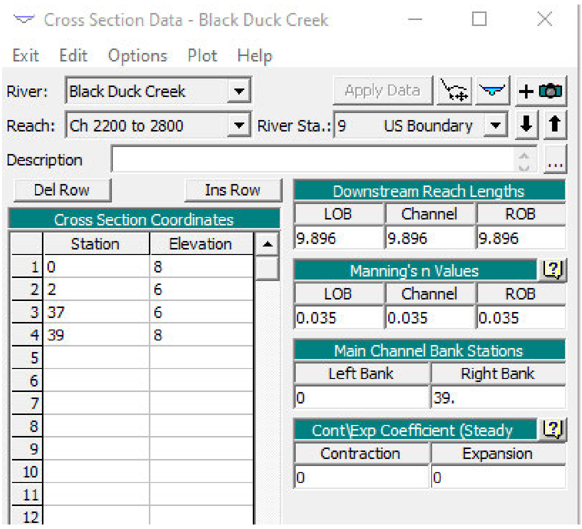


Figure 5.10. An example of entered cross-sectional data.

Next, intermediate cross sections within the reach between the defined River Stations need to be interpolated based on the cross-sectional data entered. This is done by clicking “Tools” – “XS Interpolation” – “Between 2 XS’s…” (Figure 5.11).

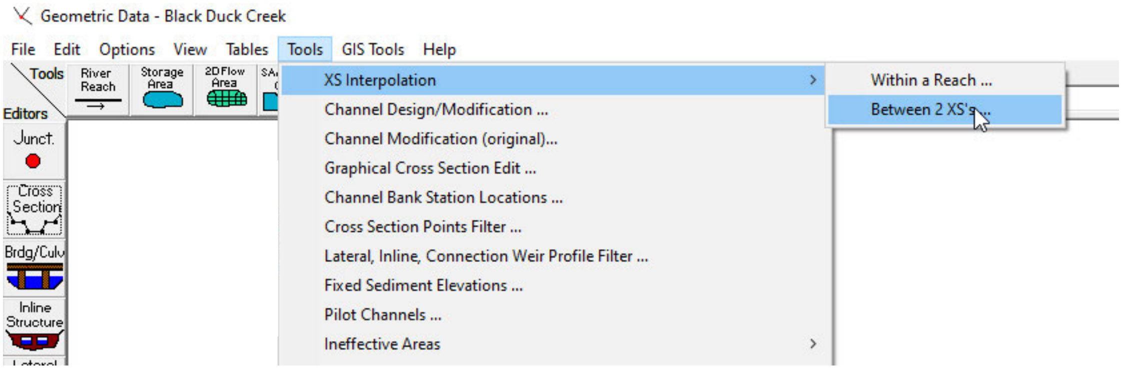


Figure 5.11. Interpolating river cross sections between user defined River Stations.

Within the field “Maximum Distance (m)” enter a value which will ensure a smooth transition is created among geometrical changes at River Station cross-sections (Figure 5.12). Once completed click “Interpolate new XS’s” (Figure 5.12). Repeat this step for all upper and lower River Stations by using the arrows at the top right corner.

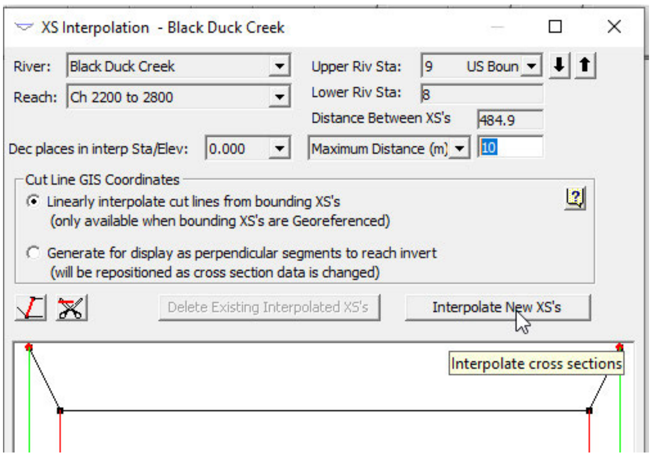


Figure 5.12. Entering cross section interpolation criteria.

Once complete click "File" – "Save Geometry Data as" (Figure 5.13) and enter a suitable title and click "OK". The "Geometric Data" window can then be exited.

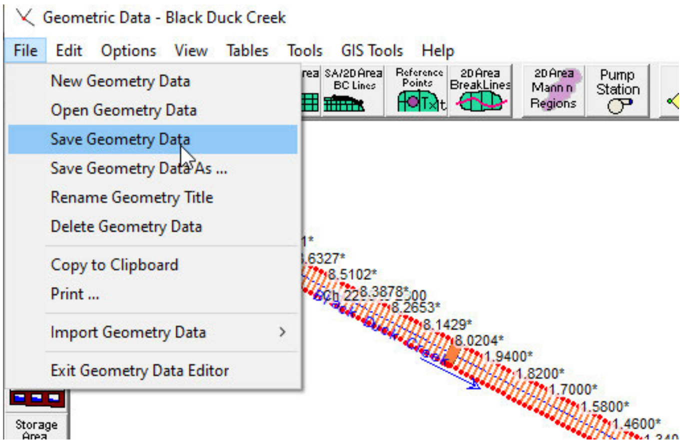


Figure 5.13. Saving geometry changes made.

3. *Entering the steady flow data*

The steady flow data provides the boundary conditions and user inputted flow profiles to enable steady water surface profile calculations to be performed.

Initially click "View/Edit steady flow data" (Figure 5.14).

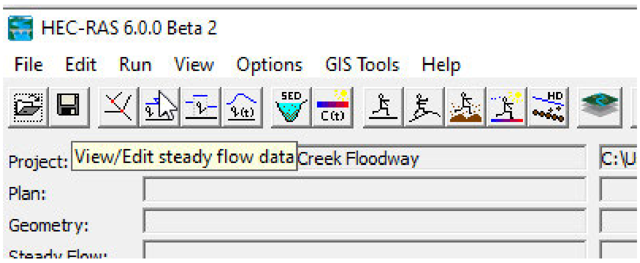


Figure 5.14. View and edit steady flow data.

Next edit “Enter/Edit Number of Profiles” to match the number of flows to be analysed (Figure 5.15). At least one profile for the flow corresponding to the 10-year ARI event should be entered, however, it is recommended that all discharges corresponding to the calculated ARI events (Table 4.1) should be entered. This provides a total of seven different flow profiles for analysis.

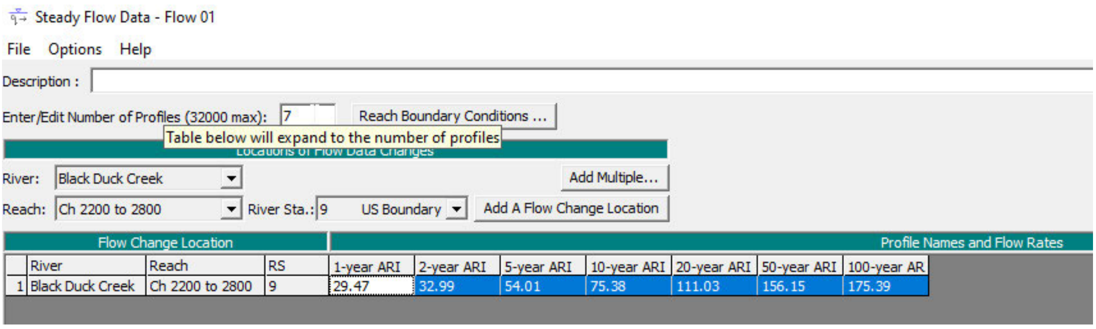


Figure 5.15. Entering and editing flow profiles.

Next the reach boundary conditions need to be assigned. To do this, the user defines the known water surface properties at the model extents (boundaries). Assuming both subcritical and supercritical flow regime are present, boundary conditions need to be assigned at both the upstream and downstream river extents. Open channel conditions and the subsequent use of Manning’s formula is applicable only if the upstream and downstream extents are modelled far enough away from the floodway structure so that any influence from the associated hydraulic control on the channel can be negated. Therefore, these boundary conditions can be assigned based on the stage-discharge curve (rating curve) calculated earlier in Example 1. This assumption is only true for uniform flow conditions within regular shaped channels.

To ascertain that no effect on flow has resulted from the floodway and boundary conditions selected a convergence study should be undertaken. This step is undertaken after the model has been solved and involves analysing the water surface profile at the upstream and downstream extents to ensure open channel flow conditions have resumed. If this is not the case the upstream and downstream model extents shall be iteratively increased until this condition is satisfied.

First, click “Reach Boundary Conditions …” (Figure 5.16).

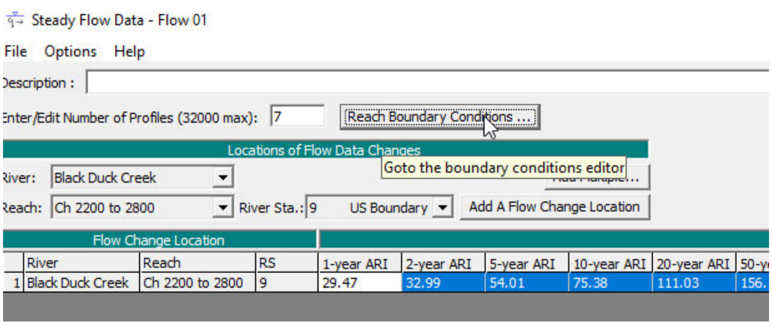


Figure 5.16. Assigning boundary conditions to the reach.

