University of Southern Queensland

Faculty of Engineering and Surveying

Investigation of Scour Mitigation Methods for Critical Road Structures

A dissertation submitted by

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in fulfilment of the requirements of

ENG4111 and ENG4112 Research Project

towards the degree of

Bachelor of Engineering (Honours) (Civil)

Submitted October, 2016

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Abstract

The flood events that occurred in 2011 and 2013 in Queensland are notable for their devastating outcomes and damages to critical road structures. Bridges are necessary for the local community to travel and provide disaster relief during times of need. Therefore, it is important to identify methods that prevent bridge scour.

To identify these countermeasures, a literature review of critical infrastructure scour prevention method was conducted. Methods that are appropriate were then analysed using the hydraulics software HEC-RAS. The Tenthill Creek Bridge was chosen as the framework of the analysis. Bridge scour depths were modelled and each method was compared. Combined with the HEC-RAS analysis, the feasibility analysis shows that a combination of collaring, riprap and wing walls is the most cost effective in decreasing the scour depths at piers and abutments.

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ENG4112 Research Project Part 2

Limitations of Use

Certification of Dissertation

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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Acknowledgements

I would like to thank my supervisor Dr Weena Lokuge for her tremendous guidance and the input of both Dr Buddhi Wahalathantri and postgraduate student Reza Shahnawaz.

I would also like to thank my partner and my family for their endless support and encouragement throughout my research.

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List of Abbreviations

ARI	Average Recurrence Interval
GCL	Geosynthetic Clay Liner
HEC-RAS	Hydrologic Engineering Centre's River Analysis System
LVRC	Lockyer Valley Regional Council
WMIP	Water Monitoring Information Portal

Chapter 1 Introduction

1.1 Background

Heavy rainfall events often trigger flood events, submerging dry land with water. This can occur to any type of water body, such as rivers, oceans or lakes. Flooding is a global phenomenon that causes damage buildings, transportation networks and other infrastructure. The results of flooding can be extremely fatal, with the deadliest 1931 Yellow River Floods taking four million lives (Hudac 1996).

Despite being known for its severe dry seasons, Australia is not foreign to flood events. The wet season during December 2010 and early January 2011 triggered significant flooding throughout Queensland, resulting in the declaration of 78% of the state as a disaster zone (Queensland Government 2015). During this period, Queensland has seen $7 billion worth of damage, experiencing above average to highest on record for rainfall (Pritchard 2013). Significant damage was done to road networks and critical road structures, such as bridges, floodways and culverts. In particular, 5% of the damage cost was allocated to the repair of bridges.

During the second week of January 2011, a rain event caused extreme flash flooding in Toowoomba and the Lockyer Valley region and major flooding in Brisbane. In the Lockyer Valley Regional Council (LVRC) area, significant damage of dealt to the road network (McPherson 2011):

- Sealed Roads: 16% replaced
- Unsealed Roads: 53% required resheeting
- Bridges: three replaced, railings replaced on most bridges
- Culverts: 256 out of 2500 replaced (10%)
- Floodways: 58% damaged
- $180 million total repair bill

Despite the insights from the 2011 Floods, Lockyer Valley region experienced damage once again during the 2013 flood event caused by Ex-Tropical Cyclone Oswald. Major rural streams results in even more significant runoff as stream banks were drier, recently burnt, or suffered loss in grass biomass (Warner 2013). This caused bank erosion to more agricultural land and road infrastructure loss.

Through the significant financial and social damages, the legacy of the Queensland floods has increased awareness of flood risk management in Australia. It is therefore important to seek methods to increase the resilience of road networks under extreme flood events.

1.2 Project Aim

The aim of this research project is to identify and analyse potential solutions to reduce or minimise soil erosion on critical road structures, specifically for bridges. The solution will then be analysed if it is applicable to reduce scour.

1.3 Project Objectives

The primary objective of this project is to research the available methods used around the world to reduce scour in critical road structures. This involves current methods used by Australia and additional methods published in journals. As scour mitigation methods of bridges are limited, the literature review will also focus on methods of floodways and culverts; from the methods found, the solutions applicable for bridges will be chosen.

The secondary objective of this project is to analyse the chosen method using softwares such as HEC-RAS. This analysis will determine whether the selected solution is appropriate for practical implementation.

1.4 Justification of Project

Determining an appropriate method for scour mitigation will allow authorities to prepare and evaluate the flood resistance of a critical road structure. By increasing the scour resilience of bridges, it will decrease effect of floods and its financial and social consequences. An undamaged bridge after flood events will be able to serve its purpose and provide disaster relief for its users.

1.5 Dissertation Outline

The dissertation will include the following:

Chapter 2 – Assessment of sustainability, safety and ethical effects

Chapter 3 – Literature review of scour and its mitigation methods

Chapter 4 – Methodology of analysis

Chapter 5 – Modelling of bridges conducted using HEC-RAS and its results

Chapter 6 – Analysis of results obtained from HEC-RAS

Chapter 7 – Conclusions and recommendation of future work

Chapter 2 Assessment of Consequential Effects

A preliminary assessment of the sustainability, safety and ethical effects of the research will be outlined in this chapter. The complete risk assessment can be found in Appendix F.

2.1 Sustainability

This consequential sustainability effects will be mainly positive. Finding a solution to increase a critical road structure flooding resilience will decrease the need to repair and frequency of maintenance altogether. This allows road structures to be sustainable in the long term, decreasing resource requirements and further expenditure. However, if the solution is not environmentally friendly, it may affect the structure's local ecosystem. It is therefore important to verify that the chosen solution will not impact the environment negatively.

2.2 Safety

If a solution of scour prevention is successfully achieved, it will provide positive safety consequences. After extreme flood events, the scouring around the structure is mitigated, meaning that the critical road structure can service a community in need of transport.

2.3 Ethical Issues

The project outcome does not carry ethical issues that will breach the Code of Ethics (Engineers Australia 2010), and is achieved on the basis of a well-informed conscience. The experiments will be conducted in an ethical manner, where modelling and analysis will not be influenced by the expected outcomes

Chapter 3 Literature Review

As defined by Middelmann et al. (2014), flood is “water which we don’t want.” In the past decade, the intensity of floods in Australia has increased, causing damage to road structures across the country. According to Setunge et al. (2015), the causes of flooding in Australia can be categorised as the following:

- Storms and cyclones
- Coastal flooding
- Spring thaw
- Heavy rains
- Levee and dam failure

In particular, the triggering factors of a flood event include:

- Rainfall intensity
- Spatial variation
- Weather condition and catchment
- Topography
- Runoff capacity of stream network
- Tidal influence
- Total rainfall amount

When drainage is poor, the risk of flash flooding is increased especially in urban or rural areas after intense rainfall (Lebbe 2014).

3.1 Failure of Road Structures under Flood Conditions

Bridges, culverts and floodways are subjected to flooding, causing scour, debris impact and removal of support. This often disrupts the road system, requiring road closure for repairs and maintenance.

3.1.1 Failure Mechanisms of Floodways

Floodway is a cost effective solution designed to be overtopped by floodwater, especially during lower average recurrence interval (ARI) floods. In comparison with road structures such as culverts and bridges, floodways reduces the risk of soil erosion more significantly due to less concentrated flow, thus providing environment advantages.

Failure Zones

The four main failure zones of floodways, as identified by Allen et al (2012), are:

1. Upstream zone: section of creek immediately upstream of roadway shoulder
2. Roadway zone: section of road enclosed and including road shoulders
3. Downstream zone: section from the roadway shoulder to the creek channel
4. Peripheral zone: section outside of the three zones, including vegetation

Different zones can be subjected to different modes of failure. The floodway may be deemed beyond repair if there is significant damage to the three main zones.

Different stages can carry different problems, ultimately leading to failure (Wahalathantri et al. 2016):

1. Design stage:
   - Insufficient discharge capacity: inability to convey flood flow
   - Misalignment: higher loads in some parts of the floodway
2. Construction stage:
   - Imperfections in material: reduce in strength, causing failure prematurely
3. Maintenance stage:
   - Vegetation
   - Not detecting minor damage
   - Aging
4. Operational stage: failure of one or more zones

Causation of Failure

Floodway failure mechanism is heavily concerned with the failure zones and causes of failure (Setunge et al. 2015). The three main causes are:

- Erosion
- Deposition
- Infrastructural failure

The most common cause of floodway failure is erosion and occurs most frequently at downstream areas, where its severity depends on the soil type and flow velocity. In comparison, upstream erosion at floodways occurs less frequently. Additionally, roadway erosion is also common as it is caused by poor drainage and increased flow velocity at the downstream end of the roadway. Erosion leads to the failure of the structure and can also be the causation of the creation of new flow paths.

An example of floodway failure due to erosion is the failure of the Blue Waters floodway in 2007 (Setunge et al. 2015). The floodway eroded due to the malfunction of the drainage system, allowing stormwater to flow along the roadway. Coupled with the change in material properties and the increase in velocity at the downstream end, the roadway zone erodes as a result.

During flood events, the expansion of the creek cross-section leads to deposition. Deposition rarely causes failure in infrastructure, but still results in difficulty for passing traffic.

Figure 3.1 shows the different types of failure stages and causations:

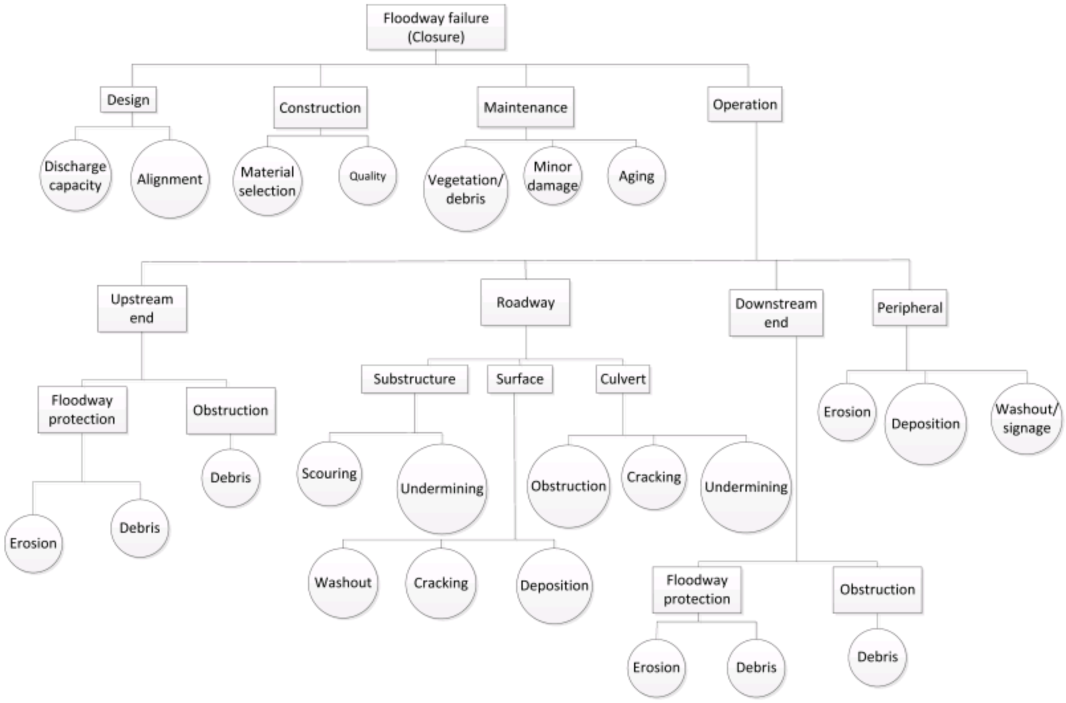

Figure 3.1, Operational failures in detail (Wahalathantri et al. 2016)

3.1.2 Failure Conditions of Bridge Structures

In general, failure of bridge structures are caused by scour, overload, overflow, lack of maintenance, construction lacks and structural lacks (Setunge et al. 2015). Particularly, scour is the most prevalent cause of bridge failure, which can be divided into three categories:

1. Local scour: removal of soil around bridge piers and increase in flow velocity, causing vortices
2. Contraction scour: removal of soil from the bed, increasing shear stress and enhancing the discharge
3. Long-term degradation scour: the result of man-made or natural causes, effecting the bridge’s reach of the river

Generally, bridges that fail due to scour were not properly designed with the hydraulic effects in mind (Lebbe 2014). In particular, the maximum scour depth, river flow patterns and basic features were not considered during its design phase.

Similar to floodways, bridges also experience failure due to deposition and infrastructure failures. Moreover, waterborne debris is also a major factor in bridge failure. Damages of superstructure and structure displacement are often caused by debris and log impact. This is a major issue as Australian standards do not address urban debris which is more significant in comparison to normal log impacts (Setunge et al. 2015).

During heavy rain, flood debris from upstream areas commonly appears in streams, including vegetation, trees, mud, soil, artificial structures and food waste (Setunge et al. 2015). This is potentially problematic as it leads to critical failure, blocking the waterway which would intensify the loading on the pier. Accumulating on top of the floodwater, the lateral displacement causes the support of the bridge to be overturned, resulting in bridge foundation scour.

From a case study conducted in Lockyer Valley in January 2013, 46 bridges were inspected in the region (Lebbe 2014). It was observed that damaged bridge approach and pier, and abutment scouring are observed as the most common causes of failure. Other causes include the built up of debris and mud on the structure, cracks in the abutment wing walls and the disconnection between the abutment headstock and the piles.

3.1.3 Failure Mechanism of Culverts

There are a number of geotechnical factors that influence the failure of culverts and accelerate the aging process. Corrosion occurs to metallic culverts and is caused by the reaction to water and soil. As a result, metal is removed from the pipe, reducing the culvert thickness. Corrosion also occurs when the culvert is subjected to high amounts of stress and consequentially leads to the failure of the structural shape

(Tenbusch 2009). The instability of the local ground can also cause culvert failure. Unanticipated slow ground movement increases the load applied to the culvert, causing instability and may lead to sudden embankment failure. Erosion at the downstream end of the culvert reduces the overall strength of the structure. The removal of soil due to erosion may cause the deflection to exceed the culvert’s limits, and eventually buckles, removing the culvert and the embankment. During heavy rain, debris can block the culvert opening, leading to the overload of loads, causing structural failure.

3.2 Bridge Scour Mechanics

Scour mitigation methods are largely based on laboratory results, and the knowledge of specific effects on critical road structures still require further understanding (Department of Transport and Main Roads 2013). It is therefore important to examine the scour process and its characteristics.

3.2.1 Indicators of Scour

Scour is generally caused by flowing water, excavating soil from the stream bed and around the base of the critical road structure. Currently, there are limited amounts of equations applicable to evaluate scour depth and results are largely based on laboratory experiments (Department of Transport and Main Roads 2013). Scour is an extremely complex process that is influenced by multiple factors (Melville & Coleman 2000). When observing the potential of soil erosion, the influencing factors are:

- Geomorphic:
    - Stream size
    - Flow habit
    - Valley setting
    - Floodplain

    - Banks
    - Bed levels
    - Channel slope
- Hydraulic:
    - Flood stages
    - Flood flows
    - Flood frequencies
    - Water surface profile
- Land use changes:
    - Deforestation
    - Agricultural activity
    - Land clearing
    - Fire
    - Catchment vegetal cover
    - Sediment dumping
    - Channel and debris clearing
    - Flow diversion

From the numerous elements listed above, it can be hypothesised that predicting scour carries high difficulty as is exceedingly complex. Although predictive methods exist, the methods are highly reliant on laboratory results, hence are less practical when applied (Department of Transport and Main Roads 2013).

3.2.2 Types of Scour at Bridges

Bridge scour can be categorised into three types:

- Local scour: abutment scour and pier scour
- Contraction scour
- Long-term degradation scour at stream bed

An example of local scour and contraction scour can be found in Figure 3.2.

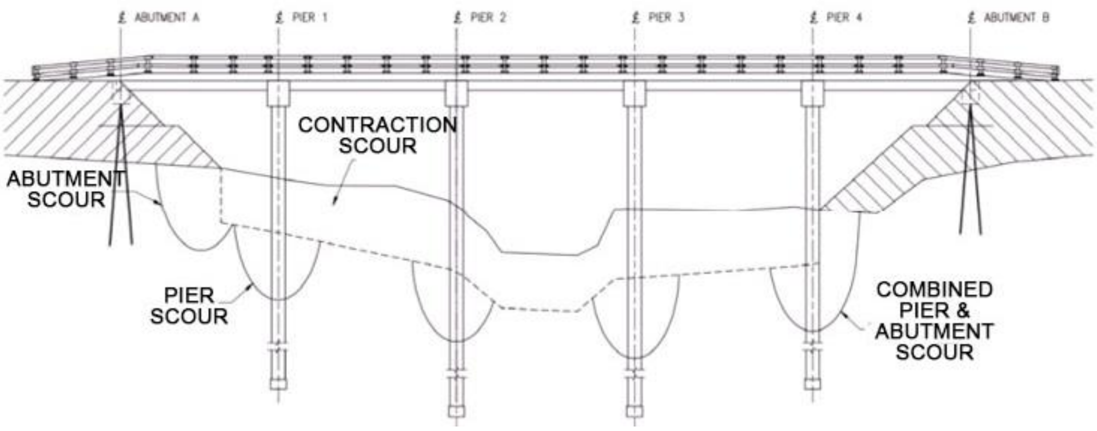

Figure 3.2, Scour locations on a Bridge (Department of Transport and Main Roads 2013)

Local Scour: Pier Scour

Pier scour occurs due to the increase in flow velocity and the generation of waves around the pier. When the flow first hits the pier, a downflow is created, which generates vortices at the base of the pier. The vortices then cause the scour hole and continue to grow until equilibrium is reached (Department of Transport and Main Roads 2013). The size and depth of the scour hole depends on the size and shape of the pier. More detail of the flows and vortices involved in pier scour can be found in Figure 3.3.