Next click "Downstream" – "Rating Curve" (Figure 5.17).

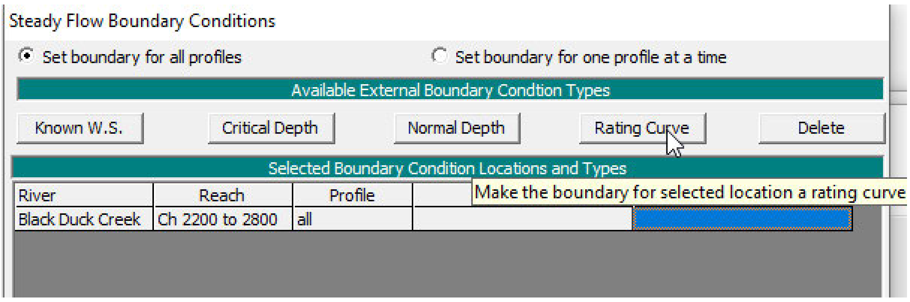


Figure 5.17. Assign a rating curve as boundary conditions.

Insert the data from the stage-discharge calculations conducted prior in Example 1 (Figure 5.18).

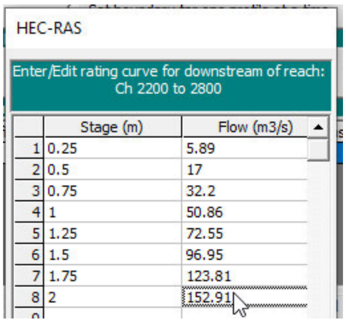


Figure 5.18. Inserting stage-discharge curve calculations.

Repeat for "Upstream", utilising the same rating curve for the open channel (Figure 5.19).

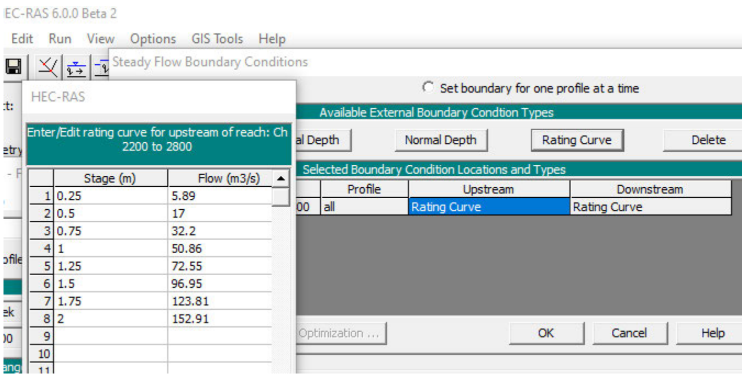


Figure 5.19. Assigning a rating curve as the upstream boundary criteria.

Click "OK" – "OK" – "Apply Data, and then exit out of the "Steady Flow Data" window.

4. *Performing a steady flow simulation*

Within this step the software will then undertake the required computations based on the channel geometry, boundary conditions and the user made flow profiles.

Click "Perform a steady flow simulation" (Figure 5.20).

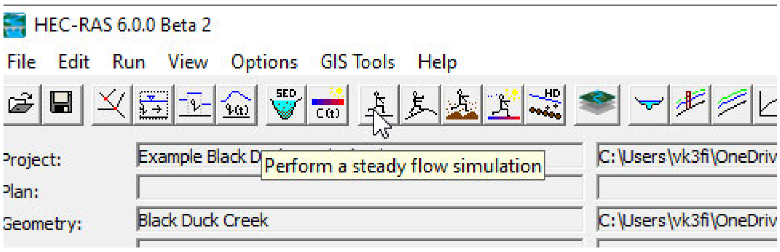


Figure 5.20. Opening the steady flow simulation window.

Then select the flow regime as "Mixed" and click "Compute" (Figure 5.21). Once finished, check the "Computation Messages" for errors and click "Close" to exit the computation window.

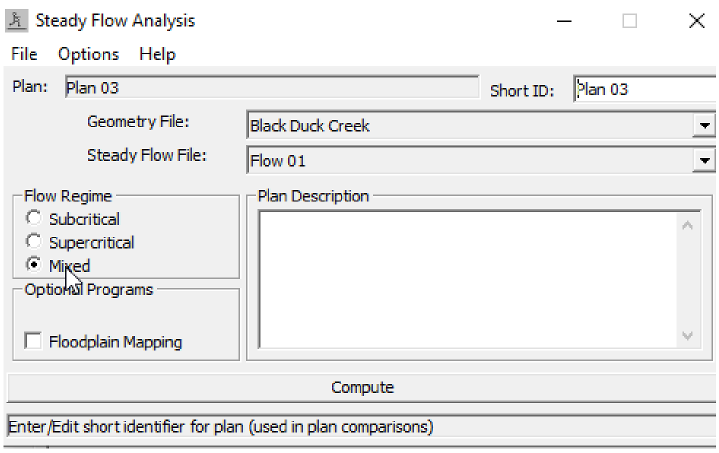


Figure 5.21. Selecting flow regime and commencing steady flow simulation.

5. Viewing and interpreting results:

   The long section should now be viewed to ensure boundary extents are sufficient and that the flows at the floodway locality are not being influenced by the assigned boundary conditions (Figure 5.22).

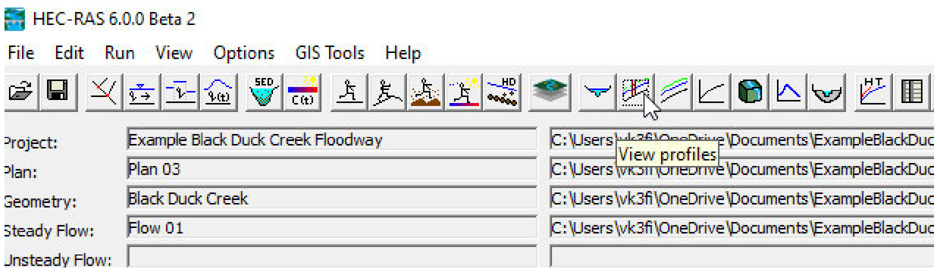


Figure 5.22. Viewing the long section profile of the computed results.

From the reach long section, it is visually evident that the select 1,000 m length is sufficient as at both the upstream and downstream extents open channel flow has resumed, therefore, the use of Manning's formula is applicable (Figure 5.23). Note flow direction is right to left and the long section is on an angle corresponding to the assigned bed slope of 0.006.

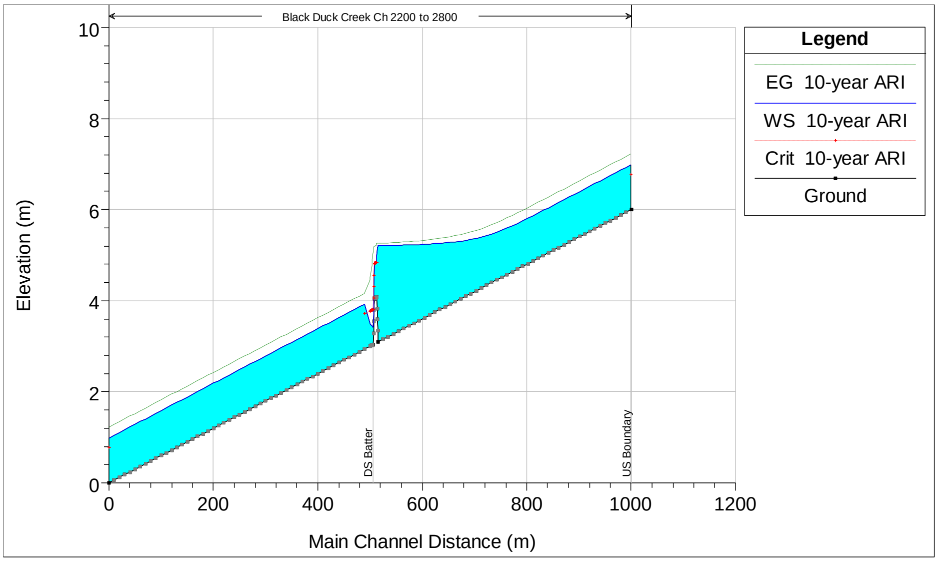


Figure 5.23. Long section profile.

The “X-Y-Z Perspective Plots (Classic)” shall also be viewed. This is done by clicking “View” – “X-Y-Z Perspective Plots (Classic)” (Figure 5.24).

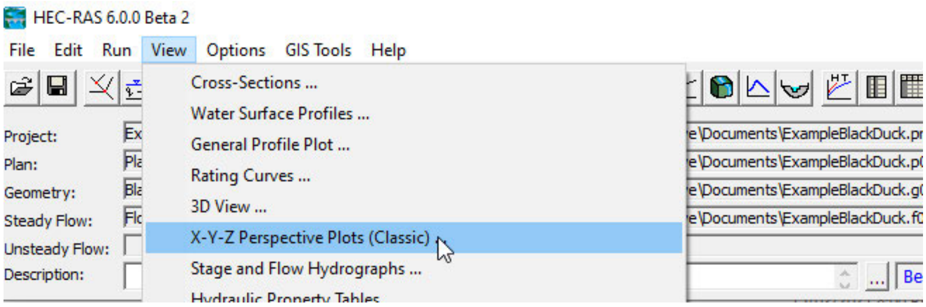


Figure 5.24. Viewing the X-Y-Z- Perspective Plots (Classic).

Visually analyse the X-Y-Z Perspective Plot to ensure cross-section distribution and geometry has been correctly assigned. The raised backwater level and associated tailwater effects can also be visualised using this plot type (Figure 5.25).

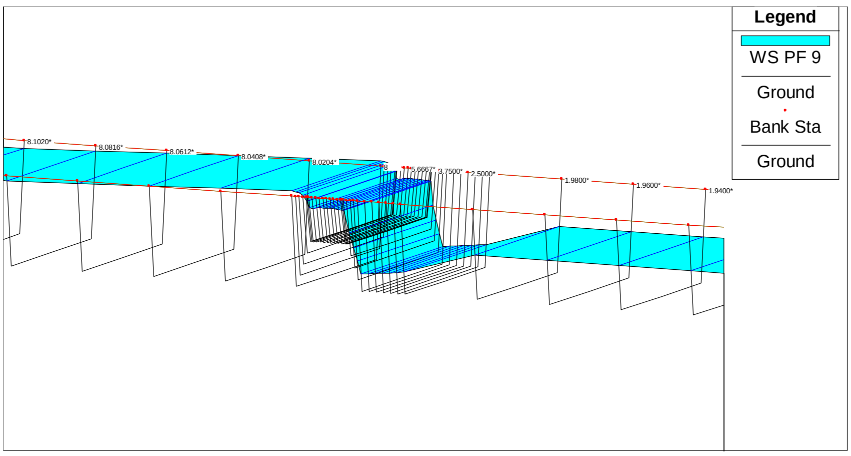


Figure 5.25. X-Y-Z perspective plot.

6. Determining the associated backwater level

The associated backwater level for the 10-year ARI storm event of 75.38 m^3/s is determined by interrogating the long section by hovering the users mouse over the location corresponding to the peak backwater surface elevation (Figure 5.26). This point occurs at chainage 513.55 m and at a water surface elevation of 5.19 m.

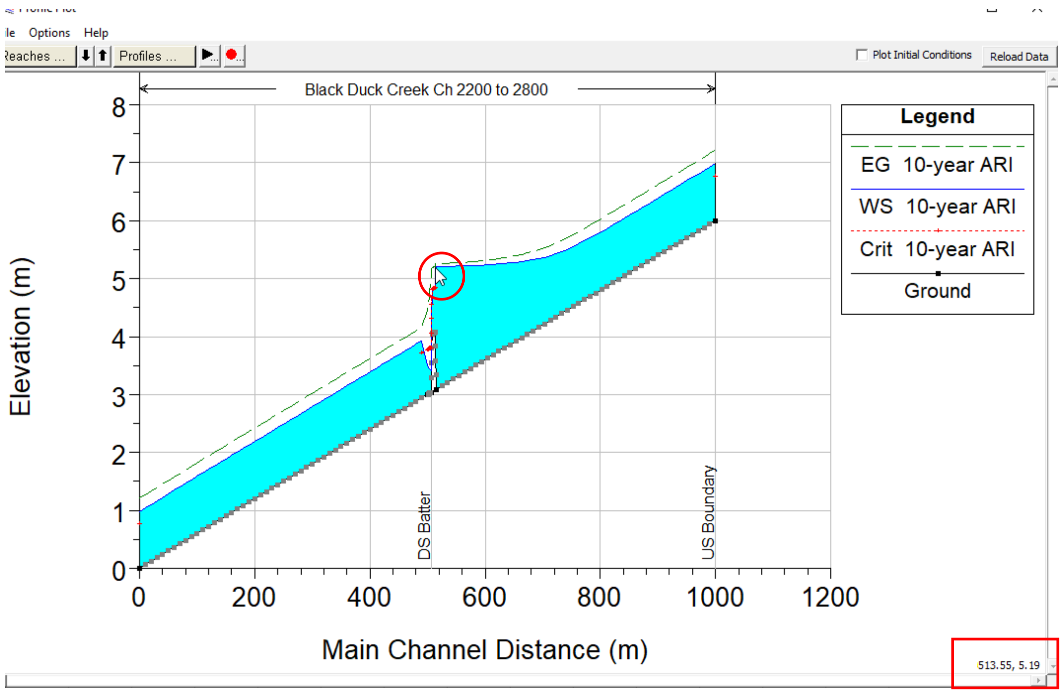


Figure 5.26. Interrogating the long section profile.

The elevation gain due to the bed-slope then needs to be subtracted from the backwater elevation as per the calculation below:

$$Backwater\ level = water\ surface\ elevation - (Chainage \times Bed\ slope\ (S))$$
$$Backwater\ level = 5.19\ m - (513.55 \times 0.006)$$
$$Backwater\ level = 2.109\ m$$

6. EXAMPLE 3: STRUCTURAL DESIGN

Expanding on Example 1 and 2, determine the cross-sectional area of steel and concrete that is required to satisfy strength, serviceability, and durability criteria for the 900 mm perimeter cut-off wall for Floodway Type 3. Given the Type 3 Floodway is situated in a Clay soil (Soil Type 3) and exposed to a 2 m flow depth based on historical flood information (flow equals 244.14 m^3/s).

Step 1 – Determine Design Parameters

- Soil Type = Clay Soil (Soil Type 3).
- Design flow velocity = 3.3 m/s (from stage-discharge curve, Example 1).
- Design flow depth = 2.0 m (from stage-discharge curve, Example 1).
- Initially assume the use of SL62 steel reinforcing mesh.

Referencing the structural design charts in Section 3.10, the maximum design bending moment and shear force which occurs at the critical section is determined (Figure 6.1). Note: If an intermediate value is required, linear interpolation of the graphs can be utilised.

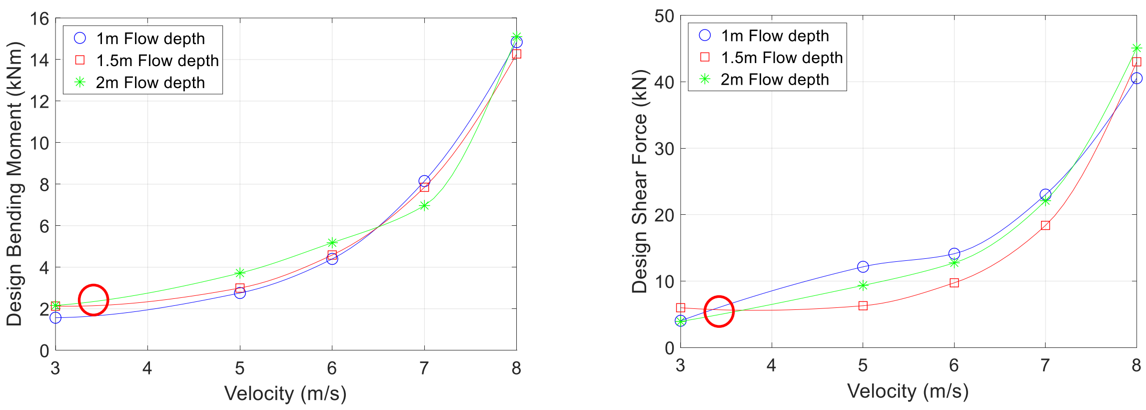


Figure 6.1. Extracting design bending moment and shear force from charts.

- Design bending moment = 2.5 kN.m
- Design shear force = 7 kN

Step 2 – Design of cut-off wall for Durability

Utilising Section 4 of AS 5100.5 (Standards Australia, 2017A) durability requirements are determined.

- Exposure classification = Class B2
- Minimum compressive strength = minimum compressive strength of 40 MPa
- Minimum cover requirements = 50 mm, assuming standard formwork and compaction

Step 3 - *Design of cut-off wall for Strength and Serviceability in Bending*

Utilising Section 8.1 of AS 5100.5 (Standards Australia, 2017A) determine design for strength and serviceability in bending.

1. Check strength of cross-section in bending utilising the equivalent rectangular stress block presented in Figure 3.51 and equilibrium considerations to satisfy clause 8.1.2 of AS 5100.5 (Standards Australia, 2017A).