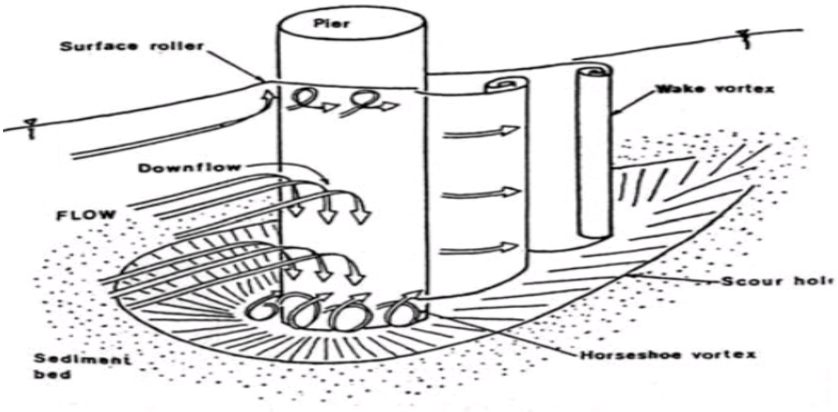

Figure 3.3, Flow activity on a pier and pier scour (Department of Transport and Main Roads 2013)

Local Scour: Abutment Scour

Abutment scour is an extremely complex process and depends on numerous factors. Its scour depth depends on the abutment shape, flow around the abutment and channel, cross-section shape of the channel and field conditions – whether there is vegetation on the abutment and stream (Department of Transport and Main Roads 2013). Abutment scour can happen at the main channel bed and embankment, and can be worsen by the scour at the flood plain and contraction scour (Agrawal et al. 2005). Detail of the flow and vortices around the abutment can be seen in Figure 3.4.

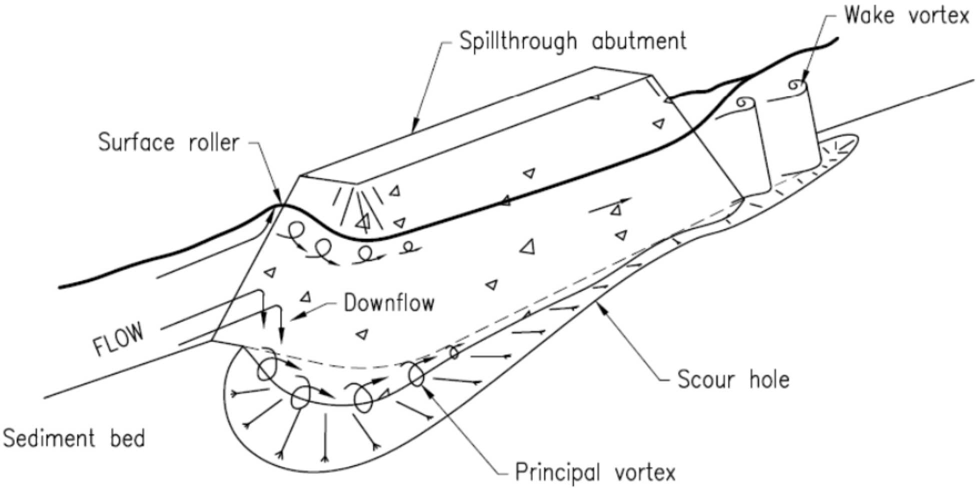

Figure 3.4, Flow activity on abutments and abutment scour (Department of Transport and Main Roads 2013)

Contraction Scour

Contraction scour occurs when the flow area suddenly decreases, consequently increasing the flow velocity. In turn, the erosive forces around the contraction increase, removing bed material from the upstream to the downstream. Contraction scour is different from long-term degradation scour as contraction scour is often triggered after a flood event around the bridge structure, whereas long-term degradation scour occurs after a long period of time on the entire streambed

(Department of Transport and Main Roads 2013). Figure 3.5 shows the flows constrained by the contraction and the process of contraction scour.

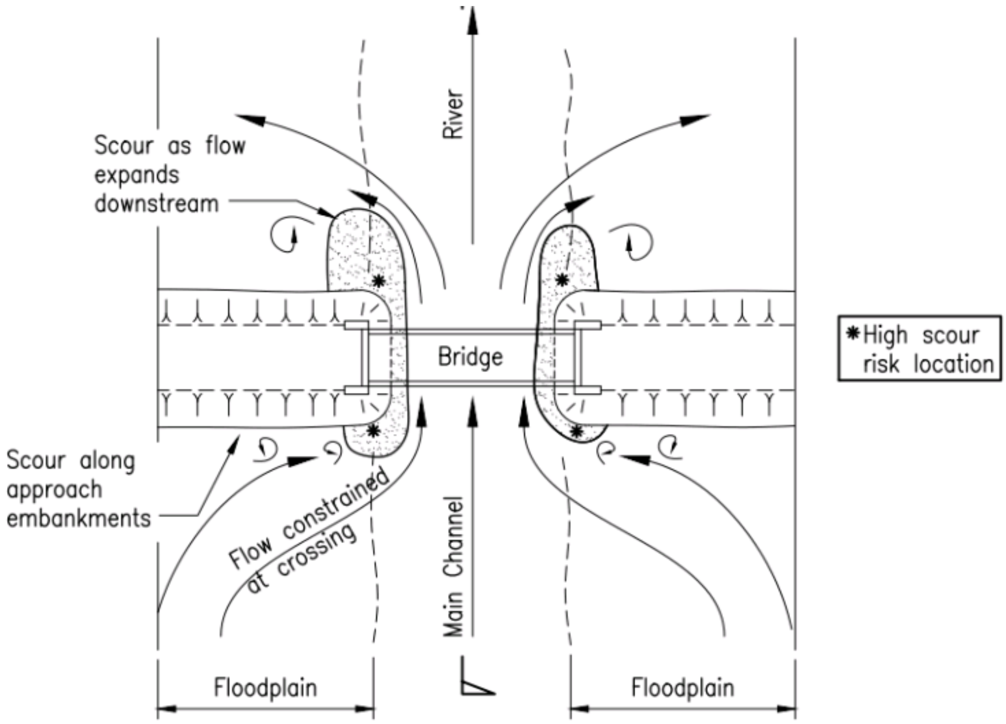

Figure 3.5, Contraction scouring at a bridge (Department of Transport and Main Roads 2013)

3.3 Methods of Scour Prevention

3.3.1 Bridge Abutment Scour Prevention Methods

There are numerous abutment scour countermeasures for bridges. These methods aim to stabilise the abutment and aligning and guiding the upstream flow (Agrawal et al. 2005). The types of abutment scour prevention methods are:

- Riprap: relatively low cost and maintenance, extremely flexible to adjust.
- Guidebanks: guides a flood plain through an opening, often used when the channel flow is undesirable.

- Spur dikes: forces the realignment of the channel flow, the channel is often widened to decrease flow velocity (Figure 3.6).

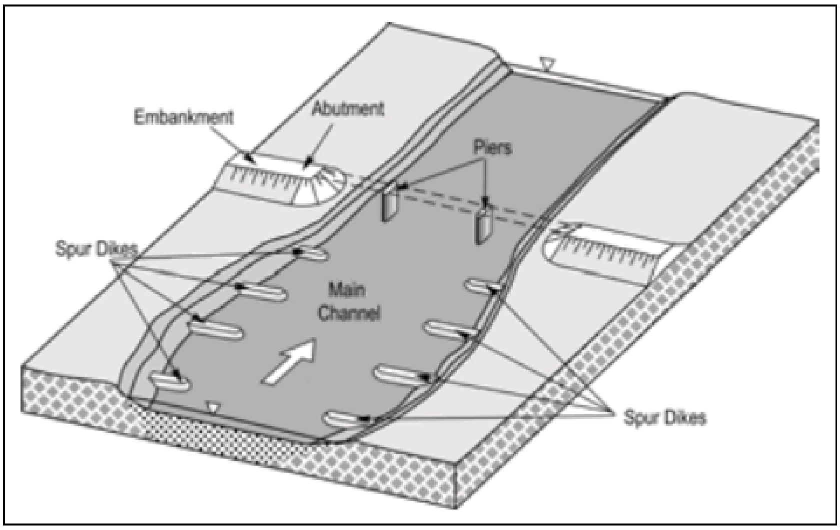

Figure 3.6, Spur dikes at the upstream of the Bridge (Ettema et al. 2006)

- Bridge widening: only used when other scour countermeasures are infeasible or when the abutments are already washed out, a pier is often added, an illustrated diagram is shown in Figure 3.7.

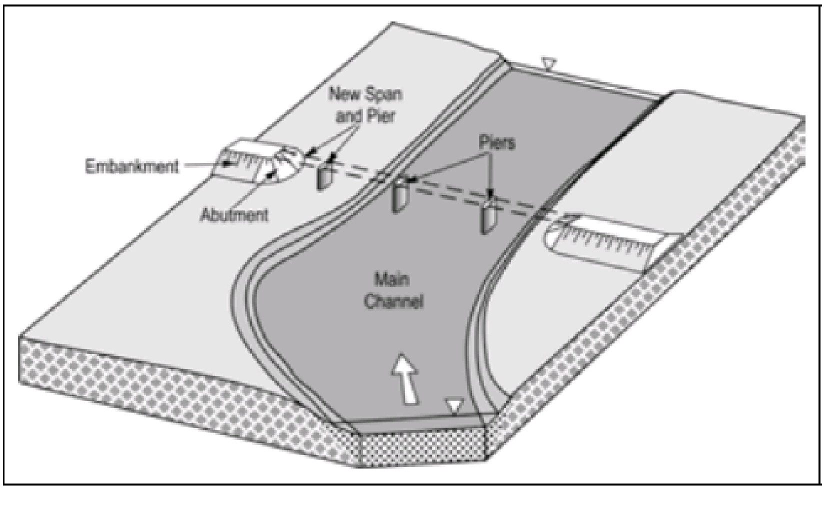

Figure 3.7, Bridge channel widening (Ettema et al. 2006)

- Removal of small and large trees: decreases the possibility of the channel being blocked by debris.
- Concrete filled mattresses: blocks do not washout and is easy to construct, however can be lifted by heavy flows.
- Wing walls: can be independent or attached to the abutment, extremely economical; acts as a retaining wall for the abutment and guides the stream into the bridge (Figure 3.8).

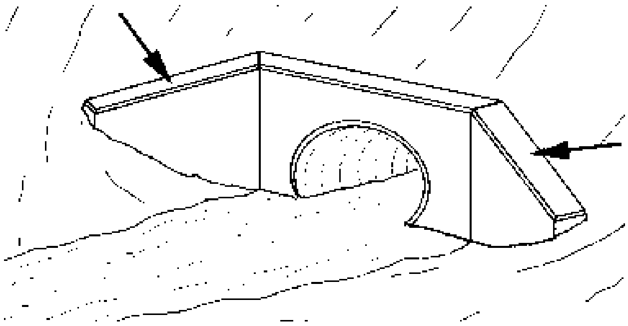

Figure 3.8, Wing wall around a culvert (Ettema et al. 2006)

- Spill-through abutments: a pier that is used as an abutment when an additional span might be added in the future, effectively limits scour depth; early slope erosion and geotechnical failure, shown in Figure 3.9, are frequent problems.

Figure 3.9, Geotechnical failure of spill-through abutments (Ettema et al. 2006)

3.3.2 Bridge Pier Scour Prevention Methods

Riprap and Collaring

In bridge structures, scour exposes the foundations by lowering the level of the river bed (Cheremisinoff et al. 1987). During peak flow, the flow velocity is higher which assists the occurrence of scouring. It is often during this flow condition that scour holes are produced around bridge piers.

Riprap aims to mimic a natural streambed, allowing for sediment transport, flood routing and debris conveyance (Crookston et al. 2012). Riprap is commonly used to prevent scour at the piers and abutment of bridges. From a study conducted by Kayaturk et al. (2004), a collar reduces the scour depth by 97% and decreases the rate of development of a scour hole. Furthermore, collaring works on both rectangular and circular piers.

Combining riprap with collaring, the risk of scouring at piers is effectively reduced as the direction of flow is altered (Figure 3.10). When a collar is added, the maximum reduction in scour depth is 57%. Additionally, the riprap volume required for scour protection is decreased when a collar is introduced. (Zarrati et al. 2010)

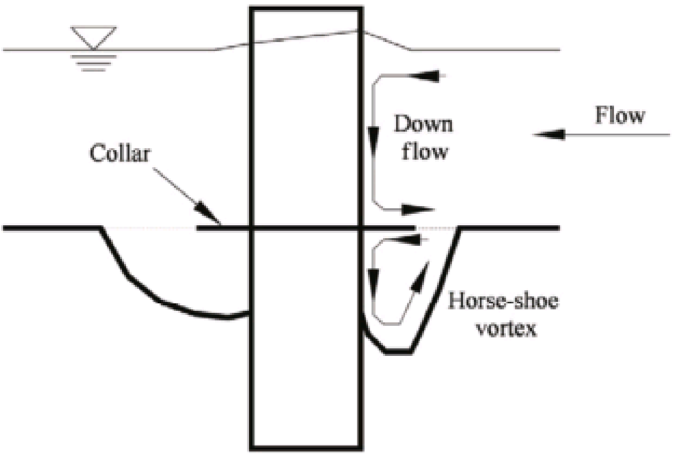

Figure 3.10, Effects of a collar (Zarrati et al. 2010)

Flow-Altering

In addition to armouring countermeasures, flow-altering also effectively prevents scour around bridges. This involves modifying the flow or break-up vortices to suit the site conditions. However, altering flow may not be cost effective as new structures will be built (Setunge et al. 2015).

Openings Arrangement Technique

Around the vicinity of the pier base, the interaction with the water flow and the scour hole creates a horseshoe vortex. As the scour depth increases, the horseshoe vortex gradually diminishes. A method developed by Entesar et al. (2013) involves the use of openings along the piers side as shown in Figure 3.11. This effectively decreases scour depth by 45% as the vortex formation in front of the pier is reduced.

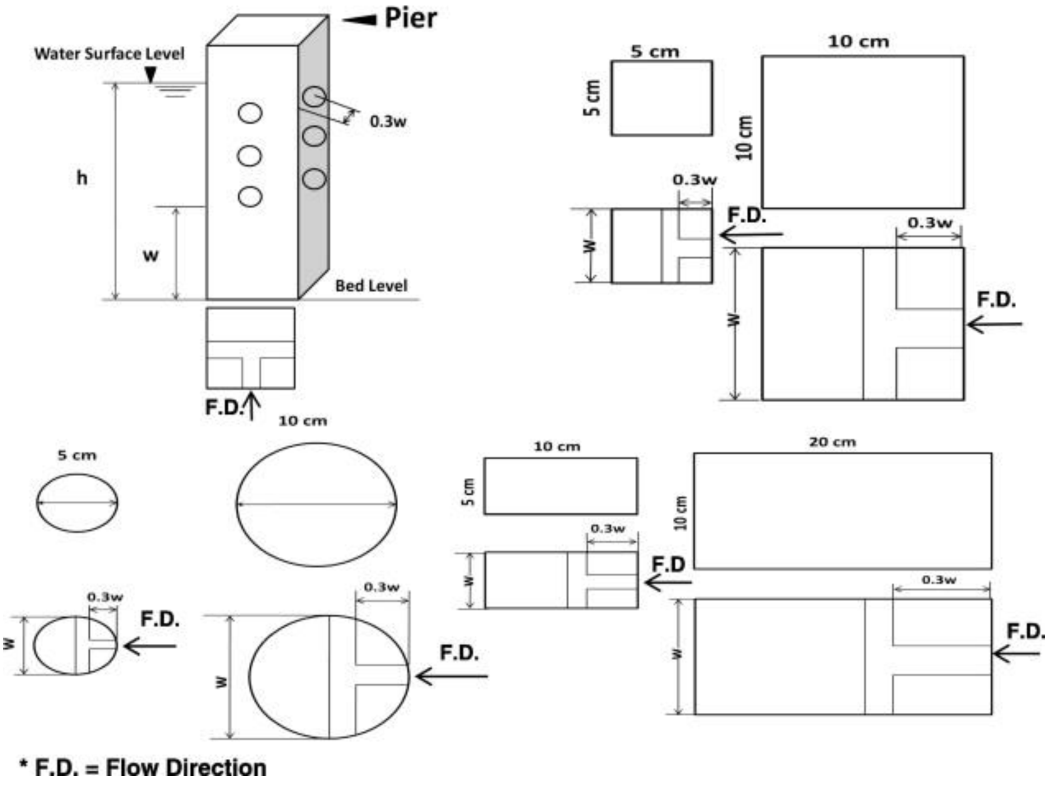

Figure 3.11, Piers shapes and opening arrangements (Entesar et al. 2013)

Cable and Collar

A study conducted by Izadinia et al. (2012) found success in using cable and collar to reduce scour around bridge piers. An illustration of how the pier is prepared is shown in Figure 3.12. The purpose of the cable is to improve the efficiency of collar even more. In particular the best cable-pier diameter ratio is 0.15 and a cable thread angle of fifteen degrees. The scour depth reduction in comparison with installing a collar only is 12.85%.

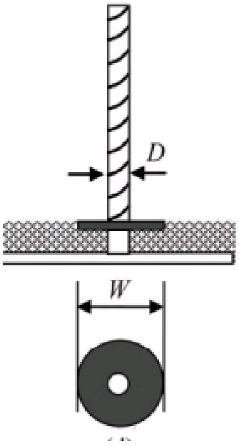

Figure 3.12, Pier with cable and collar (Izadinia et al. 2012)

Ring columns

A ring column is a scouring countermeasure that involves interlocking rings, such as the ones shown in Figure 3.13. Along with the irregular surface, the rings allow the water to flow through the gaps, reducing the strength of the horseshoe vortex and water flow. The optimal configuration reduces the scour depth by 65% (Wang et al. 2011).