   Initially assuming SL62 mesh:

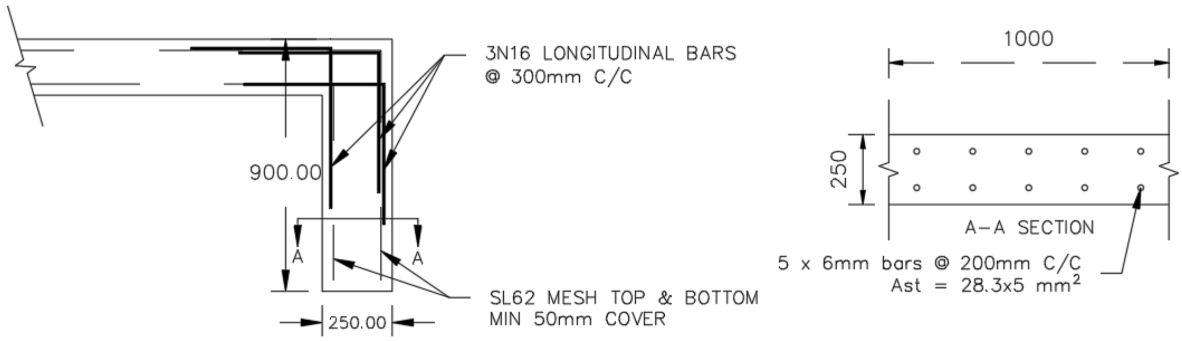


Figure 6.2. Downstream cut-off wall section for design consideration.

Calculate compressive force in concrete, F_c i.e. volume of the stress block:

$F_c = (\alpha_2 f'_c)\ (\gamma d_n)\ b$

$F_c = (0.85)(40\ MPa)(0.826)(d_n)(1000\ mm)$

$F_c = 28084(d_n)\ N$

Calculate tensile forces in steel (assuming steel yields), F_t:

$F_t = A_{st}f_{sy}$

$F_t = (141.5\ mm^2)(500\ MPa)$

$F_t = 70{,}750\ N$

Equate F_t and F_c to determine the neutral axis depth:

$F_t = F_c$

$28084\ d_n = 70{,}750$

$d_n = 2.52\ mm$

With d_n evaluated the lever arm can be calculated:

$z = D - c - B/2 - \gamma \cdot d_n/2$

$z = 250\ mm – 50\ mm – 6\ mm/2 – ((0.826)(2.52\ mm)/2)$

$z = 196\ mm$

Calculate moment capacity, M_u:

$M_u = F_t z = F_c z$

$M_u = (70{,}750\ N)(196\ N.mm) \times 10^{-3}$

$M_u = 13.9\ kN.m$

Check if assumed bending reinforcement is satisfactory:

$\varphi M_u \geq M^*$

$(0.8)(13.9) = 11.1\ kN.m \geq 2.5\ kN.m$

Therefore satisfactory.

2. Check minimum ultimate strength in bending requirements at the critical section in accordance with clause 8.1.6.1.

$A_{st} \geq [\alpha_b (D/d)^2 f'_{ct.f}/f_{sy}] b_w d$

Where $\alpha_b = 0.20$ for rectangular sections.

$A_{st} \geq [(0.20)(250\ mm / 196\ mm)^2 (2.2\ MPa / 500\ MPa)](1000\ mm)(196\ mm)$

$A_{st} \geq 280.6\ mm^2$

$141.5\ mm^2 \leq 280.6\ mm^2$

Therefore, not satisfactory since the tensile reinforcement of the cross-sectional area (A_{st}) is less than the A_{st} required to satisfy ultimate strength in bending requirements.

Try SL92 mesh:

$A_{st} = \pi r^2$ x number of bars

$A_{st} = \pi(4.5)^2 \times 5$

$A_{st} = 318\ mm^2$

$318\ mm^2 \geq 280.6\ mm^2$

Therefore satisfactory.

3. Check minimum area of steel reinforcement required to satisfy crack control against shrinkage and temperature effects:

$A_{st} \geq 0.00175bD$

$A_{st} \geq 0.00175(1000)(250)$

$A_{st} \geq 437.5\ mm^2$

$318\ mm^2 \leq 437.5\ mm^2$

Therefore, not satisfactory since the tensile reinforcement of the cross-sectional area (A_{st}) is less than the A_{st} required to satisfy crack control against shrinkage and temperature effects.

Try SL81 mesh:

$A_{st} = \pi r^2$ x number of bars

$A_{st} = \pi(4)^2 \times 10$

$A_{st} = 503\ mm^2$

$503\ mm^2 \geq 437.5\ mm^2$

Therefore satisfactory.

Outcome: SL81 mesh reinforcement to both the inner and outer faces of the floodway to satisfy bending moment requirements.

Step 4- *Design for Strength and Serviceability in Shear*

Utilising Section 8.2 of AS 5100.5 (Standards Australia, 2017A) determine design for strength and serviceability in shear.

1. Calculate the contribution of concrete to shear strength, V_{uc}:

$V_{uc} = k_v b_v d_v (f'_c)^{½}$

Where,

b_v = effective width of a web for shear = $900\ mm$

d_v = $max(0.72 \cdot D, 0.9 \cdot d) = max\big(0.72 \cdot 250, 0.9 \cdot (250 - 50 - 3)\big) = 180\ mm$

$k_v = assume\ \frac{A_{sv}}{s} \geq \frac{A_{sv.min}}{s} :\ k_v = \ 0.15$

$$V_{uc} \ = \ k_v b_v d_v (f_c')^{\frac{1}{2}}$$

$$V_{uc} \ = \ 0.15 \cdot 900 \cdot 180 \cdot 40^{\frac{1}{2}}$$

$$V_{uc} \ = \ 170.8\ kN$$

$$ØV_{uc} = 119\ kN$$

$$ØV_{uc} \geq V^*$$

$$119 \ \geq 7\ kN$$

Thus, the shear capacity of concrete alone is adequate to withstand the design shear force (V^*).

2. Check if minimum shear reinforcement is required. Since the member thickness is less than 300 mm, transverse shear reinforcement is not required as per clause 8.2.1.6 (Standards Australia, 2017A).

7. EXAMPLE 4: DESIGN OF A FLOODWAY

The raised floodway located at chainage 255.00 Williams Road, Table Top, NSW 2640 needs to be re-designed as a floodway which is level with the channel bed. The floodway needs to be able to structurally withstand a 1.5 m flow depth based on historical flood information for Eight Mile Creek. This flow depth is approximately twice the depth of the 100-year ARI rainfall event. Closure time is not a significant consideration as secondary access to residents exists.

__Location:__ Eight Mile Creek, Ch. 255.00 Williams Road, Table Top, NSW, 2640 (Figure 7.1).

__Project:__ Replacement of existing floodway structure to a structure level with channel bed

Step 1: Road asset classification

Figure 7.1. Eight Mile Creek floodway crossing.

Williams Road has the following design criteria:

Table 7.1. Williams Road asset classification.

Asset Classification Characteristic	Williams Road
Vehicles per day	~20
Design speed (km/hr)	60
Road type	unsealed
Number of properties servicing	4

Based on Table 11, the road asset is classified as an Access Road and a two or single lane floodway is suitable. Since the current road width is of single lane nature the minimum recommended width will be adopted as follows:

- 4.5 m (3.5 m lane and 2 x 0.5 m shoulders)

Step 2: Field survey

Channel Parameters:

- Hydraulic gradient 'S' = $\frac{189.43\ \text{(US Level)} - 189.10\ \text{(DS Level)}}{36.07}$ = 0.00915
- Estimated Manning's roughness coefficients 'n' from Table 2:
  - Main channel = No bush with short grass = 0.035
- Soil type = Clay Soil
- Creek Depth = 1.79 m
- Creek Width = 8.7 m

Preliminary sketch and survey:

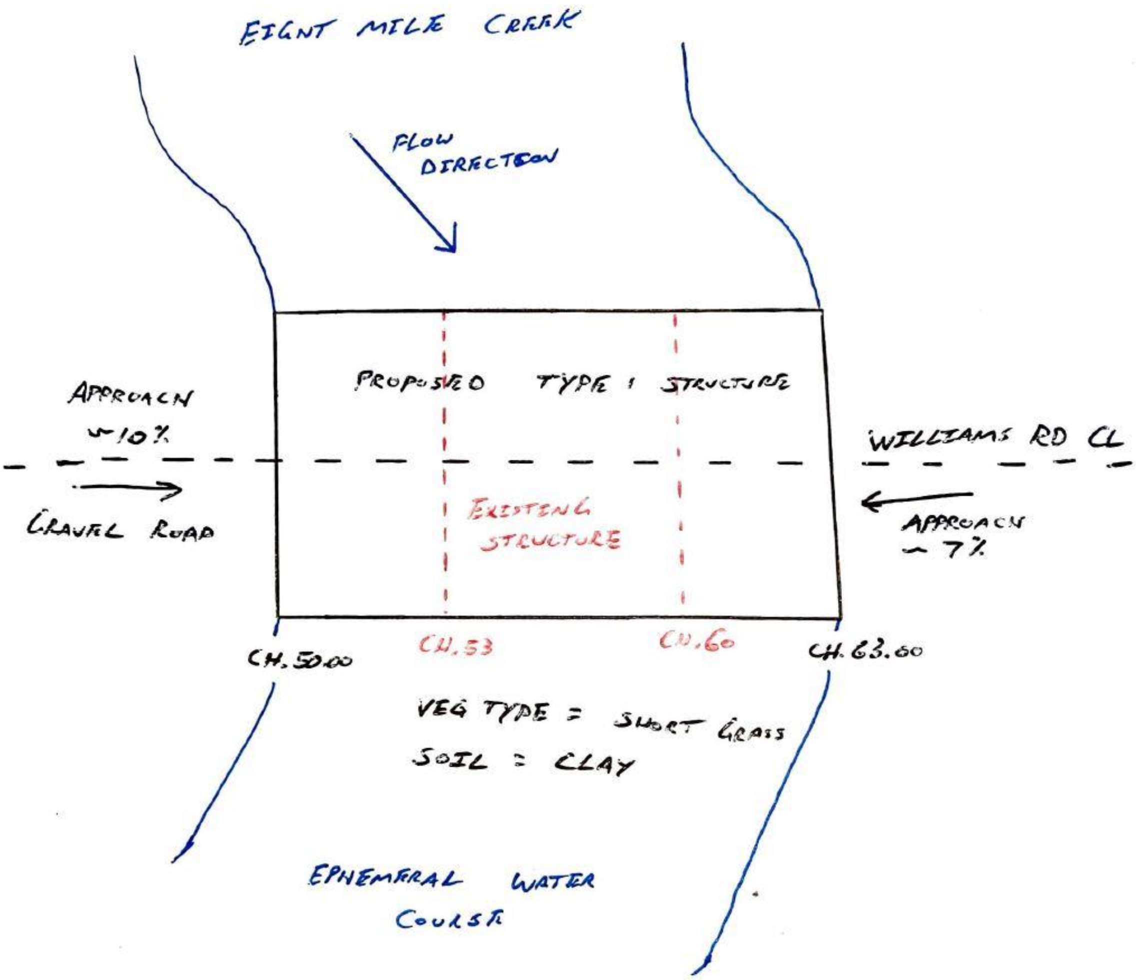


Figure 7.2. Preliminary site sketch Williams Road.

- Creek Cross-section:

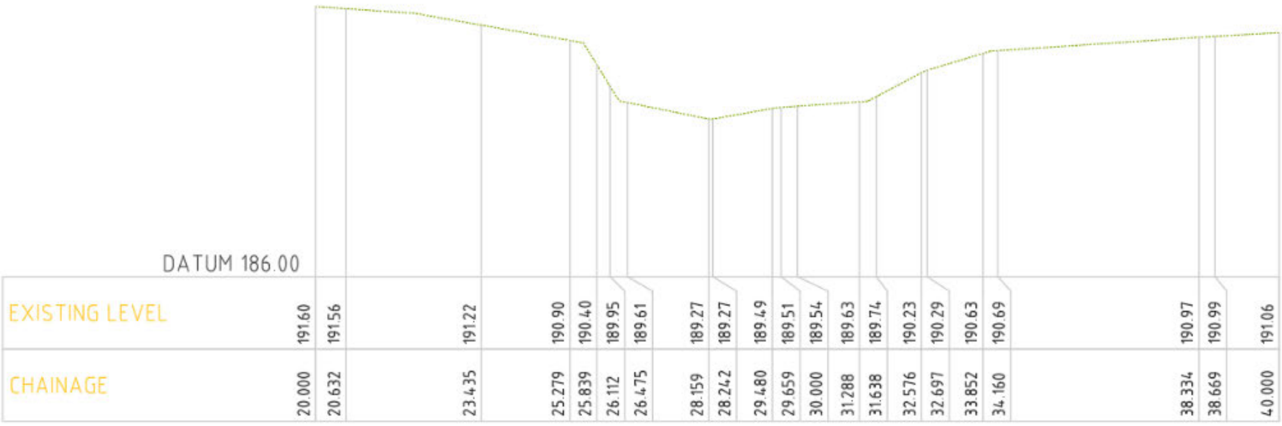


Figure 7.3. Eight Mile Creek proposed crossing location creek cross section.

- Creek Long section:

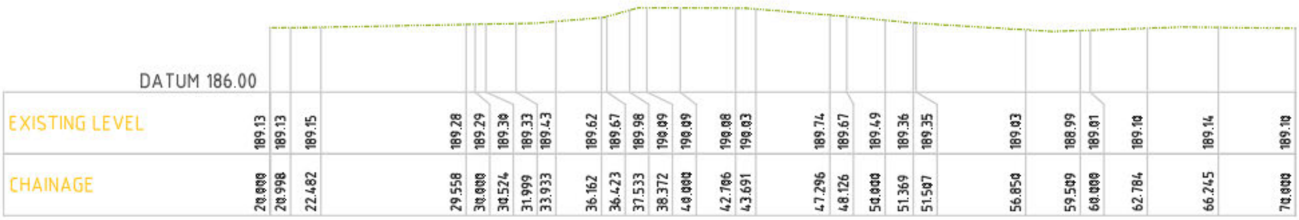


Figure 7.4. Eight Mile Creek proposed crossing location creek long section.

- Road Long section:

Figure 7.5. Eight Mile Creek proposed crossing location road long section.

Step 3: Vertical and horizontal alignment

The road has a steady vertical decline into the floodway structure which is situated raised to the creek bed and is deemed to provide motorists with appropriate sight distance and advance warning of the floodway crossing. Furthermore, the new structure is to be designed with zero super elevation to ensure a uniform water depth across the floodway.

Step 4: Determine design road level

The floodway is to be constructed level with the channel at a relative level (RL) of 189.3 m.

Step 5: Select floodway type

Since the floodway services a very low number of vehicles per day and secondary access to dwellings exists, a category A, level floodway is acceptable in the installation. Further and in reference with Table 8.2, the channel slope is relatively constant, and the catchment discharge values are low and therefore standard engineering Floodway Type 1 is acceptable and is selected for design.

Step 6: Hydraulic Design

Develop the stage-discharge for Williams Creek at the point of crossing.

Catchment characteristics:

- Latitude: -36.01620
- Longitude: 147.01124
- Catchment slope: (260m - 200m) / 8,140 m = 0.74%
- Stream length: 8,140 m
- Country type = Rolling Country
- Catchment Area = 3806 ha
- Land description = Good grass cover and high-density pasture with medium Soil permeability.

Figure 7.6. Catchment area for Williams Creek floodway site.

Table 7.2. Design ARI rainfall discharges for William Creek.

ARI (%)	1	2	5	10	20	50	100
Discharge (m^3/s)	7.03	7.76	10.1	11.65	13.2	15.31	20.66

*Note. the discharge values were determined through a catchment analysis in reference to QUDM (IPWEAQ, 2016B) and the Design Rainfall Data System (Bureau of Meteorology, 2016).

Channel characteristics:

- Hydraulic gradient 'S' = $\frac{189.13\text{ (US Level)} - 189.10\text{ (DS Level)}}{50}$ = 0.00915
- Estimated Manning's roughness coefficients 'n' from Table 2: Main channel consists of no bush and short grass = 0.035
- Channel depth = 1.79 m
- Channel Width = 8.7 m
- Channel sides = 1:1 ratio

Step 1 – Determine maximum flow the natural channel can contain.

Area, A = $\left(8.7 + \frac{1.79}{\tan 45}\right) \cdot 1.79 = 8.78\, m^2$

Wetted perimeter, P = $8.7 + \frac{2 \cdot 1.79}{\sin 45} = 13.76\, m$

Hydraulic radius, R = $\frac{A}{P} = \frac{8.78}{13.76} = 1.36\, m$

$$Flow, Q \; = \frac{1}{n} \cdot A \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}} = \frac{1}{0.035} \cdot 8.78 \cdot 1.36^{\frac{2}{3}} \cdot 0.00915^{\frac{1}{2}} = 63.13\, m^3/s$$

Therefore, the natural channel has the capacity to contain the 100-year ARI storm event.

Next a series of 0.25 m iterations for channel depth is undertaken between 0 m to the maximum channel depth of 1.79 m (Table 8.3).

Table 7.3. Iterations of channel depth to determine Stage-Discharge graph.

Depth	Area (A) (m2)	Wetted perimeter (p) (m)	Hydraulic radius (R) (m)	Velocity (V) (m/s)	(Discharge (Q) (m3/s)
0	0	0	0	0	0
0.25	2.2375	9.41	0.24	1.05	2.35
0.5	4.6	10.11	0.45	1.62	7.44
0.75	7.0875	10.82	0.65	2.06	14.61
1	9.7	11.53	0.84	2.44	23.63
1.25	12.4375	12.24	1.02	2.76	34.36
1.5	15.3	12.94	1.18	3.06	46.75
1.79	18.7771	13.76	1.36	3.36	63.13

Plot the iterations to form the stage-discharge curve (Figure 7.7).