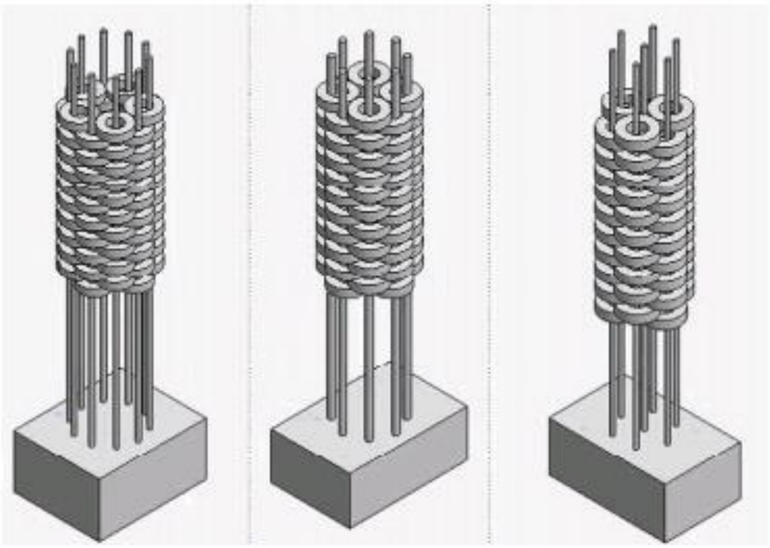

Figure 3.13, Configuration of different ring columns (Wang et al. 2011)

3.3.3 Erosion Protection Methods in Floodways

Depending on the position of the floodway, the type of scour prevention method is chosen after investigating its flow velocities, orientation, condition and performance. Scour generally occurs on the pavement, within the channel and at the shoulders and batters of the floodway. Table 3.1 illustrates the considerations of floodway position and the appropriate erosion protection:

Table 3.1, Erosion position and protection type (GHD 2010)

Position	Considerations	Erosion Protection
Upstream	• Approaching flow velocity • Submerged period • Direction of flow with respect to floodway	• At road shoulder and top of the road batter • Similar protection with downstream batter
Pavement	• Traffic volumes during wet and dry periods	• Use flexible or rigid unsealed pavement

	• Overtopping duration • Erosion damage potential • Cost of construction and maintenance	
Downstream Embankment	• Approaching flow velocity • Direction of flow with respect to floodway	• Either flexible or rigid • Avoid sharp steps or grade changes

The Austroads “Waterway Design – A Guide to the Hydraulic Design of Bridges, Culverts and Floodways”, the “Road Drainage Manual – A Guide to Planning, Design, Operation and Maintenance of Road Drainage Infrastructure” prepared by the Queensland Department of Transport and Main Roads, and Main Roads Western Australia’s “Floodway Design Guide” are all design guides that provides detailed reference for floodways.

The “Floodway Design Guide” provides numerous flexible protection methods:

Riprap

Riprap, shown inFigure 3.14, is the most affordable type of erosion prevention method, which features graded rock dumped on a treated slope. To protect the floodway from high velocities at the change of grade, a toe length of 1 to 1.5 times the embankment height is prepared. Depending on the flow velocity, the class and thickness of the rock used can be determined. Additionally, a concrete cut-off wall might be included between the pavement and rock riprap if high velocity is expected.

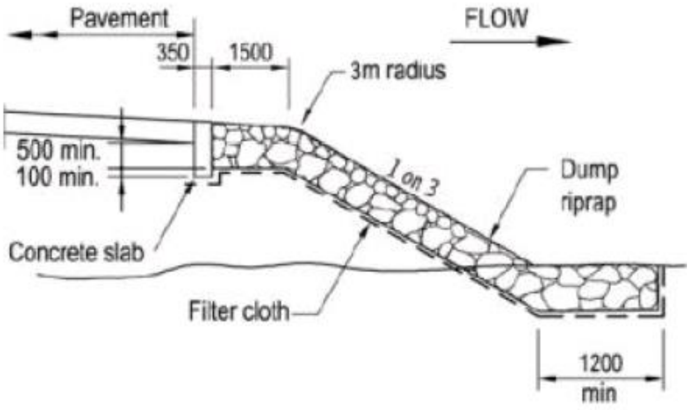

Figure 3.14, Riprap protection (Department of Transport and Main Roads)

Rock Mattresses

Shown in Figure 3.15, rock mattress is a method that involves placing rocks within wire baskets or wire covering. This method is used when dumped rock is not available locally or cannot be imported economically. Similar to rock riprap, a suitable length toe is required and a cut-off wall at the interface may be required. It is also important to ensure that the wire enclosure should be smaller than the rock and the wire is coated with PVS to reduce corrosion.

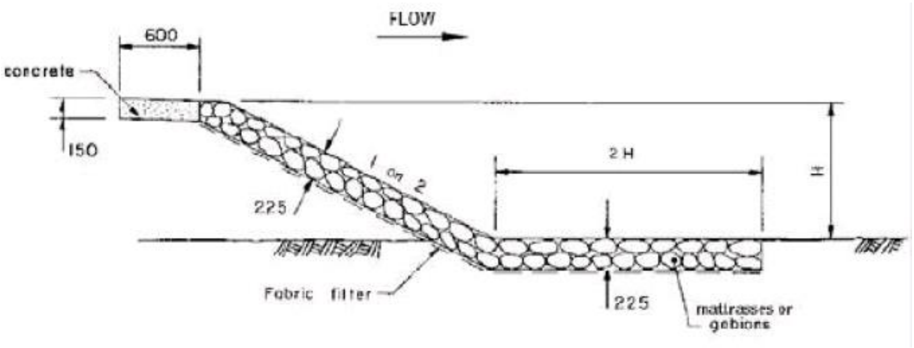

Figure 3.15, Rock mattress protection (Department of Transport and Main Roads)

Vegetation Cover

Vegetative cover can be used on top of a primary erosion protection system. This should only be used under low floodway velocities and low embankment, within humid regions.

Other Flexible Protection Methods

Proprietary products are also commonly used as a measure of erosion protection. These often have varied application and installation which requires the referral of the manufacturer's technical manual. For example, a flexible mat is a geotextile loop matting casted by small concrete blocks, which provides protection by overlapping. Another example is flexible pump-up revetment mattresses, which are nylon mattresses filled by concrete.

Although more susceptible to erosion at toe of batters, rigid protections are also used to prevent scour at floodways:

Grouted Rock

Grouted rock is used when small stone is the only resource readily available locally or where a low depth of protection is required. It involves filling the void of the dumped rock layer with concrete over the full depth.

Concrete Slab Protection

Illustrated in Figure 3.16, concrete slab protection involves pouring plain or reinforced concrete on the intended surface. This protection type is used in high velocity conditions, and due to its high cost is only used when other types of protection are inappropriate.

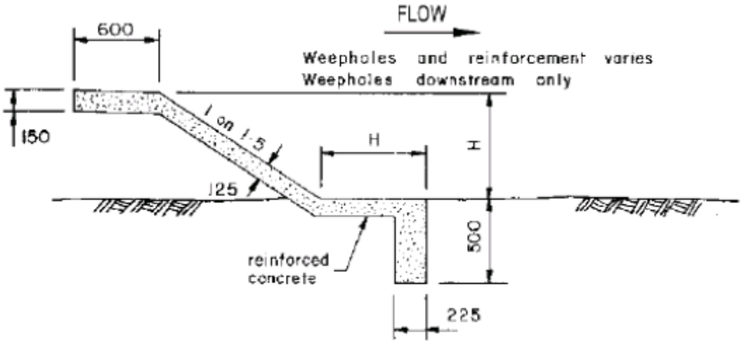

Figure 3.16, Concrete slab protection (Department of Transport and Main Roads)

3.3.4 Scour Prevention in Other Road Structures

In additional to the available protection methods used on floodways, there are still potential improvements that can be implemented from other road structures. Scour countermeasures to culverts and levees can all be considered. Riprap, in particular, is extremely cost effective when combined with other methods.

Culverts: Riprap and Adjustment to Entrance Contraction

In an experiment involving the scour prevention of bottomless arch culverts, four riprap stone sizing were tested (Crookston et al. 2012). These stones include:

- 7mm gravel
- 16mm angular gravel
- 35mm cobbles
- 37mm angular rock

Figure 3.17, Bottomless arch culvert entrance conditions (Crookston et al., 2012)

Additionally, the entrance inlet traction ratio is also adjusted when the contraction percentages of 0% (A), 33% (B) and 75% (C) as shown in Figure 3.17. Out of the four rock types, the 16mm angular gravel performed the best due to its sufficient size and ability to resist movement. The rock also produced smaller local scour holes. However, it should be noted that angular rocks are more costly than rounded streambed materials. The contraction of the culvert entrance also decreased the scour depth, causing the phenomena of decreasing the flow velocity when scour occurs until the bed material is stable.

Levee: Riprap and Gravel Underlayer

Furthermore, riprap is also used frequently in levee scour protection. When riprap is accompanied with a gravel underlayer (Figure 3.18), its prevention ability jumped significantly. The scour depth and length reduction percentage of riprap by itself is 32% and 23.9%, respectively. However, after adding an underlayer, the reduction percentage increased respectively to 88% and 83% (Johnson et al. 2013). Sediment removal was avoided with the presence of an underlayer as the small pores reduce the flow interaction with the soil.

Figure 3.18, Gravel underlayer on the levee (Johnson et al., 2013)

3.3.5 Other Scour Prevention Methods

Geosynthetics

Geosynthetics control soil erosion by separating soil and water completely. This is achieved by its properties in drainage, durability, flexibility and strength (Heibaum 2014). In particular, the impervious geosynthetic material directs surface water flow, where the overflow is mitigated allowing scour mitigation. It is recommended by Heibaum (2014) that the geosynthetic solution can be applied on waterways and flood protection structures as a scour countermeasure. Additionally, geomembranes are used to decrease runoff in the rain retention basins and are more affordable. The geosynthetic clay liner (GCL) is impervious linings found in irrigation and retention ponds, canals and dikes.

Natural Polymer Derivatives (NPD)

A study conducted by Liu et al. (2014) suggests that macromolecular polymers can improve soil structure. Specifically, NPD can be used to prevent sheet erosion on hillslopes, where the higher the concentration of NPD, the lower the cumulative erosion modulus. A concentration of $5 g/m^2$ decreased the cumulative erosion modulus by 56-61%. However, it is unclear whether NPD is effective during intensive rainfall and other types of erosion.

Geotextile Tube Technology

Geotextile tubes are used in coastal erosion protection, flood control and environmental applications (Shin et al. 2007). The tubes are filled with dredged materials and are staked on dikes and levees. From laboratory experiments, the wave height is decreased when geotextile tubes are present. Geotextile tubes are an economic solution and require minimal construction time. Moreover, the tubes are also environmental friendly and coexist with the marine life.

3.4 Conclusion

From the literature review, the available scour mitigation methods are:

1. Bridges:
    - Riprap
    - Bridge widening
    - Wing walls
    - Spill-through abutments

  - Combination of riprap and collaring
  - Flow-altering
  - Openings arrangement technique
  - Combination of cable and collar
  - Ring columns
2. Floodways:
   - Riprap (flexible)
   - Rock mattresses (flexible)
   - Vegetation cover (flexible)
   - Geotextile loop matting (flexible)
   - Flexible pump-up revetment mattresses (flexible)
   - Grouted rock (rigid)
   - Concrete slab protection (rigid)
3. Culverts:
   - Combination of riprap and adjustment to entrance contraction
4. Levee:
   - Combination of riprap and gravel underlayer
5. Others:
   - Geosynthetics
   - Natural Polymer Derivatives (NPD)
   - Geotextile tube technology

Floodways, culverts and bridges share similar failure mechanisms, where the three main causes of scouring are erosion, deposition and infrastructural failure. Therefore, these critical road structures may share scour countermeasures. Applications of riprap can be seen in culverts, bridges and floodways. This is due to its affordability and effectiveness. Riprap behaves similarly to a natural streambed, while providing resilience to soil erosion. Hence it is no surprise that riprap is already commonly used to prevent scour in bridges. Sourcing from other road structures, riprap is also used as a combination with another mitigation method. Appropriately, this can be applied on bridges to further decrease the likelihood of structural failure.

From Johnson et al. (2013), a gravel underlayer accompanied with riprap significantly increased the scour depth reduction percentage in the case of levee scour protection. This is due to the prevention of water-soil interaction from the underlayer’s small pores. Similarly, this combination of riprap and gravel can be applied to bridges.

Geosynthetics is also an appropriate selection for scour mitigation. It minimises water-soil interaction while still allowing drainage. As well as durability, geosynthetics is also high in tensile strength and sustainable. Additionally, geotextile filters below an armour layer provides scour resistance if the armour is scoured. This makes geotextile an excellent combination with riprap.

Bridge widening may increase the flow area, which means that contraction scour is minimised. As a result, the abutment scour and pier scour may be decreased, consequently decreasing the overall total scour depth.

The abutment structures, wing walls and spill-through abutments, may also be applicable in the HEC-RAS analysis. Both methods decrease abutment scour and have been proven effective.

Chapter 4 Methodology

In order to achieve the goals of this project, the following tasks will be completed:

1. Background information gathering
2. Data selection for software modelling
3. Result and feasibilit1y analysis

Details of the HEC-RAS inputs can be found in Appendix D.

4.1 Background Information Gathering

This research project involves a thorough electronic literature review, including relevant sources such as journal articles and reports. These findings will allow further understanding of the problem and the current methods of scour mitigation used.

The literature review will not be limited to scour mitigation methods of bridge, but also floodways and culverts as these systems behave similarly. Additionally, these resources will not be limited by their country of origin as the problem is universal. After thorough investigation, a number of methods will be chosen for further analysis.

From the literature review, the following can be used for bridge scour mitigation and in HEC-RAS analysis:

- Riprap with a gravel underlayer
- Widening of bridge (additional pier)
- Collaring
- Vertical abutment with wing walls
- Spill-through abutment

4.2 Data Selection for HEC-RAS Modelling

The chosen analysis software is HEC-RAS, which specialises in water flow calculation and simulations. To obtain the appropriate data for HEC-RAS modelling, a creek in the Lockyer Valley Region will be selected, as well as an existing bridge in the selected area. The data required for HEC-RAS modelling are:

- Bridge design and dimensions
- Creek cross-section
- Flood data of the Queensland floods in 2011 and 2013
- Mitigation method dimensions and information

The focus area of this research will be the Lockyer Valley Regional Council (LVRC) area. In this research, Tenthill Creek and the Tenthill Creek Bridge are chosen and the appropriate data selection will be made based on this location.

4.2.1 Bridge Design and Dimensions

In HEC-RAS, the bridge dimensions must be defined in order to conduct the analysis. The Tenthill Creek Bridge is built in 1976, located south of Gatton, Queensland, Australia. A photo of the bridge can be seen in Figure 4.1. It spans over the Tenthill Creek and used to carry traffic from Toowoomba to Ipswich. The Tenthill Creek Bridge is a simple span reinforced concrete bridge and is 82.15m long and 9m wide (Setunge 2002). The bridge is supported by two abutments and two piers. Its specific design details can be found in Appendix C. The overall dimensions of the bridge are summaries in Table 4.1.

Figure 4.1, Tenthill Creek Bridge (Setunge 2002)

Table 4.1, Tenthill Creek Bridge dimensions (Setunge 2002)

Bridge Dimensions (m)	
Overall Length	82.15
Pier Span	27.383
Deck Height	3.228
Deck Width	9
Pier Width	1.067

4.2.2 Creek Dimensions

Tenthill Creek is located in Gatton, highlighted in yellow in Figure 4.2. The creek spans 43.7km in length and is connected to Blackfellow Creek, Deep Gully and Wonga Creek (Digital Atlas 2016). The cross-section of the creek can be found in Table 4.2. Furthermore, the creek bed material consists of moist clay, hence having a particle size of less than 0.01mm (Powell et al. 2002). More details of the creek can be found in Appendix B.

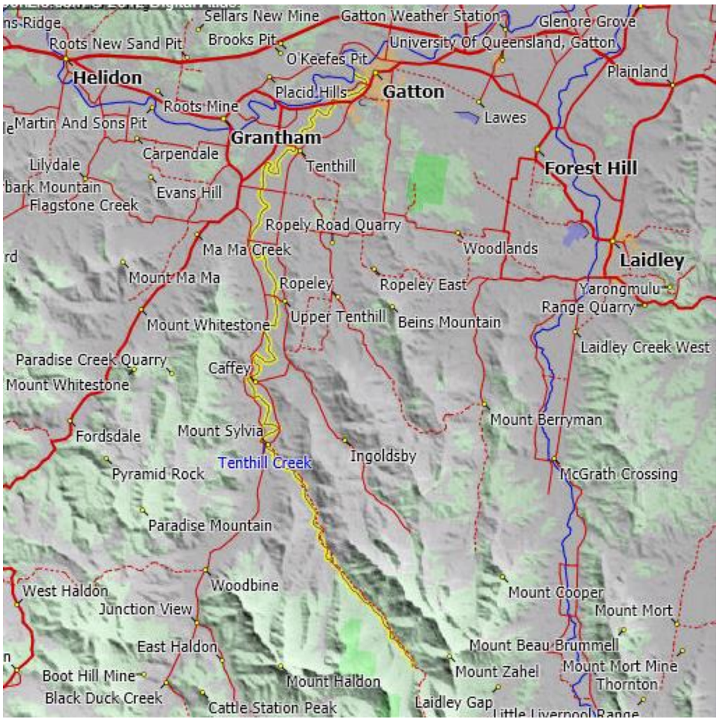

Figure 4.2, Tenthill Creek location (Digital Atlas 2016)

Table 4.2, Tenthill Creek cross-section (Water Monitoring Information Portal 2016)

Creek Data (m)	
Station	**Height**
0	12.45
32	8.95
56	7.1
73	5.05
85	4.75
97	0.4
112	0.4
118	1.4
147	8.95
156	11.95
200	11.95

4.2.3 Flood Data

To examine the effects of the dry season prior to the flood in 2013, both maximum discharges of the 2011 Queensland Floods and the 2013 Queensland Floods will be examined. The discharge data in Table 4.3 are obtained from the Water Monitoring Information Portal (WMIP). The maximum discharge of the 2011 Queensland Flood occurred on 27th December 2010, whereas the maximum discharge of the 2013 Queensland Floods occurred on 28th January 2013.