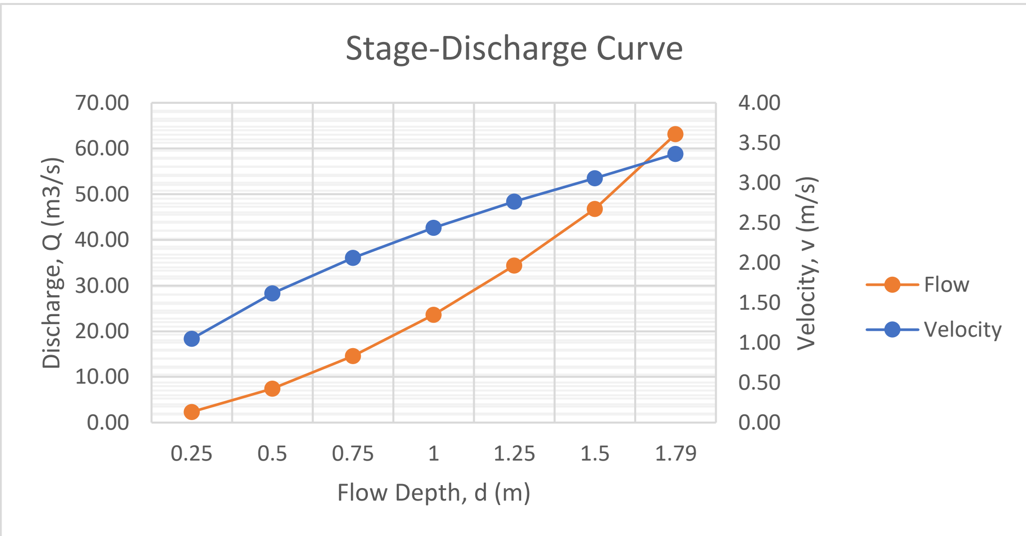


Figure 7.7. Stage-Discharge curve for Williams Creek.

<u>Determine effects tailwater and backwater effects:</u>

As the floodway is to be constructed level with the channel bed the flow regime will remain relatively unaffected. The change in roughness coefficients based on varying surface profiles (rock, concrete and natural) may result in slight variations to the tailwater and backwater levels, particularly at higher flow velocities and in the immediate vicinity of the floodway. These heights should be assessed. This assessment should be undertaken using software such as HEC-RAS as per the procedure outlined in

Example 5 or equivalent. Below is the assessment of the impact of back water level based on the 100-year ARI design discharge of 20.66 m^3/s (Figure 7.8).

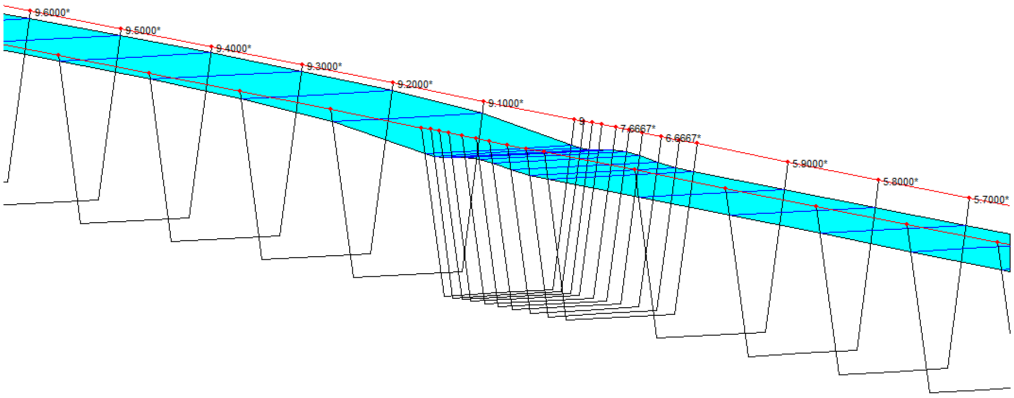


Figure 7.8. X-Y-Z perspective plot for the proposed Williams Creek floodway.

The peak back water level determined from the long section profile occurs upstream of the floodway structure at cross-section 9.400 or chainage 153.30 (Figure 7.9).

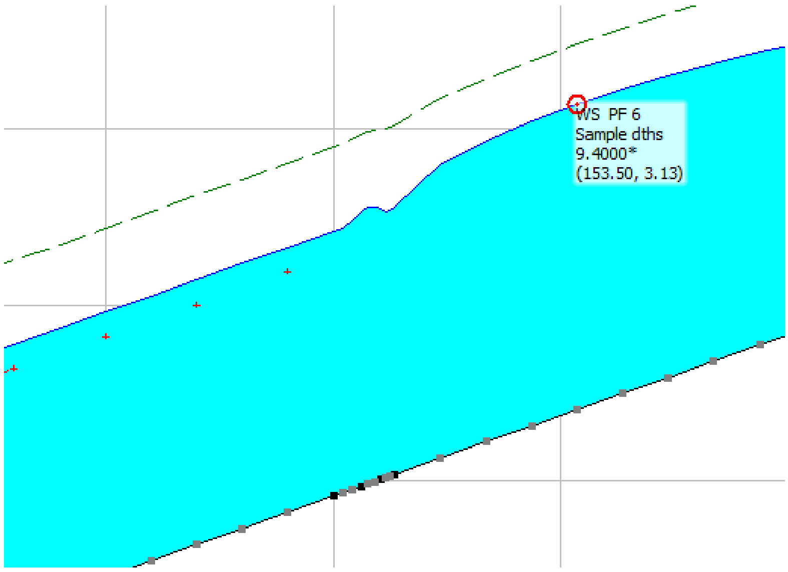


Figure 7.9. Identification of cross section which equates to the largest increase in backwater level.

Studying cross-section 9.400 shows that the backwater level at the 100-year ARI flood event is contained by the natural waterway channel and does not impose on surrounding land, therefore acceptable (Figure 7.10).

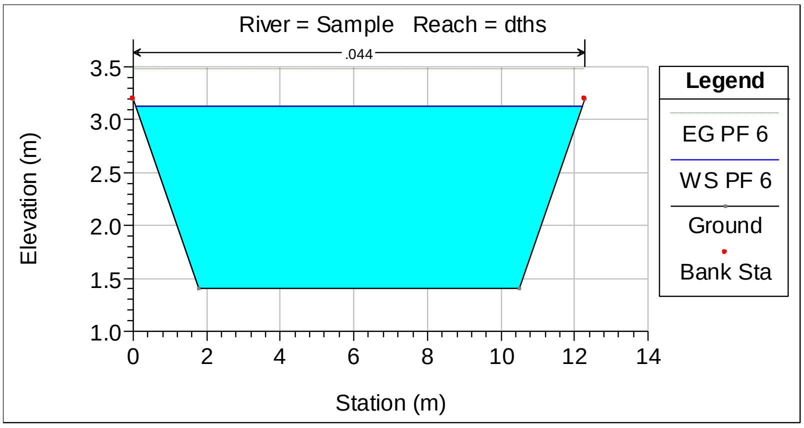


Figure 7.10. Cross section corresponding to the largest backwater level.

Determine Peak Flow Characteristics:

It is requested that the floodway can structurally withstand a rainfall event corresponding to a 1.5 m discharge depth. Therefore, the corresponding discharge and velocity obtained from the stage-discharge curve is 46.75 m^3/s or 3.06 m/s respectively. As the floodway is situated level with the channel, these flow velocities and depths represent the flow conditions at the floodway as no effect on hydraulic control is present.

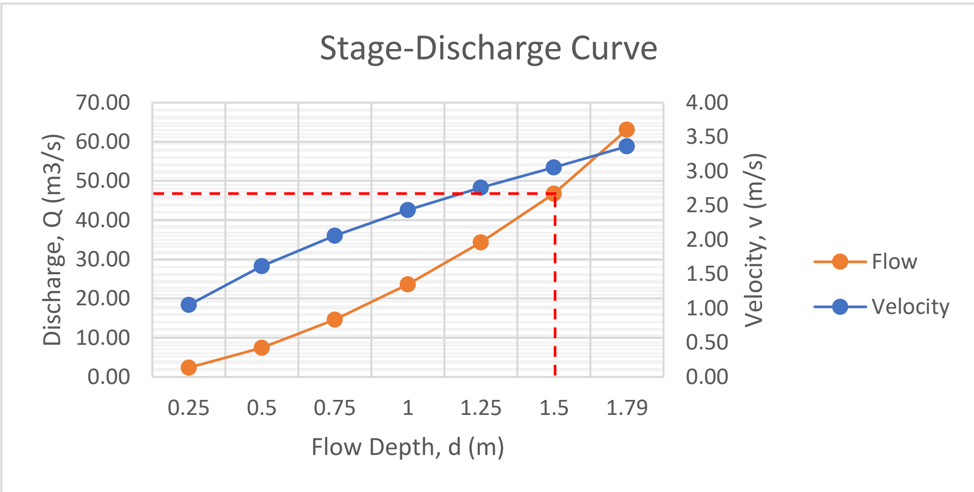


Figure 7.11. Extracting structural design velocity from the Stage-Discharge graph.

Determine Trafficability Characteristics:

Main Roads Western Australia (2006) details that the maximum permittable crossing depth should be limited to less than 300 mm for a floodway to remain serviceable and in a trafficable state. Therefore, by interrogating the Stage-Discharge curve for a flow depth of 300 mm the maximum allowable discharge can be determined (Figure 7.12).