Table 4.3, Discharge data of the 2011 and 2013 Queensland Floods (Water Monitoring Information Portal 2016)

Discharge (Cumecs)			
2011		**2013**	
Average	Maximum	Average	Maximum
37.93	1176.46	26.77	1359.36

4.2.4 Mitigation Method Dimensions

Riprap with a gravel underlayer is chosen as one of the scour mitigation methods. It is therefore a requirement to define its Manning’s n value in HEC-RAS to incorporate the roughness change in the streambed. The full list of Manning’s values can be found in Appendix F. The channel in 2011 matched the description of a clean winding channel with some weeds and stones (1-d). The Manning’s values chosen for 2013 takes into account the drier beds that experience weathering, hence a match with the descriptions of no vegetation and winding (4-b-1). Riprap with a gravel underlayer matches the description of an artificially constructed layer of riprap with a gravel layer (5-e-3), hence the chosen values. These Manning’s roughness n values are summarised in Table 4.4.

Table 4.4, Manning's value of the stream bed and riprap (Brunner 2016)

Manning's n Values			
	LOB	Channel	ROB
2011 (pre-weathering)	0.050	0.035	0.050
2013 (weathered)	0.030	0.023	0.030
Rip-rap	0.036	0.023	0.036

For the particle size of riprap, 35mm cobblestones are chosen, as it is more likely to be locally available and were the second most effective scour preventer in the experiments conducted by Crookston et al. (2012). For the dimensions of a widened bridge, an extra span of 27.383 and an extra pier is to be added. The length of the bridge is also increased to 109.533m. Furthermore, the use of alternative abutment designs changes the K_2 value in the Froehlich's formula. The K_2 value of vertical abutments, vertical abutment with wing walls and spill-through abutments are 1.00, 0.82 and 0.55, respectively.

These values will fulfil the design parameters required for the HEC-RAS analysis.

4.3 Tabulate and Result Analysis

With the HEC-RAS modelling completed, results will be tabulated and compared. Furthermore, a feasibility analysis will be conducted. The final chosen solution, limitations and recommendations will then be outlined.

Chapter 5 HEC-RAS Modelling and Results

5.1 General

A HEC-RAS model was generated based on the dimensions of the Tenthill Creek Bridge. The flood loads of both the 2011 Queensland Floods and the 2013 Queensland Floods were taken into account. Numerous scour mitigation methods and possible combinations were applied. After all of the inputs are complete, a steady flow analysis is conducted and the hydraulic design bridge scour function is triggered on HEC-RAS. The results of the analysis are then generated. For more detailed results, see Appendix E.

5.2 HEC-RAS Results

5.2.1 2011 Queensland Floods

For the analysis of the 2011 Queensland floods, the maximum discharge used is 1176.46 m^3/s. The results of the HEC-RAS analysis can be seen in Table 5.1 and the graphical comparison is shown in Figure 5.1.

Table 5.1, Scour depths of the 2011 Queensland Floods

	Scour Depth (m)		
	Abutment Scour Left	**Abutment Scour Right**	**Pier Scour**
Original	12.74	7.16	2.6
Riprap	10.66	4.24	1.16
Wing Wall	11.45	6.43	2.6
Spill	9.51	5.33	2.6
Bridge Widening	8.97	7.1	2.6
Riprap & Wing Wall	9.43	3.72	1.17

Riprap & Spill	7.59	2.95	1.17
Wing Wall & Widening	8.13	6.37	2.6
Spill & Widening	6.86	5.28	2.6
Riprap & Wing Wall & Widening	5.92	3.66	1.17
Riprap & Spill & Widening	4.81	2.9	1.17

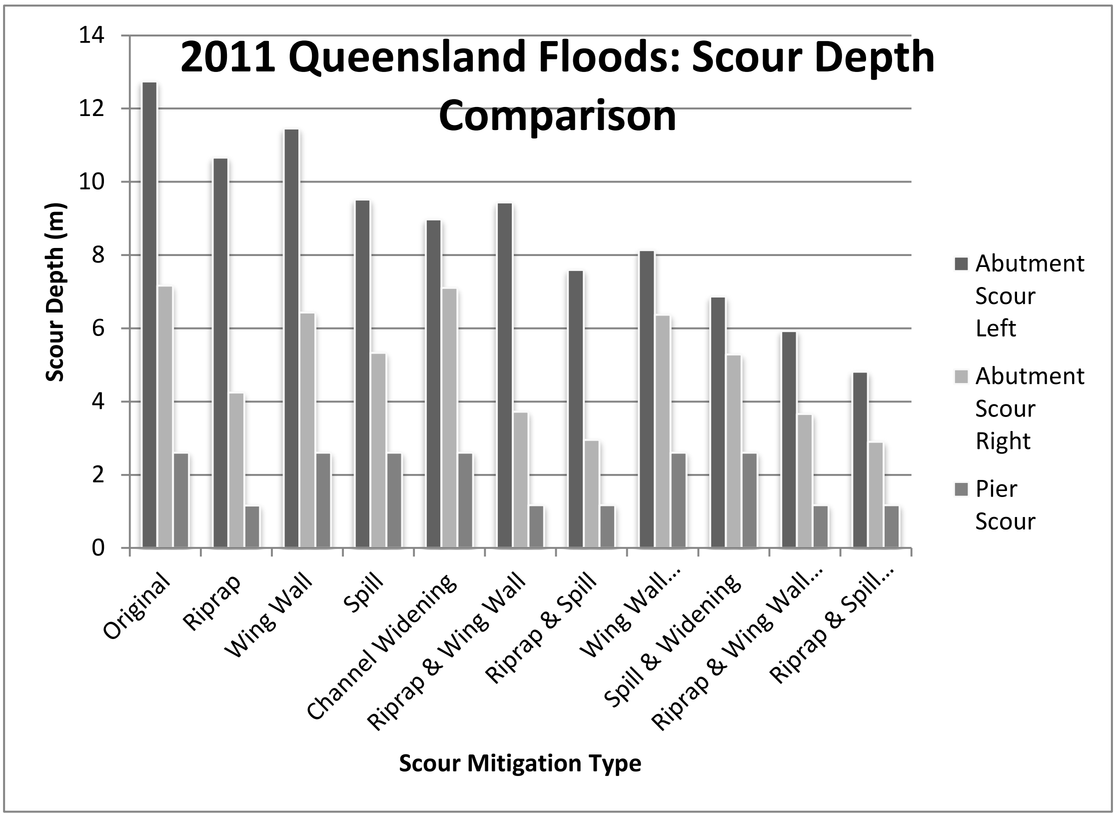

Figure 5.1, 2011 Queensland Floods: Scour Depth Comparison

5.2.2 2013 Queensland Floods

For the analysis of the 2011 Queensland floods, the maximum discharge used is 1359.36 m^3/s. The results of the HEC-RAS analysis can be seen in Table 5.2 and the graphical comparison is shown in Figure 5.2.

Table 5.2, Scour depths of the 2013 Queensland Floods

	Scour Depth (m)		
	Abutment Scour Left	**Abutment Scour Right**	**Pier Scour**
Original	13.53	7.46	2.73
Riprap	12.11	5.79	1.19
Wing Wall	12.07	6.65	2.73
Spill	9.88	5.43	2.73
Bridge Widening	9.4	7.38	2.74
Riprap & Wing Wall	10.76	5.13	1.19
Riprap & Spill	8.72	4.13	1.19
Wing Wall& Widening	8.45	6.57	2.74
Spill & Widening	7.02	5.36	2.74
Riprap & Wing Wall & Widening	7.16	5.07	1.19
Riprap & Spill & Widening	5.88	4.08	1.19

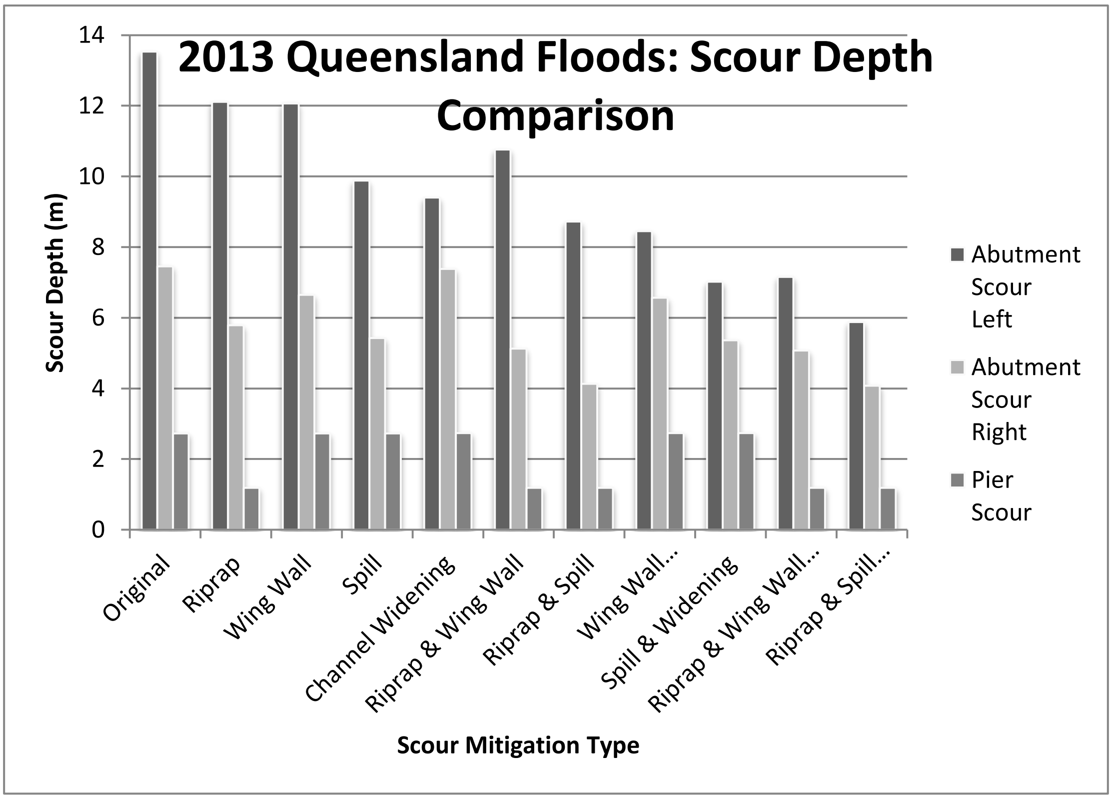

Figure 5.2, 2013 Queensland Floods: scour depth comparison

Chapter 6 Analysis of Results

The aim of the analysis is to seek the most effective scour mitigation method and also considering its cost effectiveness. The analysis will also take into account the combination of scour prevention methods.

6.1 Scour Mitigation Method Selection

In order to compare each scour mitigation method, the scour depth of the bridge without any protection is subtracted by the scour depth of each of the methods. The results are shown in Figure 6.1 and Figure 6.2, where the numerical details are shown in Table 6.1 and Table 6.2.

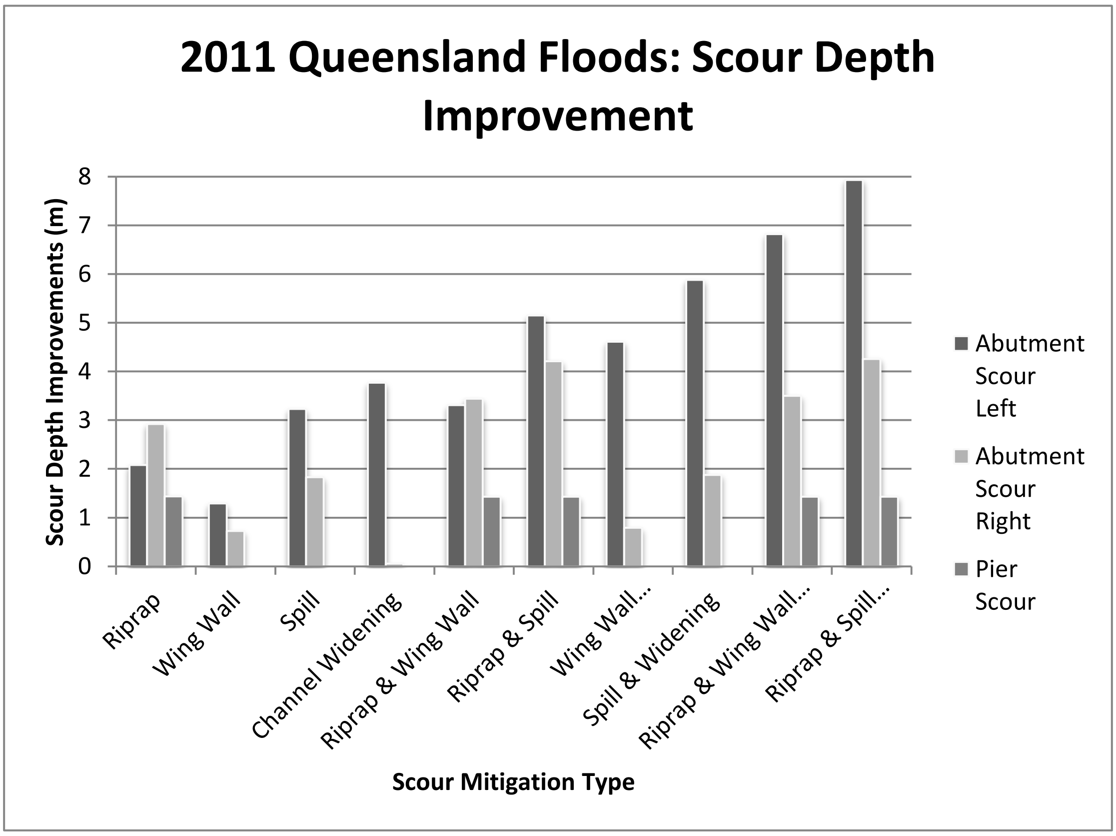

Figure 6.1, 2011 Queensland Floods: Scour Depth Improvement

Comparing the methods by its own and ignoring the combinations, riprap decreases the scour depths all abutment scour and pier scour. The wing wall and the spill-through abutment decreases both abutment scours on the left and right side, but does not influence the pier scour depth. Widening the channel under the bridge and adding an additional pier significantly decreases the abutment scour depth at the left of the channel, but does not influence other scour depths.

Table 6.1, Improvement of scour mitigation methods

2011			
	Scour Depth Improvement (m)		
	Abutment Scour Left	Abutment Scour Right	Pier Scour
Riprap	2.08	2.92	1.44
Wing Wall	1.29	0.73	0
Spill	3.23	1.83	0
Bridge Widening	3.77	0.06	0
Riprap & Wing Wall	3.31	3.44	1.43
Riprap & Spill	5.15	4.21	1.43
Wing Wall & Widening	4.61	0.79	0
Spill & Widening	5.88	1.88	0
Riprap & Wing Wall & Widening	6.82	3.5	1.43
Riprap & Spill & Widening	7.93	4.26	1.43

These results are as expected, as riprap is applied throughout the channel bed, whereas the wing wall and the spill-through abutment are primarily used to decrease abutment scour. In the HEC-RAS model, the widening of the channel is applied to the left of the channel hence the decrease in scour depth is only on the left hand side.

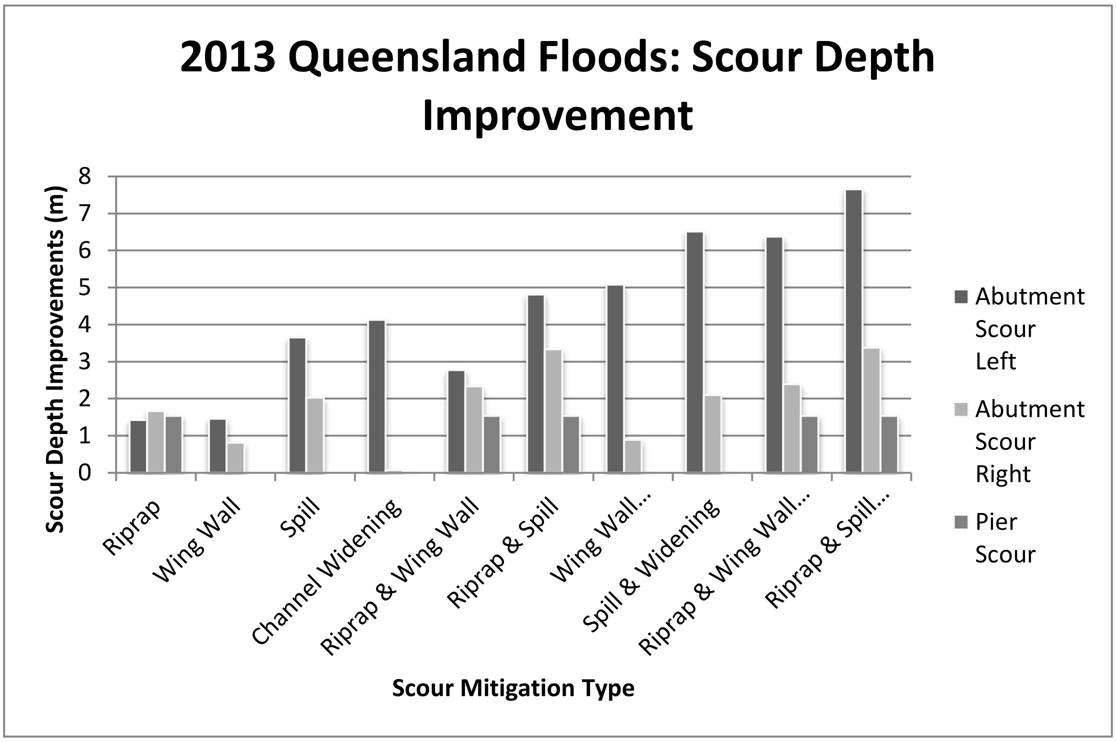

Figure 6.2, 2013 Queensland Floods: Scour Depth Improvement

Table 6.2, Scour depth improvement of mitigation methods

2013			
	Scour Depth Improvement (m)		
	Abutment Scour Left	Abutment Scour Right	Pier Scour
Riprap	1.42	1.67	1.54
Wing Wall	1.46	0.81	0
Spill	3.65	2.03	0
Bridge Widening	4.13	0.08	-0.01
Riprap & Wing Wall	2.77	2.33	1.54
Riprap & Spill	4.81	3.33	1.54
Wing Wall & Widening	5.08	0.89	-0.01
Spill & Widening	6.51	2.1	-0.01
Riprap & Wing Wall & Widening	6.37	2.39	1.54
Riprap & Spill & Widening	7.65	3.38	1.54

Comparing the combination of methods, combining riprap, spill-through abutments and widening the channel yield the highest decrease in scour depth. It is worth noting that most of the scour depth difference only occur at the left bank of the channel, whereas the pier scour depth and right bank abutment scour depth are generally unchanged.

6.2 Feasibility Analysis

The results show that a combination of widening the channel, adding riprap on top of a gravel layer and applying a spill-through abutment is the most effective scour countermeasure for bridges. Despite having the best result, the construction difficultly and construction and future maintenance costs must be considered.

Ettema et al. (2006) states that bridge widening is extremely costly and should be only used as a final resort. Additionally, spill-through abutments encounter frequent slope and geotechnical failures. Its construction is also difficult the abutment material is often not compacted properly. This means that spill-through abutments require a higher future maintenance cost and requires frequent monitoring. With these disadvantages in mind, the most effective scour countermeasure combinations may not be feasible economically and socially.

As bridge widening is the least cost effective, methods that combine this are removed from the method selection list. To compare the options, Figure 6.3 and Figure 6.4 are produced. From both figures, it is evident that the two competing combinations are riprap with wing walls and riprap with spill-through abutments. The difference of the left bank scour depth reduction between the two options is approximately two metres, whereas the difference between the right bank scour depth reductions is approximately one metre. Moreover, the pier scour depths of the two methods remain the same.

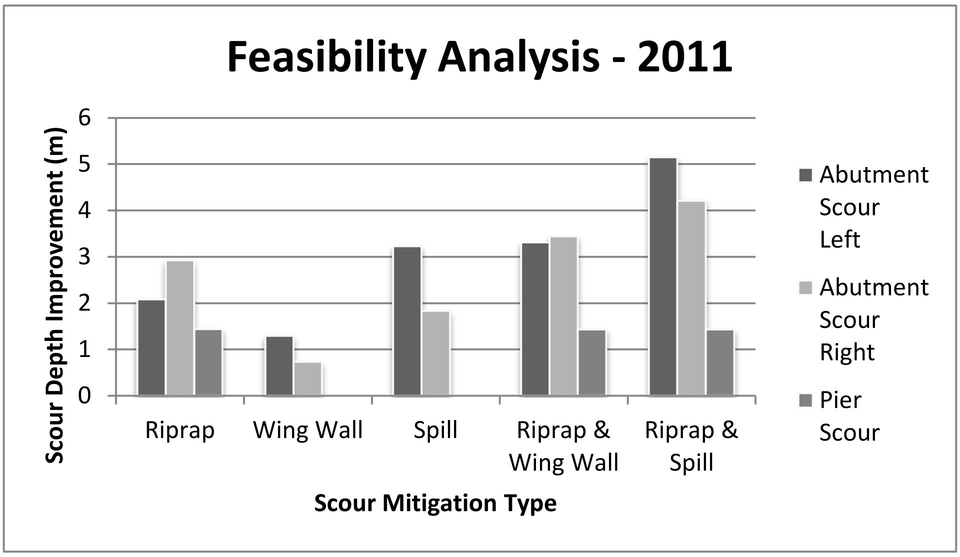

Figure 6.3, Feasibility analysis for the remaining options - 2011

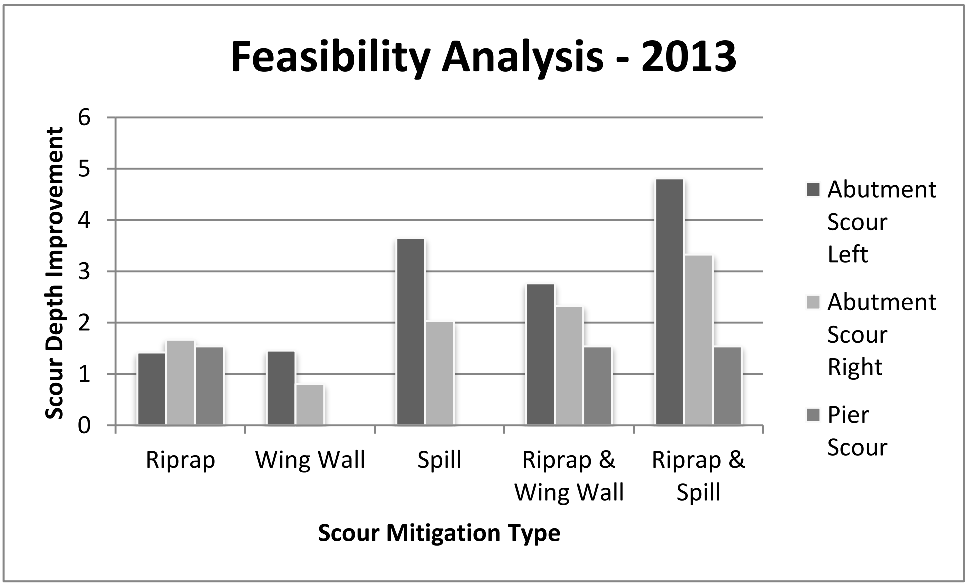

Figure 6.4, Feasibility analysis for the remaining options - 2013

Taking into account the possible geotechnical failure repair and construction costs of spill-through abutments, and the economic advantages of wing walls, the combination of riprap and wing walls ultimately is the most cost effective solution. Furthermore, riprap is adequate in preventing abutment scour when subjected to the average flow rate of the creek, where only pier scour is present (Figure 6.5).

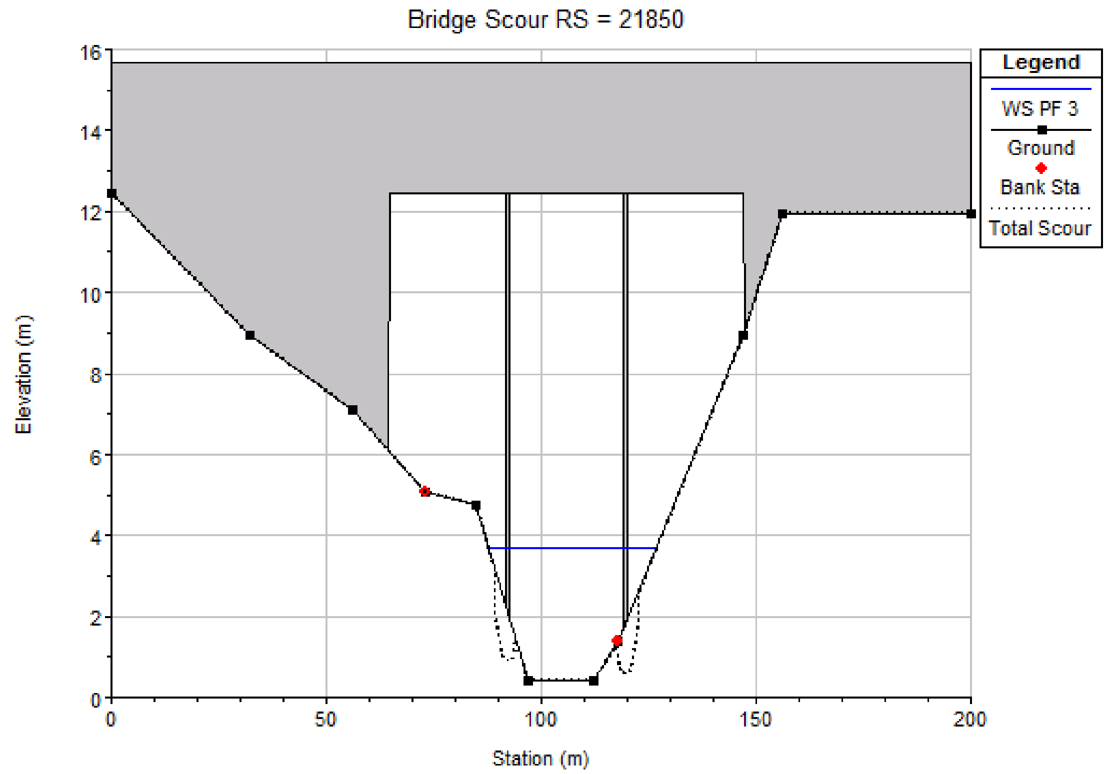

Figure 6.5, Pier scour of the bridge when protected by riprap and wing walls - 2011

To further counter pier scour during average flow, a collar is to be installed. According to Zarrati et al. (2010), applying a collar with a radius three times the pier radius reduces the pier scour depth by 57%. The reduction of scour depth is shown in Table 6.3.

Table 6.3, Effects of collaring

	Pier Scour Depth Under Average Flow (m)	Pier Scour Depth with Collar (m)
2011	1.21	0.52
2013	1.11	0.48

Therefore, the most cost effective and feasible scour reduction method is the combination of riprap, collaring and wing walls. This combination effectively reduces abutment scour and pier scour, while being easy to construct and maintain in the future.

6.3 Analysis Limitation

The HEC-RAS software carries limitations in its analysis, as most of its predictions are based on formulas much as the Froehlich's formula and the CSU equation. Additionally, when defining the design parameters of the bridge piers in HEC-RAS, collaring cannot be incorporated into the design. Therefore, using the laboratory results of Zarrati et al. (2010) is necessary to predict the effects of collaring. HEC-RAS also does not define the depths pf soil, riprap and gravel. This may cause inaccuracies in the model analysis. Furthermore, as mentioned frequently in the Queensland Bridge Scour Manual (Department of Transport and Main Roads 2013), most of the abutment and pier scour equations are based largely on laboratory results, and are rarely tested in practical environments. This means that the research results carries limitations and can only be used as an indicator of the effectiveness of the scour countermeasure method.

Chapter 7 Conclusion

7.1 Project Summary

A thorough literature review has been conducted on the scour mitigation methods of critical road structures. Firstly, the failure mechanism of critical road structures was examined. The bridge scour types are then identified to understand the background of the objective. Furthermore, methods of scour prevention for floodways, bridges, levees and culverts were compiled.

The Tenthill Creek Bridge at Gatton was chosen for the HEC-RAS analysis. The HEC-RAS model subjected to flood loads of the maximum discharge of the 2011 Queensland Floods and 2013 Queensland Floods. Additionally, the roughness of the channel and other dimensions were altered to simulate the effects of bridge scour countermeasures.

From the model, the combination of riprap, spill-through abutments and bridge widening proves to be the most effective in decreasing abutment and pier scour. However, spill-through abutments are prone to geotechnical failure at slopes while bridge widening is extremely costly. Ultimately, based on the feasibility analysis, the combination of riprap, wing walls and collaring at piers was chosen.

7.2 Achievement of Project Objectives

The following project objectives were accomplished:

- Available methods of scour mitigation of critical road structures have been found from journals and other resources
- Numerous bridge scour countermeasures were chosen
- The methods and combination of methods were analysed by HEC-RAS

- An appropriate method was chosen and was found feasible for practical implementation

7.3 Recommendation for Further Work

As HEC-RAS is only limited to hydraulic data, a finite element analysis may be conducted, subjecting the model to traffic and debris loads. Additionally, unconventional methods of scour mitigation may be examined. In addition to the Tenthill Creek Bridge, other bridges and creeks can be chosen for case studies, increasing the reliability of the HEC-RAS analysis. Lastly, practical experiments in wave tanks can be conducted using the chosen materials to further testify the method’s feasibility.

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APPENDICES

Appendix A – Project Specification

ENG 4111/4112 Research Project

PROJECT SPECIFICATION

For:	Peggy Chou Pei-Chen
Title:	Investigation of scour mitigation methods for critical road structures
Supervisor:	Dr Weena Lokuge Dr Buddhi Wahalathantri
Project Aim:	To investigate different ways to reduce or minimise soil erosion on critical road structures
Enrolment:	ENG4111 – ONC S1, 2016 ENG4112 – ONC S2, 2016
Programme:	**Issue C, 1st October 2016**

1. Research the available methods used around the world to reduce scour in critical road structures
2. Investigate the possibility of adopting one or multiple scour prevention methods for bridge scour mitigation.
3. Identify additional critical parameters required for analysis from additional literature, such as the Water Monitoring Information Portal.
4. Investigate the effects of scour protection methods by conducting a Hydraulic analysis using HEC-RAS. Complete the different mitigation methods.

If time and resources permit:

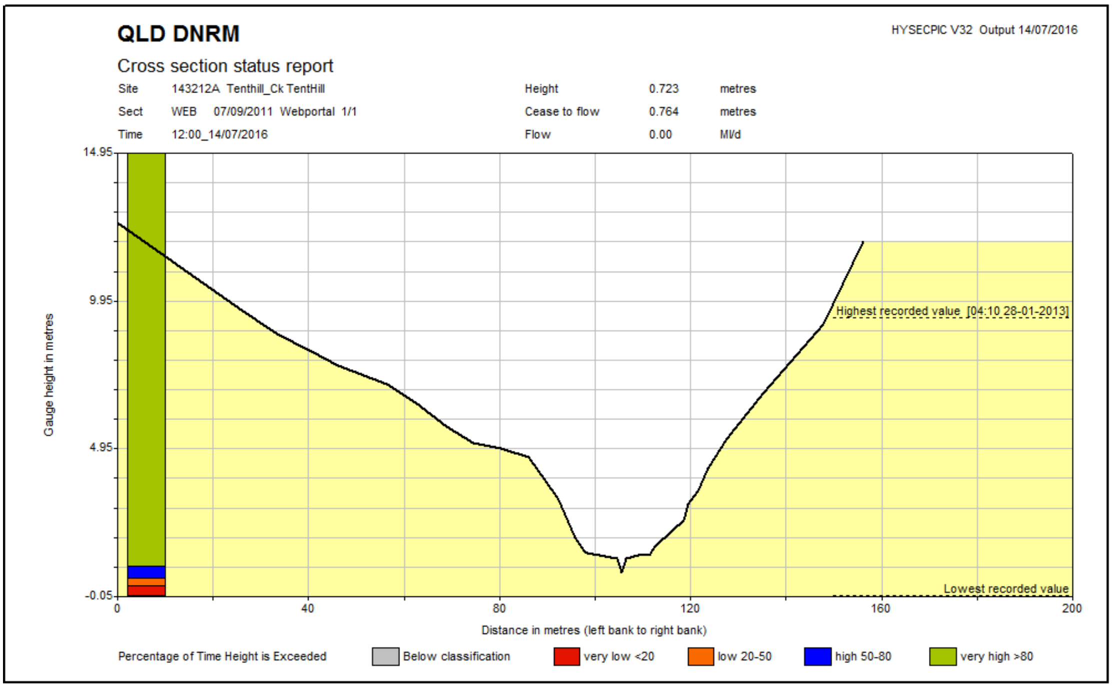
5. Conduct finite analysis of the chosen method using Stand 7.

AGREED________________(Student) ___________________ (Supervisor)

DATE: / /2016 DATE: / /2016

Examiner / Co-examiner_________________

Appendix B – Tenthill Creek Flood Data and Information



QLD DNRM

Cross section status report

Site 143212A Tenthill_Ck TentHill

Sect WEB 07/09/2011 Webportal 1/1

Time 12:00_14/07/2016

Height 0.723 metres

Cease to flow 0.764 metres

Flow 0.00 Ml/d

Time and Date	143212A		Time and Date	143212A	
	140			140	
	Discharge (Cumecs)			Discharge (Cumecs)	
	Max	Qual		Max	Qual
27/01/2013 0:00	42.202	30	27/12/2010 0:00	182.121	30
27/01/2013 1:00	46.256	30	27/12/2010 1:00	176.983	30
27/01/2013 2:00	50.17	30	27/12/2010 2:00	163.238	30
27/01/2013 3:00	60.967	30	27/12/2010 3:00	139.928	30
27/01/2013 4:00	77.831	30	27/12/2010 4:00	138.834	30
27/01/2013 5:00	89.046	30	27/12/2010 5:00	146.901	30
27/01/2013 6:00	122.23	30	27/12/2010 6:00	150.923	30
27/01/2013 7:00	149.99	30	27/12/2010 7:00	150.923	30
27/01/2013 8:00	150.612	30	27/12/2010 8:00	146.594	30

27/01/2013 9:00	169.924	30	27/12/2010 9:00	137.942	30
27/01/2013 10:00	184.314	30	27/12/2010 10:00	134.502	30
27/01/2013 11:00	205.065	30	27/12/2010 11:00	132.751	30
27/01/2013 12:00	260.519	60	27/12/2010 12:00	141.729	30
27/01/2013 13:00	345.668	60	27/12/2010 13:00	229.471	60
27/01/2013 14:00	452.572	60	27/12/2010 14:00	620.096	60
27/01/2013 15:00	538.833	60	27/12/2010 15:00	1072.938	60
27/01/2013 16:00	590.413	60	27/12/2010 16:00	1176.461	60
27/01/2013 17:00	629.177	60	27/12/2010 17:00	1129.676	60
27/01/2013 18:00	673.525	60	27/12/2010 18:00	834.096	60
27/01/2013 19:00	710.472	60	27/12/2010 19:00	565.81	60
27/01/2013 20:00	769.675	60	27/12/2010 20:00	388.213	60
27/01/2013 21:00	828.797	60	27/12/2010 21:00	276.044	60
27/01/2013 22:00	888.641	60	27/12/2010 22:00	224.039	60
27/01/2013 23:00	942.502	60	27/12/2010 23:00	190.507	30
28/01/2013 0:00	1058.396	60	28/12/2010 0:00	165.414	30
28/01/2013 1:00	1166.258	60	28/12/2010 1:00	151.027	30
28/01/2013 2:00	1308.566	60	28/12/2010 2:00	137.25	30
28/01/2013 3:00	1351.855	60	28/12/2010 3:00	124.653	30
28/01/2013 4:00	1359.358	60	28/12/2010 4:00	115.748	30
28/01/2013 5:00	1307.605	60	28/12/2010 5:00	107.891	30
28/01/2013 6:00	1296.103	60	28/12/2010 6:00	101.335	30
28/01/2013 7:00	1185.427	60	28/12/2010 7:00	95.245	30
28/01/2013 8:00	1079.029	60	28/12/2010 8:00	89.906	30
28/01/2013 9:00	862.486	60	28/12/2010 9:00	85.496	30
28/01/2013 10:00	760.051	60	28/12/2010 10:00	81.801	30
28/01/2013 11:00	706.826	60	28/12/2010 11:00	78.049	30