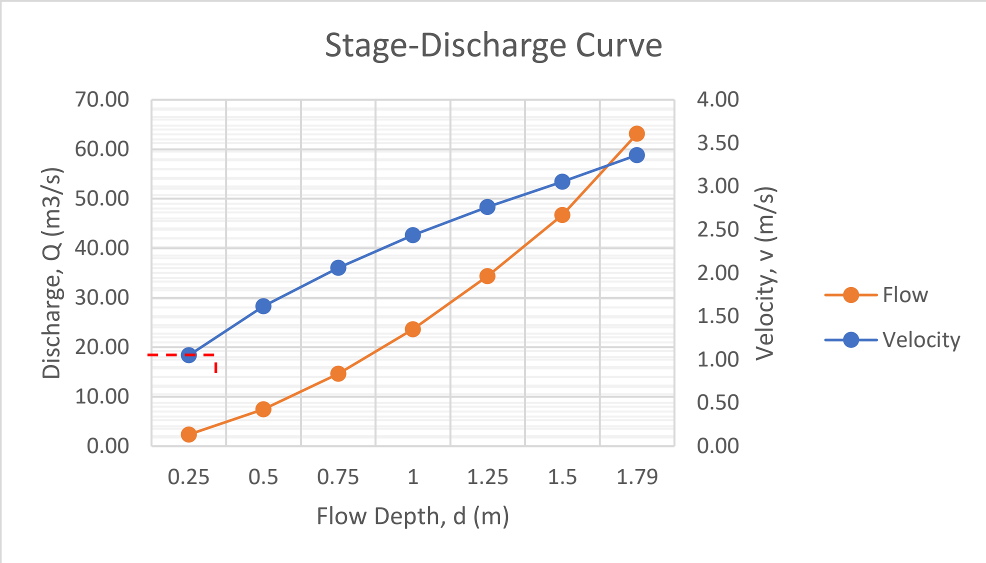


Figure 7.12. Extracting maximum trafficable discharge from the Stage-Discharge graph.

Therefore, the floodway should be closed to traffic for a discharge of $3m^3/s$ or greater which corresponds to a less than the 1-year ARI rainfall event for this catchment. In this example this is acceptable since secondary all-weather access exists to service the residences.

Step 7: Select protection type for critical zones

The velocity for use in selecting protection type for the critical locations can be selected based on the velocity determined in the hydraulic design. The HEC-RAS model should also be consulted to determine if any localised increases in flow velocity exist, if so these shall be designed for. Rock protection can be selected in reference to Tables 3.8, 3.9 and 3.10. In the case of Eight Mile Creek the flow velocity is 3.06 m/s and therefore requires either 1/4 Tonne Class dumped rock riprap with a 1.00 m section thickness or rock mattress protect 0.15 to 0.17 m thick filled with rocks 70 – 100 mm in size and 50% less than 85 mm in diameter. Either of these protection types should be adopted in the upstream, downstream and bank slope protection zones.

Step 8: Structural design of floodway

Determine the cross-sectional area of steel and concrete that satisfies requirements for strength, serviceability and durability for the critical section within floodway Type 1 in accordance with Australian Standard AS 5100.5 (Standards Australia, 2017A). Design parameter are determined from the hydraulic design and field survey. For Eight Mile Creek the following parameters are used:

- Soil type = Clay Soil
- Maximum flow velocity = 3.06 m/s
- Maximum flow depth = 1.79 m

Referencing the design charts the maximum design bending moment and shear force for use in strength and serviceability calculations is determined (Figure 7.13). Note: If an intermediate value is required linear interpolation of the graphs can be utilised.

- Maximum design bending moment = 2 kN.m
- Maximum design shear force = 6 kN

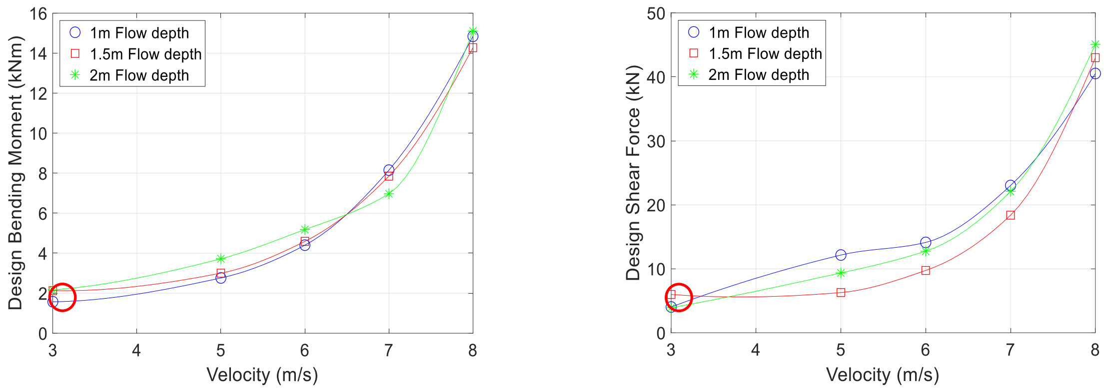


Figure 7.13. Extracting design bending moment and shear force from charts.

<u>Step 2 - Design for Durability</u>

Utilising Section 4 of AS 5100.5 (Standards Australia, 2017A) durability requirements are determined.

- Exposure classification = Class B2
- Minimum compressive strength = minimum compressive strength of 40 MPa
- Minimum cover requirements = 50 mm, assuming standard formwork and compaction

<u>Design for Strength and Serviceability in Bending</u>

Utilising Section 8.1 of AS3600 determine design for strength and serviceability in bending.

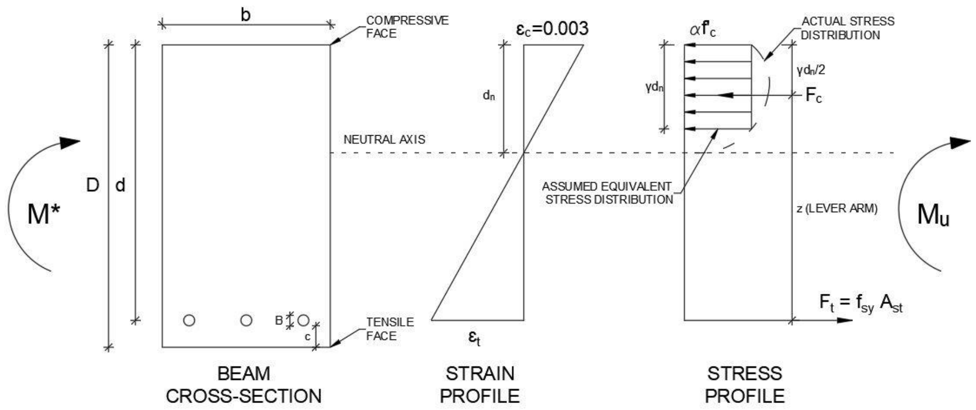


Figure 7.14. Equivalent concrete stress block - Example 4.

Select $M*$ from design graphs. Assume the use of SL62 mesh to satisfy bending reinforcement requirements.

<u>Step 2 – Design of cut-off wall for Durability</u>

Utilising Section 4 of AS 5100.5 (Standards Australia, 2017A) durability requirements are determined.

- Exposure classification = Class B2
- Minimum compressive strength = minimum compressive strength of 40 MPa
- Minimum cover requirements = 50 mm, assuming standard formwork and compaction

Step 3 - *Design of cut-off wall for Strength and Serviceability in Bending*

1. Check strength of cross-section in bending utilising the equivalent rectangular stress block presented in Figure 3.51 and equilibrium considerations to satisfy clause 8.1.2 of AS 5100.5 (Standards Australia, 2017A).

   Initially assuming SL62 mesh:

   Calculate compressive force in concrete, F_c i.e. volume of the stress block:

   $F_c = (\alpha_2 f'_c)\ (\gamma d_n)\ b$

   $F_c = (0.85)(40\ MPa)(0.826)(d_n)(1000\ mm)$

   $F_c = 28084(d_n)\ N$

   Calculate tensile forces in steel (assuming steel yields), F_t:

   $F_t = A_{st} f_{sy}$

   $F_t = (141.5\ mm^2)(500\ MPa)$

   $F_t = 70{,}750\ N$

   Equate F_t and F_c to determine the neutral axis depth:

   $F_t = F_c$

   $28084\ d_n = 70{,}750$

   $d_n = 2.52\ mm$

   With d_n evaluated the lever arm can be calculated:

   $z = D - c - B/2 - \gamma \cdot d_n/2$

   $z = 250\ mm – 50\ mm – 6\ mm/2 – ((0.826)(2.52\ mm)/2)$

   $z = 196\ mm$

   Calculate moment capacity, M_u:

   $M_u = F_t z = F_c z$

   $M_u = (70{,}750\ N)(196\ N.mm)\ x\ 10^{-3}$

   $M_u = 13.9\ kN.m$

   Check if assumed bending reinforcement is satisfactory:

   $\varphi M_u \geq M^*$

   $(0.8)(13.9) = 11.1\ kN.m\ \geq\ 2\ kN.m$

   Therefore satisfactory.

2. Check minimum ultimate strength in bending requirements at the critical section in accordance with clause 8.1.6.1.

   $A_{st} \geq [\alpha_b (D/d)^2 f'_{ct.f}/f_{sy}] b_w\, d$

   Where $\alpha_b = 0.20$ *for rectangular sections.*

$A_{st} \geq [(0.20)(250\ mm\ /\ 196\ mm)^2(2.2\ MPa\ /\ 500\ MPa)](1000\ mm)(196\ mm)$

$A_{st} \geq 280.6\ mm^2$

$141.5\ mm^2 \leq 280.6\ mm^2$

Therefore, not satisfactory since the tensile reinforcement of the cross-sectional area (A_{st}) is less than the A_{st} required to satisfy ultimate strength in bending requirements.