28/01/2013 12:00	614.231	60	28/12/2010 12:00	74.322	30
28/01/2013 13:00	498.729	60	28/12/2010 13:00	70.418	30
28/01/2013 14:00	411.198	60	28/12/2010 14:00	67.767	30
28/01/2013 15:00	316.432	60	28/12/2010 15:00	64.908	30
28/01/2013 16:00	277.107	60	28/12/2010 16:00	62.824	30
28/01/2013 17:00	269.414	60	28/12/2010 17:00	60.333	30
28/01/2013 18:00	248.062	60	28/12/2010 18:00	58.393	30
28/01/2013 19:00	250.066	60	28/12/2010 19:00	56.486	30
28/01/2013 20:00	276.803	60	28/12/2010 20:00	54.854	30
28/01/2013 21:00	288.022	60	28/12/2010 21:00	53.307	30
28/01/2013 22:00	292.078	60	28/12/2010 22:00	51.667	30
28/01/2013 23:00	278.478	60	28/12/2010 23:00	50.055	30

SOIL TYPE: ***Tenthill***
SITE NO: 55
MGA REFERENCE: 433 106 mE 6 951 937 mN ZONE 56

GREAT SOIL GROUP: Black earth
PRINCIPAL PROFILE FORM: Ug5.15
SOIL TAXONOMY UNIT: Chromustert
FAO UNESCO UNIT:
AUSTRALIAN SOIL CLASSIFICATION: MOTTLED,
SELF-MULCHING, BROWN, VERTOSOL. (Confidence level 1)

SUBSTRATE MATERIAL: unconsolidated material
CONFIDENCE SUBSTRATE IS PARENT MATERIAL:

SLOPE: 1 %
LANDFORM ELEMENT TYPE: prior stream
LANDFORM PATTERN TYPE: alluvial plain

VEGETATION
STRUCTURAL FORM: Isolated trees
DOMINANT SPECIES: *Eucalyptus tessellaris*
ANNUAL RAINFALL: 780 mm

PROFILE MORPHOLOGY:

CONDITION OF SURFACE SOIL WHEN DRY: self-mulching, periodic cracking

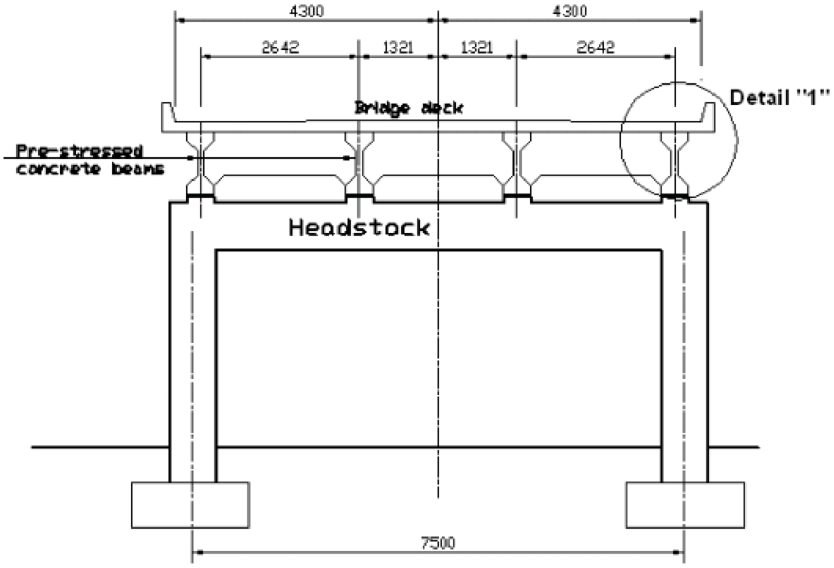
HORIZON	**DEPTH**	**DESCRIPTION**
AP	0 to .17 m	Brownish black (7.5YR2/2) moist; common coarse brown mottles; medium clay; massive clod; moist; moderately firm. clear to-
B	.17 to .60 m	Dark brown (7.5YR3/4) moist; common medium dark mottles; medium clay; strong 10-20mm lenticular largest peds, parting to moderate 10-20mm angular blocky; common macropores; moist; moderately weak. gradual to-
D1k	.60 to 1.40 m	Brown (7.5YR4/3) moist; fine sandy, clay; strong 20-50mm prismatic; common clay skin; common macropores; moist; moderately weak; few calcareous concretions. gradual to-
D2	1.40 to 1.50 m	Brown (7.5YR4/3) moist; fine sandy, clay; few medium pebbles, rounded basalt; strong 20-50mm prismatic; moist; moderately weak.

Depth	1:5 Soil/Water			Particle Size				Exch. Cations					Total Elements			Moistures			Disp.Ratio		Exch	Exch	ECEC	pH
metres	pH @ 40C	EC dS/m @ 40C	Cl % @105C	CS % @ 105C	FS	S	C	CEC m.eq/100g @ 105C	Ca	Mg	Na	K	P % @ 80C	K	S	ADM % @ 105C	33*	1500*	R1 @ 40C	R2	Al m.eq/100g @ 105C	Acid		CaCl2 @ 40C
0.10	7.4	.10	63	9	30	16	49	32	16	16	.94	.70	.111	0.91	.021	4.8			.53					
0.30	6.9	.09	86	9	24	13	57	46	19	17	1.4	.52	.083	0.88	.017	7.4			.54					
0.60	7.8	.10	75	10	32	14	48	44	22	18	1.7	.44	.097	0.75	.016	6.7			.50					
0.90	8.1	.08	74	6	55	13	28	44	22	18	1.8	.53	.119	0.85	.011	6.1			.47					
1.20	8.1	.12	115	12	51	14	26	40	21	17	1.7	.64	.112	0.88	.009	5.0								
1.50	8.0	.12	157	44	28	9	21	29	15	10	1.0	1.8	.116	0.75	.006	4.4			.74					

Depth	Org.C	Tot.N	Extr. P		HCl	CaCl2 Extr		DTPA-extr.					Extractable			P		Alternative Cations				
metres	(W&B) % @ 105C	% @ 105C	Acid mg/kg @ 105C	Bicarb.	K meq% @105C	K mg/kg @ 105C	P	Fe mg/kg @ 105C	Mn	Cu	Zn	B	SO4S mg/kg @ 105C	NO3N	NH4N	Buff Cap @ 40C	Equil ug/L	CEC m.eq/100g @ 105C	Ca	Mg	Na	K
0.10	1.2	.08	211	165	.64			49	9	1.5	1.4											
	1.1	.09	211	134	.49																	

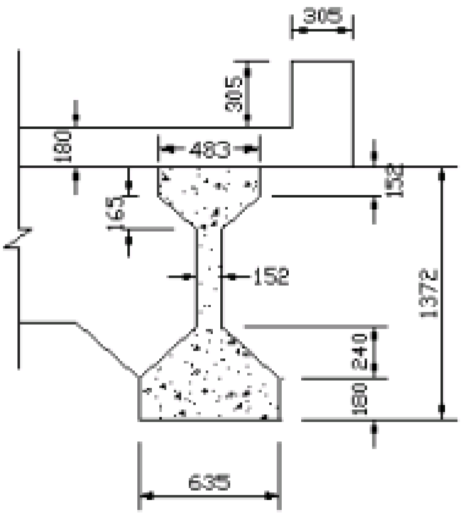
* -33kPa (-0.33bar) and -1500kPa (-15 bar) using pressure plate apparatus.
Cation method: .
CEC methods: .
Alternative cation method: .
Alternative CEC method: .
ECEC METHOD: .

Appendix C – Bridge Design



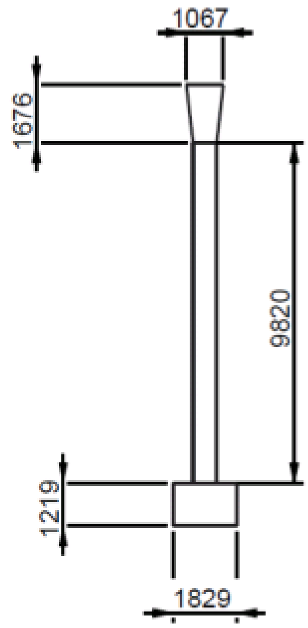
The section details of a typical pre-stressed concrete beam is shown in Figure 2-4

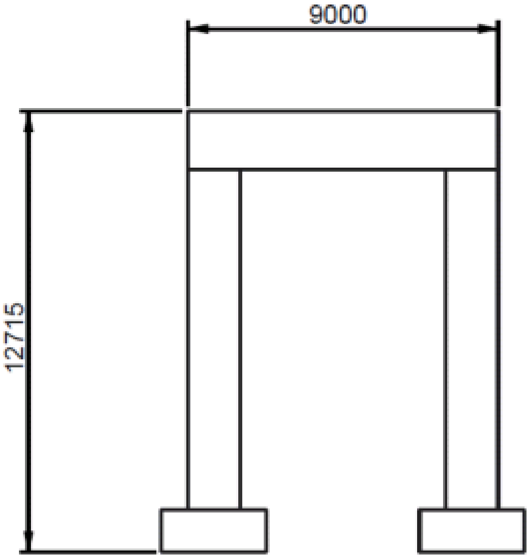
Figure 2-4: Section details of a longitudinal pre-stressed beam

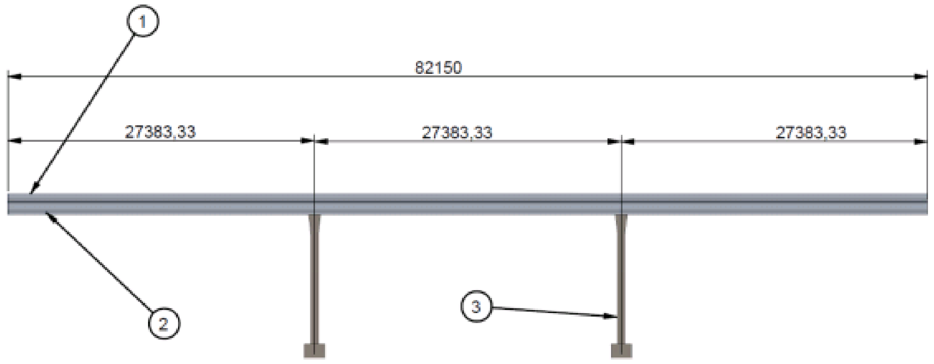


Beam span = 27382 mm.

Detail "1"





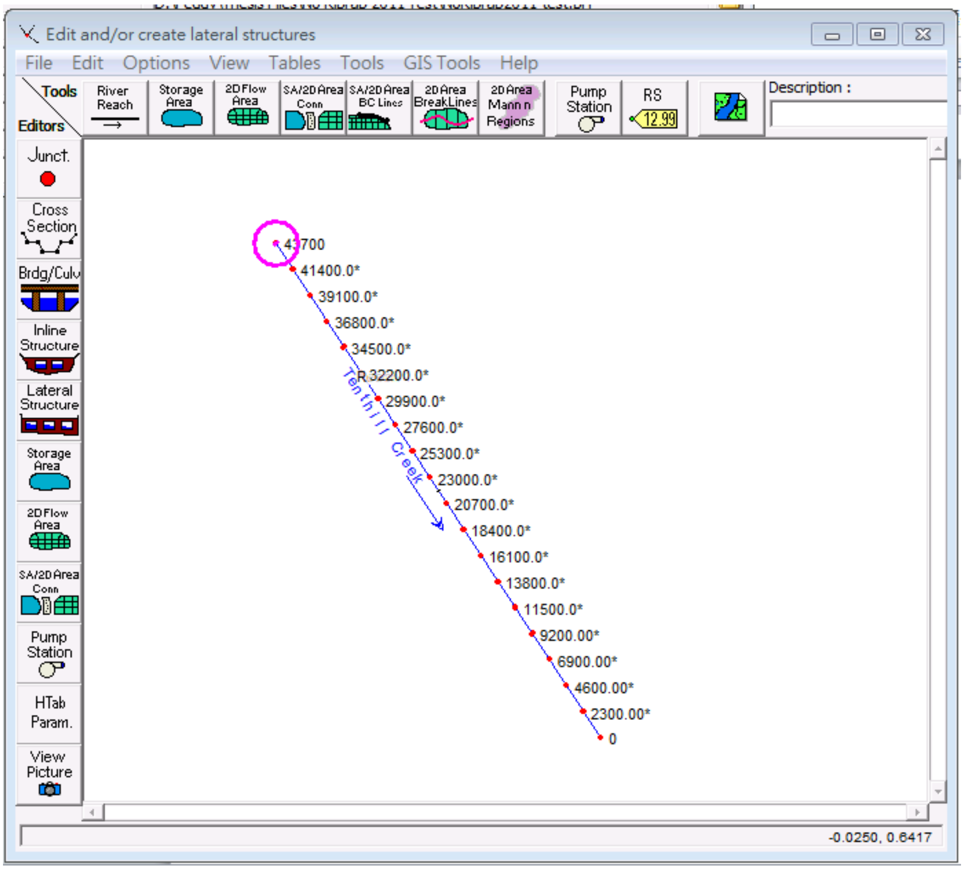


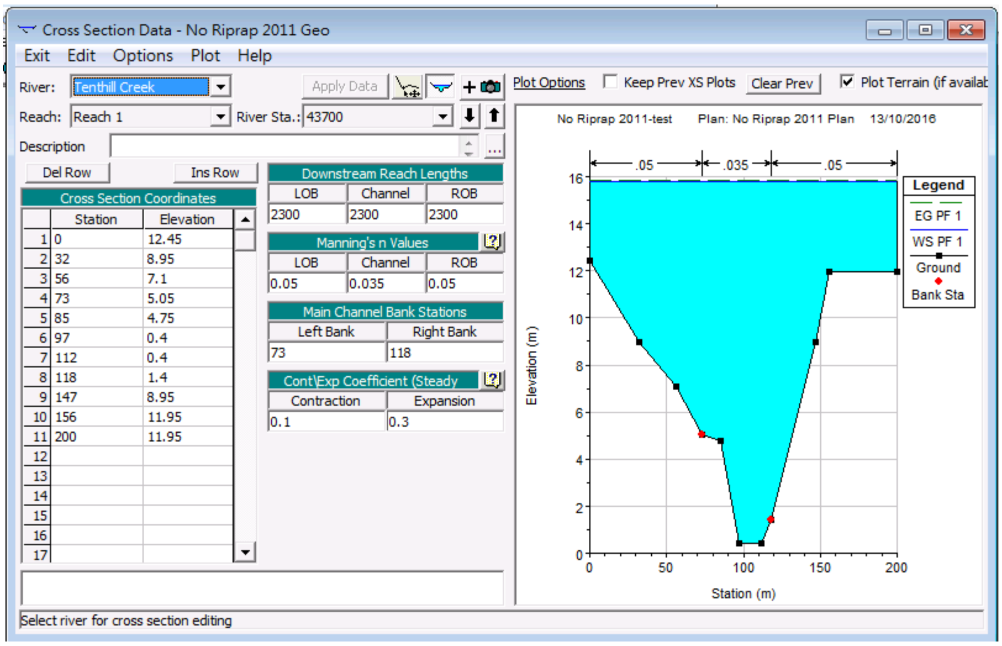
SCALE 0,003

1	Deck
2	Girder
3	Pier

Appendix D – HEC-RAS Input Data

Sample input files for 2011 Queensland Floods without protection





River: Tenthill Creek

Reach: Reach 1 River Sta.: 43700

Description

Cross Section Coordinates

	Station	Elevation
1	0	12.45
2	32	8.95
3	56	7.1
4	73	5.05
5	85	4.75
6	97	0.4
7	112	0.4
8	118	1.4
9	147	8.95
10	156	11.95
11	200	11.95
12		
13		
14		
15		
16		
17		