Try SL92 mesh:

$A_{st} = \pi r^2$ x number of bars

$A_{st} = \pi(4.5)^2 \times 5$

$A_{st} = 318\ mm^2$

$318\ mm^2 \geq 280.6\ mm^2$

Therefore satisfactory.

3. Check minimum area of steel reinforcement required to satisfy crack control against shrinkage and temperature effects:

$A_{st} \geq 0.00175bD$

$A_{st} \geq 0.00175(1000)(250)$

$A_{st} \geq 437.5\ mm^2$

$318\ mm^2 \leq 437.5\ mm^2$

Therefore, not satisfactory since the tensile reinforcement of the cross-sectional area (A_{st}) is less than the A_{st} required to satisfy crack control against shrinkage and temperature effects.

Try SL81 mesh:

$A_{st} = \pi r^2$ x number of bars

$A_{st} = \pi(4)^2 \times 10$

$A_{st} = 503\ mm^2$

$503\ mm^2 \geq 437.5\ mm^2$

Therefore satisfactory.

Outcome: SL81 mesh reinforcement to both the inner and outer faces of the floodway to satisfy bending moment requirements.

Step 4- *Design for Strength and Serviceability in Shear*

Utilising Section 8.2 of AS 5100.5 (Standards Australia, 2017A) determine design for strength and serviceability in shear.

1. Calculate the contribution of concrete to shear strength, V_{uc}:

$V_{uc} = k_v b_v d_v (f'_c)^{½}$

Where,

b_v = effective width of a web for shear = $900\ mm$

$d_v = max(0.72 \cdot D, 0.9 \cdot d) = max\big(0.72 \cdot 250, 0.9 \cdot (250 - 50 - 3)\big) = 180\ mm$

$K_v = assume\ \frac{A_{sv}}{s} \geq \frac{A_{sv.min}}{s} :\ k_v =\ 0.15$

$$V_{uc}\ =\ k_v b_v d_v (f_c')^{\frac{1}{2}}$$

$$V_{uc}\ =\ 0.15 \cdot 900 \cdot 180 \cdot 40^{\frac{1}{2}}$$

$$V_{uc}\ =\ 170.8\ kN$$

$$\emptyset V_{uc} = 119\ kN$$

$$\emptyset V_{uc} \geq V^*$$

$$119\ \geq 6\ kN$$

Thus, the shear capacity of concrete alone is adequate to withstand the design shear force (V^*).

2. Check if minimum shear reinforcement is required. Since the member thickness is less than 300 mm, transverse shear reinforcement is not required as per clause 8.2.1.6 (Standards Australia, 2017A).

Step 9: Produce detailed design and structural drawings.

Detailed structural drawings for the Eight Mile Creek, Category A floodway (Figure 7.15). It would also be expected that a civil drawing set be produced for the road approaching and departing the floodway based on standard road design principles.

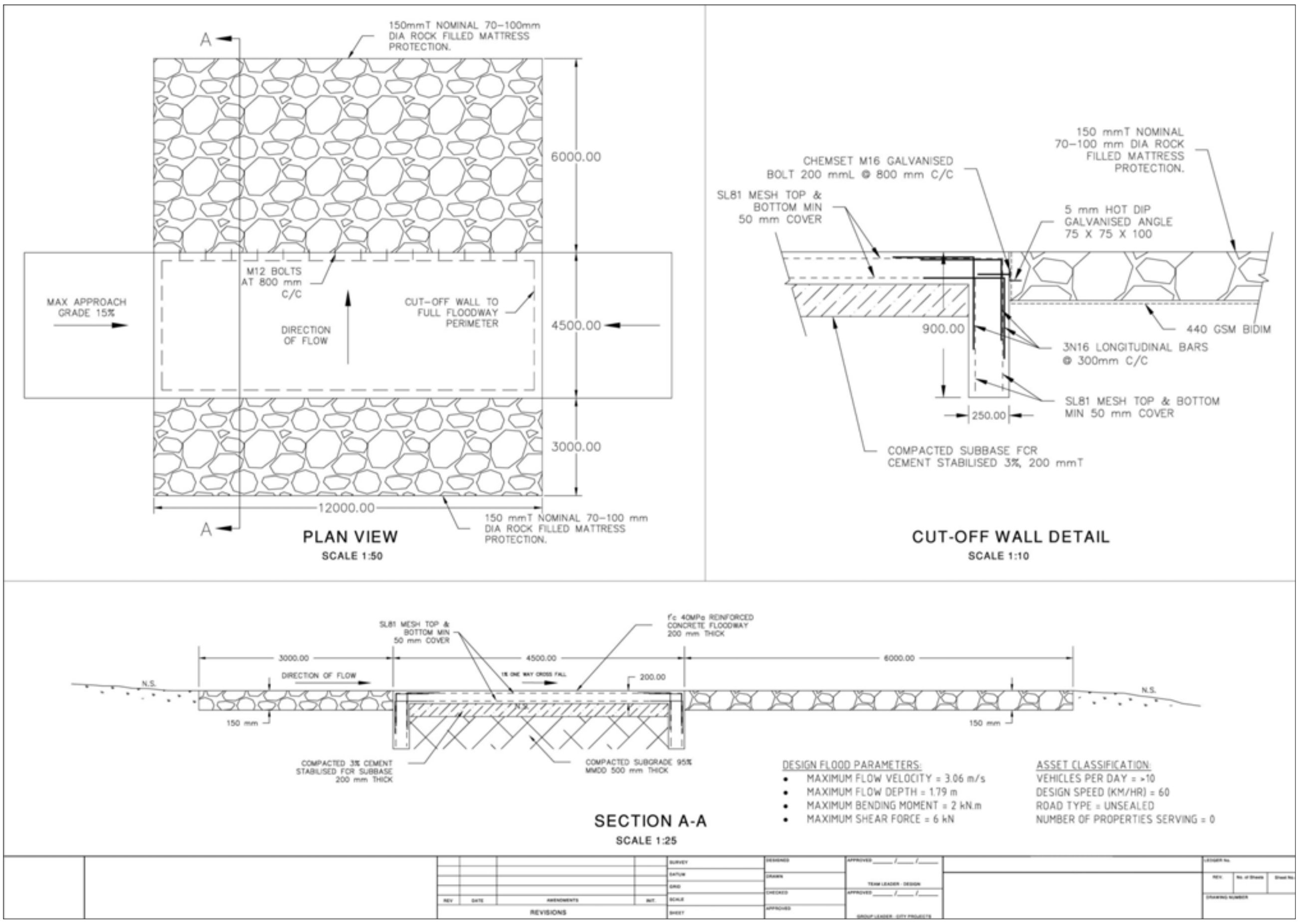


Figure 7.15. Example 4 - Category A floodway design drawing (Williams Creek).

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APPENDIX A – DESIGN REFERENCE TABLES

Manning's coefficient of roughness values:

Table A.1. Manning's roughness coefficients (Main Roads Western Australia, 2006).

Type	Manning's roughness coefficient 'n'	Notes
Minor Waterways		
Grass and weeds	0.030 – 0.035	If channel is irregular channel, has pools of water or channel meanders increase by 0.010 – 0.020.
Dense growth of weeds	0.035 – 0.050	" "
Some weeds, light brush on banks	0.035 – 0.050	" "
Some weeds, heavy brush on banks	0.050 – 0.070	" "
Mountain waterways with gravel and no boulders	0.040 – 0.050	" "
Mountain waterways with large boulders	0.050 – 0.070	" "
Flood plains		
Pasture with no brush.	0.030 – 0.070	Variable based on grass length i.e. short to long.
Cultivated areas	0.030 – 0.050	Variable based on crop maturity i.e.no crops, mature row crops or mature field crops.
Brush	0.050 - 0.070	Scattered brush.
Trees	0.050 – 0.070 0.060 – 0.080 0.100 – 0.160	Scattered brush/heavy weeds. Light brush and trees. Dense brush and trees.
Major Waterways		
Subtract 0.020 from the Manning's roughness coefficient 'n' used in minor streams as banks will cause less resistance.		

Mechanical properties of watercourse in-situ soil types:

Table A.2. Mechanical properties of the water courses in-situ soil.

Material Type	**E (MPa)** [11]	ν [12]	**ρ (kg/m³)** [13]	**c' (MPa)** [14]	**ϕ (°)** [15]	**K0** [16]	**e** [17]
Soil 1: Silty Sand (Silts and silty sands)	40	0.3	1,700	0.01	25	0.426	0.4
Soil 2: Sandy Soil (Poorly graded sands with fines and gravely-sand-silt mixtures)	30	0.25	1,800	0.075	34	0.44	0.3
Soil 3: Clay Soil (Clays of varying plasticity and clayey fine sands and silts)	100	0.3	1,900	0.01	20	0.658	0.15

[11] Modulus of Elasticity (MPa)
[12] Poisson Ratio
[13] Density (kg/m^3)
[14] Cohesion (MPa)
[15] Angle of friction (degrees)
[16] Horizontal stress ratio
[17] Void ratio