Downstream Reach Lengths

LOB	Channel	ROB
2300	2300	2300

Manning's n Values

LOB	Channel	ROB
0.05	0.035	0.05

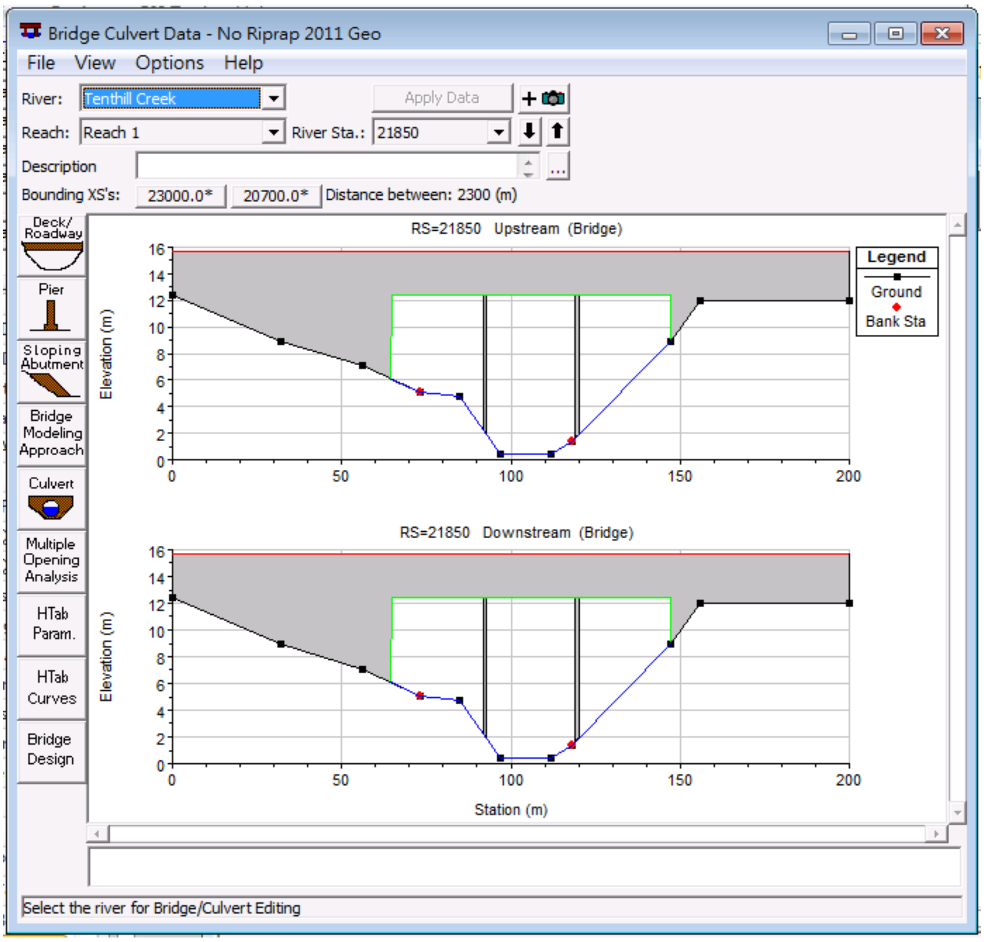
Main Channel Bank Stations

Left Bank	Right Bank
73	118

Cont\Exp Coefficient (Steady

Contraction	Expansion
0.1	0.3

Select river for cross section editing



Bridge Culvert Data - No Riprap 2011 Geo

River: Tenthill Creek

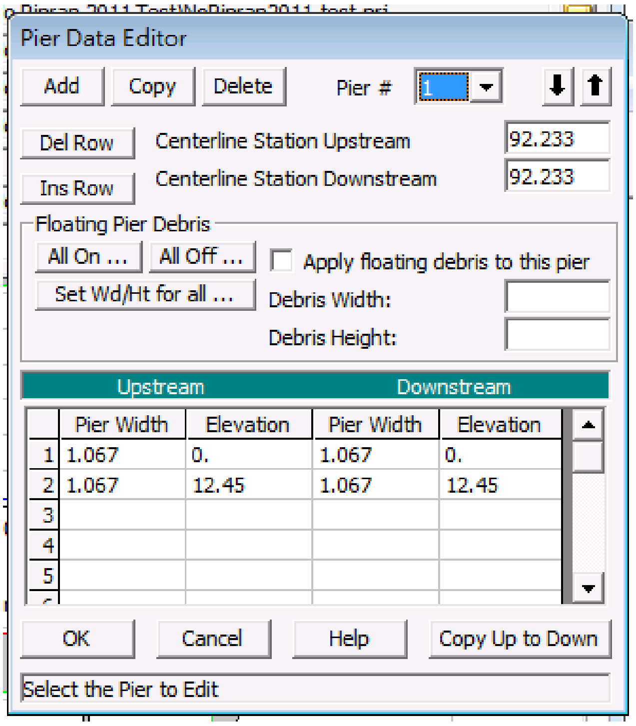
Reach: Reach 1

River Sta.: 21850

Bounding XS's: 23000.0* 20700.0*

Distance between: 2300 (m)

Select the river for Bridge/Culvert Editing



Pier Data Editor

Pier # 1

Centerline Station Upstream 92.233

Centerline Station Downstream 92.233

Floating Pier Debris

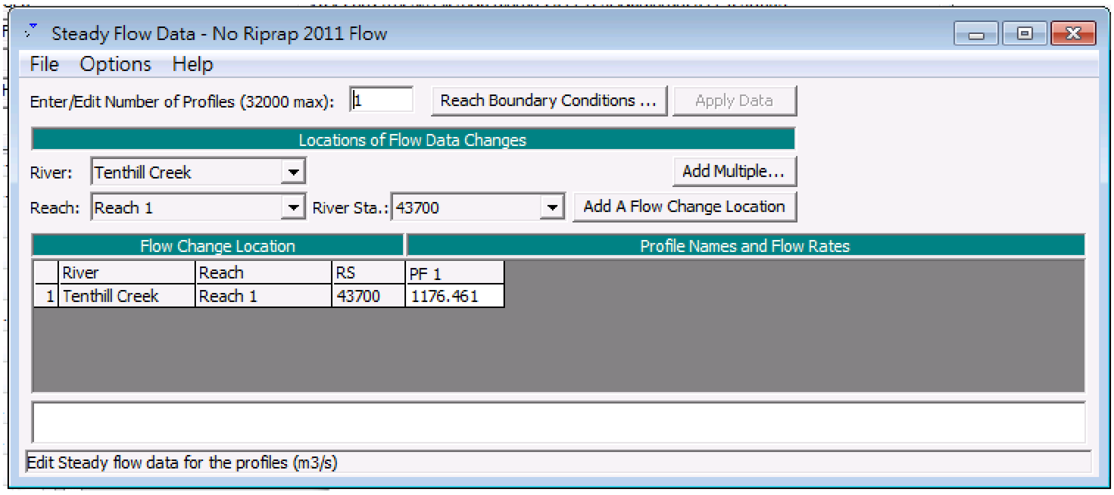
☐ Apply floating debris to this pier

Debris Width:

Debris Height:

	Upstream		Downstream	
	Pier Width	Elevation	Pier Width	Elevation
1	1.067	0.	1.067	0.
2	1.067	12.45	1.067	12.45
3				
4				
5				

Select the Pier to Edit



Enter/Edit Number of Profiles (32000 max): 1

River: Tenthill Creek

Reach: Reach 1

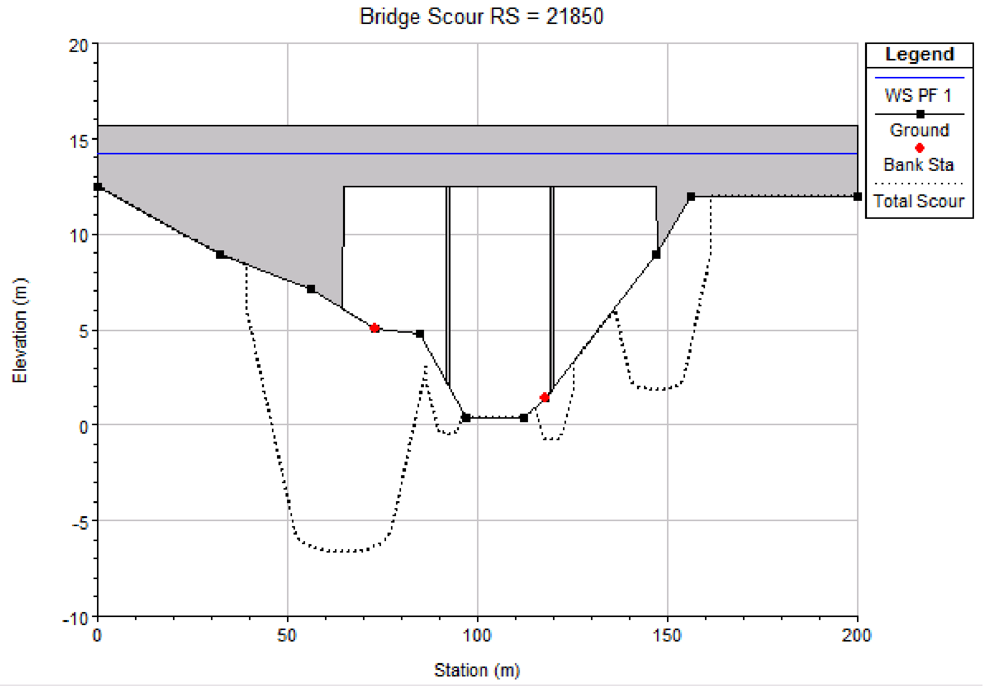
River Sta.: 43700

	River	Reach	RS	PF 1
1	Tenthill Creek	Reach 1	43700	1176.461

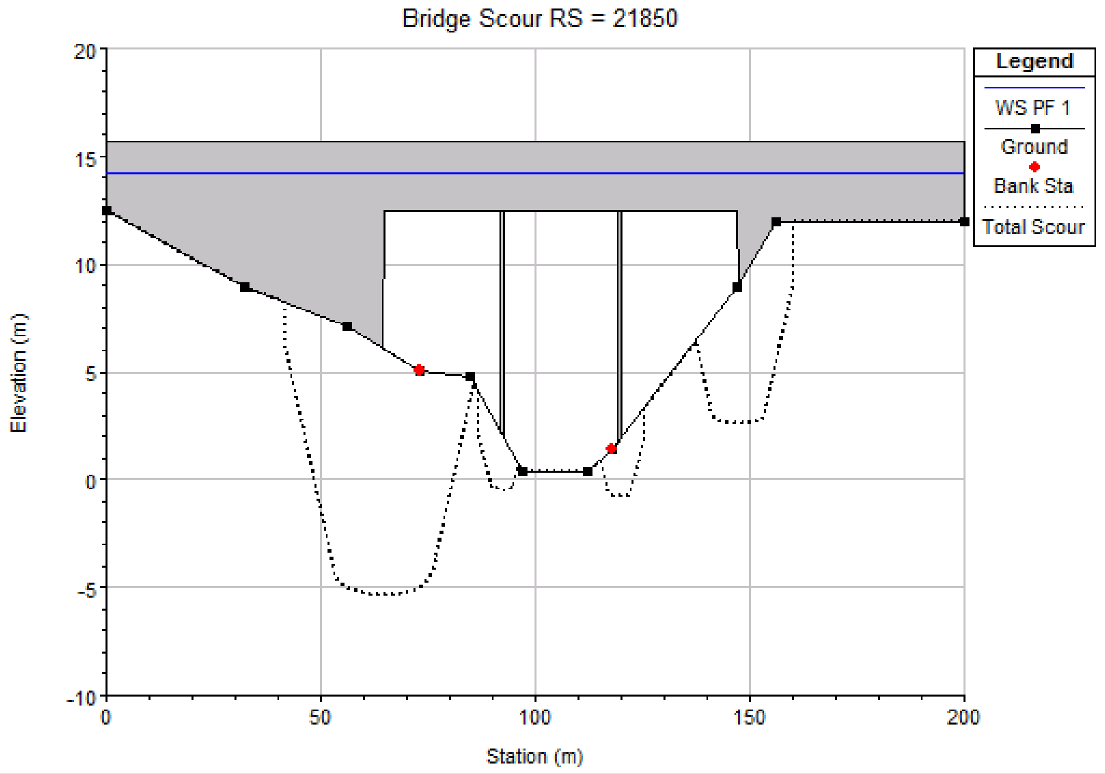
Edit Steady flow data for the profiles (m3/s)

Appendix E – HEC-RAS Complete Data

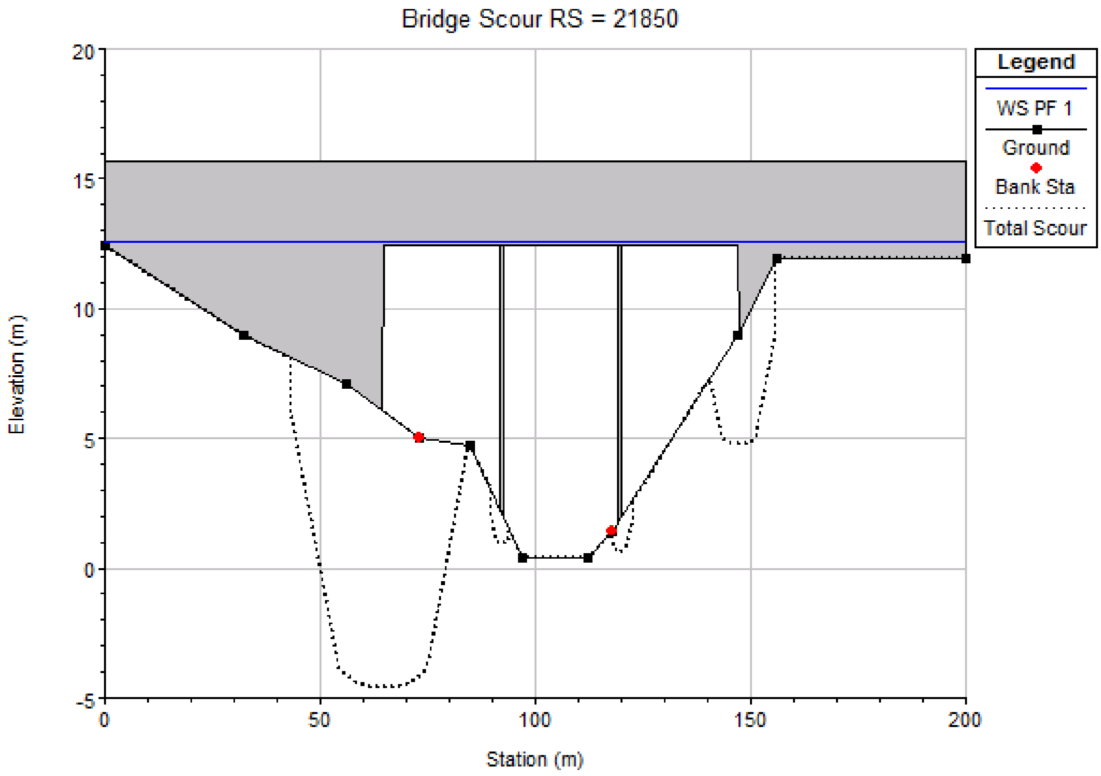
2011 without protection:



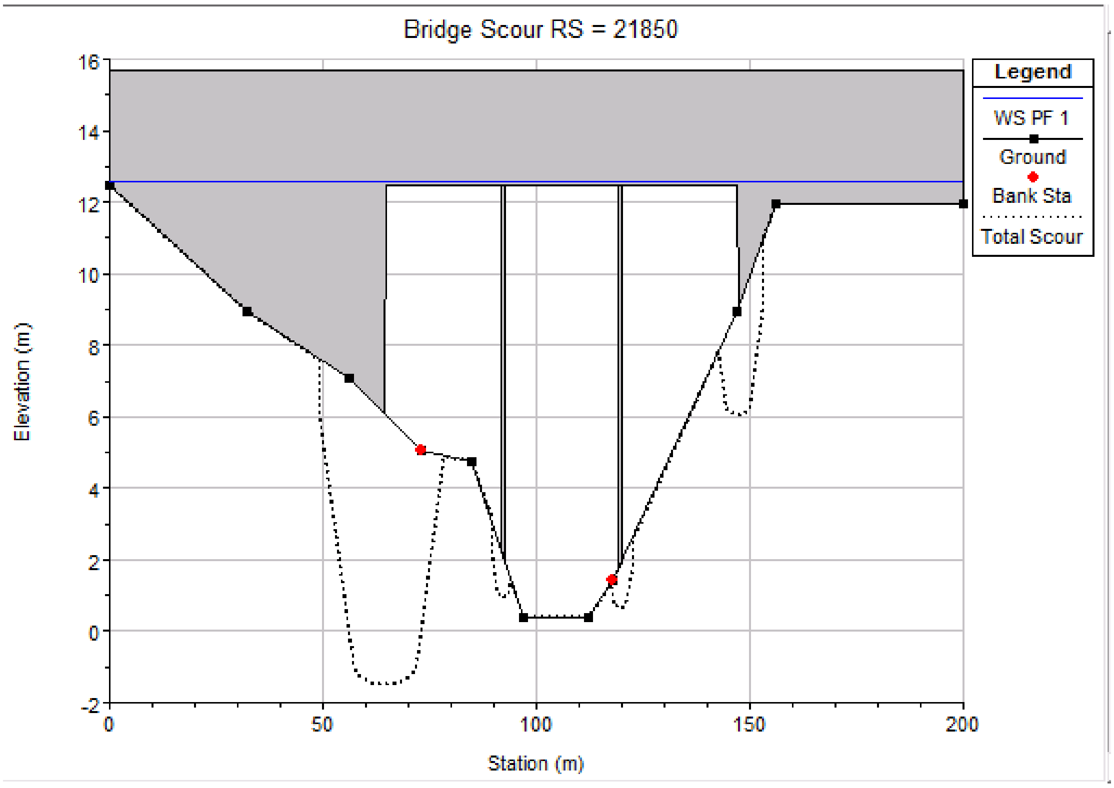
2011 with wing walls:



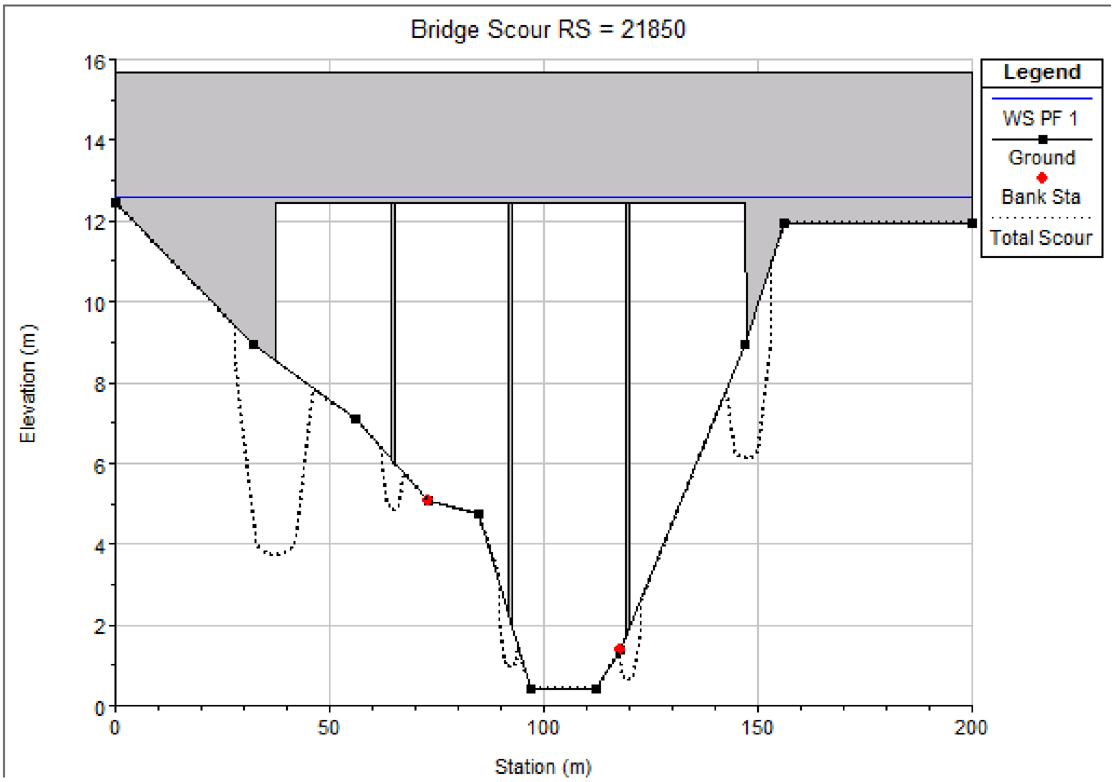
2011 with riprap

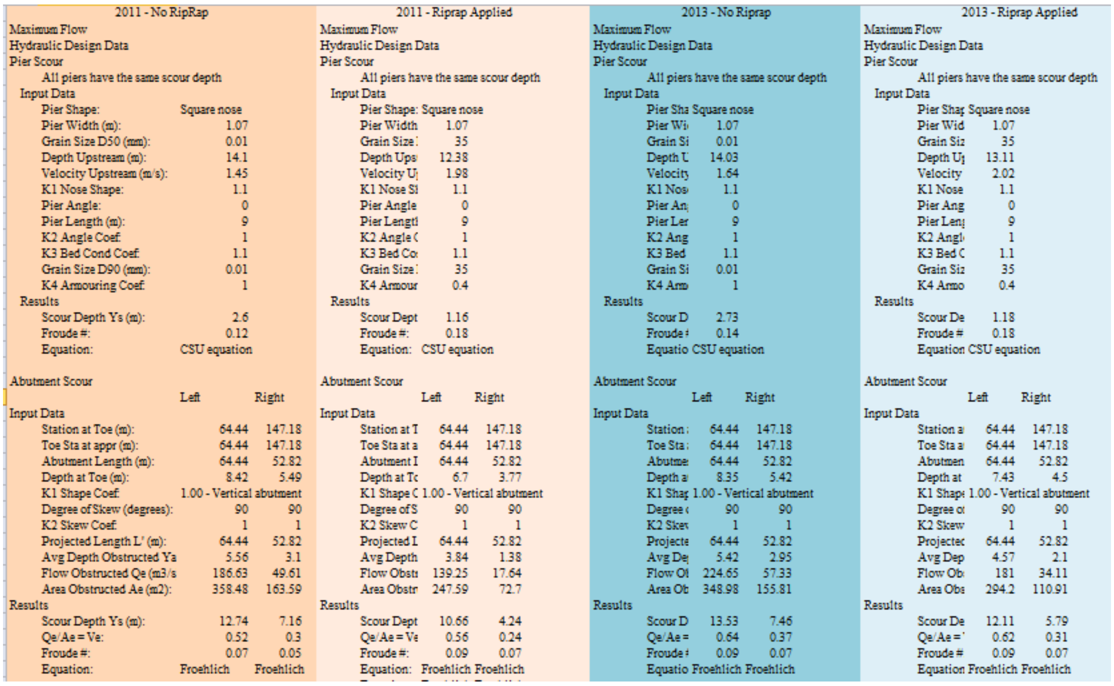


2011 with riprap, spill-through abutments:

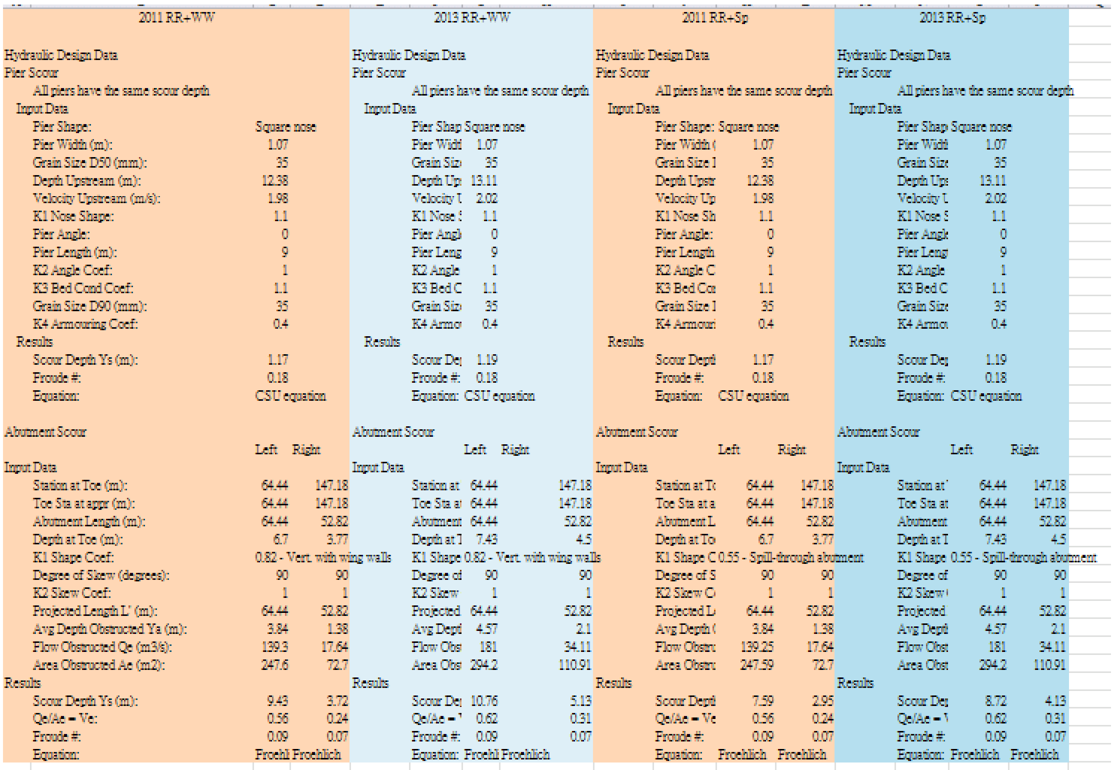


2011 with riprap, spill-through abutments and widened channel:





	2011 - No RipRap		2011 - Riprap Applied		2013 - No Riprap		2013 - Riprap Applied	
Maximum Flow								
Hydraulic Design Data								
Pier Scour								
All piers have the same scour depth								
Input Data								
Pier Shape:	Square nose		Square nose		Square nose		Square nose	
Pier Width (m):	1.07		1.07		1.07		1.07	
Grain Size D50 (mm):	0.01		35		0.01		35	
Depth Upstream (m):	14.1		12.38		14.03		13.11	
Velocity Upstream (m/s):	1.45		1.98		1.64		2.02	
K1 Nose Shape:	1.1		1.1		1.1		1.1	
Pier Angle:	0		0		0		0	
Pier Length (m):	9		9		9		9	
K2 Angle Coef.	1		1		1		1	
K3 Bed Cond Coef.	1.1		1.1		1.1		1.1	
Grain Size D90 (mm):	0.01		35		0.01		35	
K4 Armouring Coef.	1		0.4		1		0.4	
Results								
Scour Depth Ys (m):	2.6		1.16		2.73		1.18	
Froude #:	0.12		0.18		0.14		0.18	
Equation:	CSU equation		CSU equation		CSU equation		CSU equation	
Abutment Scour								
	Left	Right	Left	Right	Left	Right	Left	Right
Input Data								
Station at Toe (m):	64.44	147.18	64.44	147.18	64.44	147.18	64.44	147.18
Toe Sta at appr (m):	64.44	147.18	64.44	147.18	64.44	147.18	64.44	147.18
Abutment Length (m):	64.44	52.82	64.44	52.82	64.44	52.82	64.44	52.82
Depth at Toe (m):	8.42	5.49	6.7	3.77	8.35	5.42	7.43	4.5
K1 Shape Coef.	1.00 - Vertical abutment		1.00 - Vertical abutment		1.00 - Vertical abutment		1.00 - Vertical abutment	
Degree of Skew (degrees):	90	90	90	90	90	90	90	90
K2 Skew Coef.	1	1	1	1	1	1	1	1
Projected Length L' (m):	64.44	52.82	64.44	52.82	64.44	52.82	64.44	52.82
Avg Depth Obstructed Ya	5.56	3.1	3.84	1.38	5.42	2.95	4.57	2.1
Flow Obstructed Qe (m3/s	186.63	49.61	139.25	17.64	224.65	57.33	181	34.11
Area Obstructed Ae (m2):	358.48	163.59	247.59	72.7	348.98	155.81	294.2	110.91
Results								
Scour Depth Ys (m):	12.74	7.16	10.66	4.24	13.53	7.46	12.11	5.79
Qe/Ae = Ve:	0.52	0.3	0.56	0.24	0.64	0.37	0.62	0.31
Froude #:	0.07	0.05	0.09	0.07	0.09	0.07	0.09	0.07
Equation:	Froehlich	Froehlich	Froehlich	Froehlich	Froehlich	Froehlich	Froehlich	Froehlich



2011 RR+WW		
Hydraulic Design Data		
Pier Scour		
All piers have the same scour depth		
Input Data		
Pier Shape:	Square nose	
Pier Width (m):	1.07	
Grain Size D50 (mm):	35	
Depth Upstream (m):	12.38	
Velocity Upstream (m/s):	1.98	
K1 Nose Shape:	1.1	
Pier Angle:	0	
Pier Length (m):	9	
K2 Angle Coef:	1	
K3 Bed Cond Coef:	1.1	
Grain Size D90 (mm):	35	
K4 Armouring Coef:	0.4	
Results		
Scour Depth Ys (m):	1.17	
Froude #:	0.18	
Equation:	CSU equation	
Abutment Scour		
	Left	Right
Input Data		
Station at Toe (m):	64.44	147.18
Toe Sta at appr (m):	64.44	147.18
Abutment Length (m):	64.44	52.82
Depth at Toe (m):	6.7	3.77
K1 Shape Coef:	0.82 - Vert. with wing walls	
Degree of Skew (degrees):	90	90
K2 Skew Coef:	1	1
Projected Length L' (m):	64.44	52.82
Avg Depth Obstructed Ya (m):	3.84	1.38
Flow Obstructed Qe (m3/s):	139.3	17.64
Area Obstructed Ae (m2):	247.6	72.7
Results		
Scour Depth Ys (m):	9.43	3.72
Qe/Ae = Ve:	0.56	0.24
Froude #:	0.09	0.07
Equation:	Froehl	Froehlich

2013 RR+WW		
Hydraulic Design Data		
Pier Scour		
All piers have the same scour depth		
Input Data		
Pier Shap	Square nose	
Pier Widtl	1.07	
Grain Siz	35	
Depth Up:	13.11	
Velocity U	2.02	
K1 Nose S	1.1	
Pier Angl	0	
Pier Leng	9	
K2 Angle	1	
K3 Bed C	1.1	
Grain Siz	35	
K4 Armo	0.4	
Results		
Scour De	1.19	
Froude #:	0.18	
Equation:	CSU equation	
Abutment Scour		
	Left	Right
Input Data		
Station at	64.44	147.18
Toe Sta a	64.44	147.18
Abutment	64.44	52.82
Depth at T	7.43	4.5
K1 Shape	0.82 - Vert. with wing walls	
Degree of	90	90
K2 Skew	1	1
Projected	64.44	52.82
Avg Dept	4.57	2.1
Flow Obs	181	34.11
Area Obs	294.2	110.91
Results		
Scour De	10.76	5.13
Qe/Ae = V	0.62	0.31
Froude #:	0.09	0.07
Equation:	Froehl	Froehlich

2011 RR+Sp		
Hydraulic Design Data		
Pier Scour		
All piers have the same scour depth		
Input Data		
Pier Shape:	Square nose	
Pier Width (	1.07	
Grain Size I	35	
Depth Upstr	12.38	
Velocity Up	1.98	
K1 Nose Sh	1.1	
Pier Angle:	0	
Pier Length	9	
K2 Angle C	1	
K3 Bed Co	1.1	
Grain Size I	35	
K4 Armour	0.4	
Results		
Scour Dept	1.17	
Froude #:	0.18	
Equation:	CSU equation	
Abutment Scour		
	Left	Right
Input Data		
Station at To	64.44	147.18
Toe Sta at a	64.44	147.18
Abutment L	64.44	52.82
Depth at To	6.7	3.77
K1 Shape C	0.55 - Spill-through abutment	
Degree of S	90	90
K2 Skew C	1	1
Projected L	64.44	52.82
Avg Depth (	3.84	1.38
Flow Obstr	139.25	17.64
Area Obstr	247.59	72.7
Results		
Scour Dept	7.59	2.95
Qe/Ae = Ve	0.56	0.24
Froude #:	0.09	0.07
Equation:	Froehlich	Froehlich

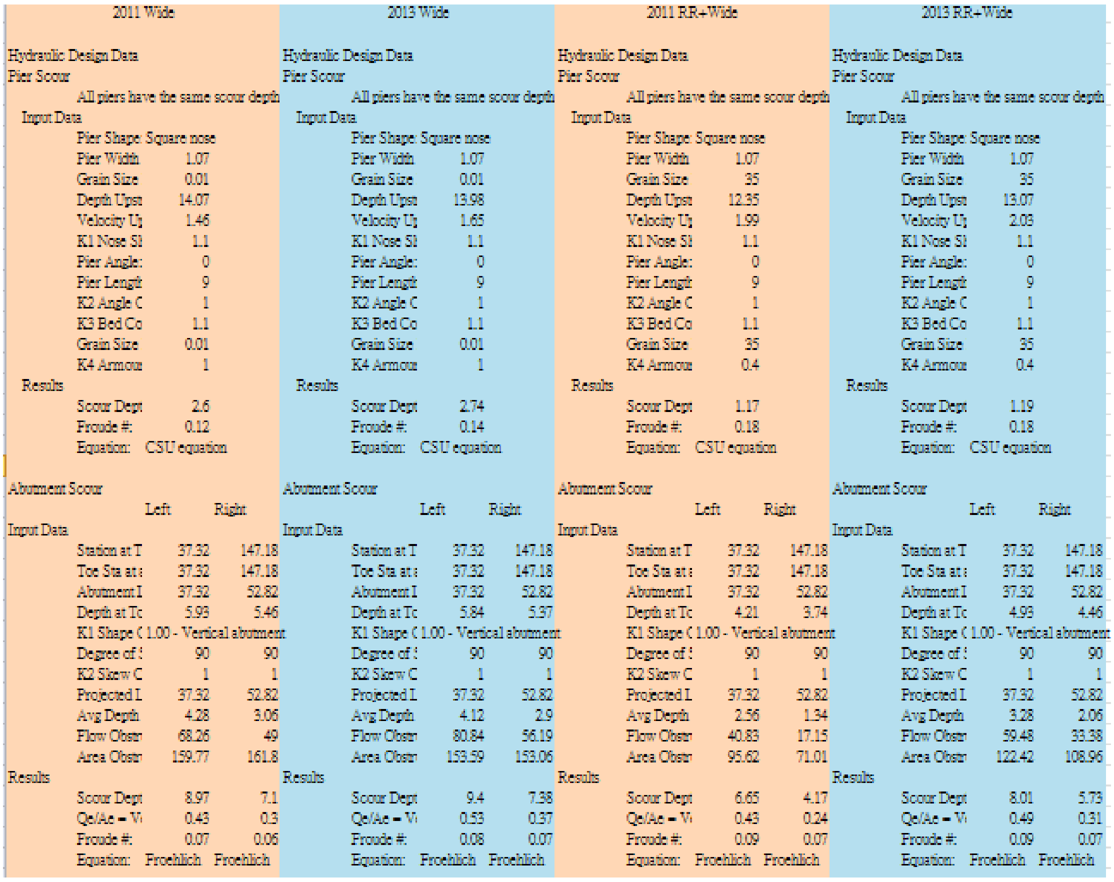
2013 RR+Sp		
Hydraulic Design Data		
Pier Scour		
All piers have the same scour depth		
Input Data		
Pier Shap	Square nose	
Pier Widtl	1.07	
Grain Size	35	
Depth Up:	13.11	
Velocity U	2.02	
K1 Nose S	1.1	
Pier Angle	0	
Pier Leng	9	
K2 Angle	1	
K3 Bed C	1.1	
Grain Size	35	
K4 Armou	0.4	
Results		
Scour De	1.19	
Froude #:	0.18	
Equation:	CSU equation	
Abutment Scour		
	Left	Right
Input Data		
Station at	64.44	147.18
Toe Sta at	64.44	147.18
Abutment	64.44	52.82
Depth at T	7.43	4.5
K1 Shape	0.55 - Spill-through abutment	
Degree of	90	90
K2 Skew	1	1
Projected	64.44	52.82
Avg Dept	4.57	2.1
Flow Obst	181	34.11
Area Obst	294.2	110.91
Results		
Scour De	8.72	4.13
Qe/Ae = V	0.62	0.31
Froude #:	0.09	0.07
Equation:	Froehlich	Froehlich

2011 Wing		
Hydraulic Design Data		
Pier Scour		
All piers have the same scour depth		
Input Data		
Pier Shape:	Square nose	
Pier Width	1.07	
Grain Size	0.01	
Depth Upst	14.1	
Velocity Uj	1.45	
K1 Nose Sl	1.1	
Pier Angle:	0	
Pier Lengt	9	
K2 Angle C	1	
K3 Bed Co	1.1	
Grain Size	0.01	
K4 Armour	1	
Results		
Scour Dept	2.6	
Froude #:	0.12	
Equation:	CSU equation	
Abutment Scour		
	Left	Right
Input Data		
Station at T	64.44	147.18
Toe Sta at	64.44	147.18
Abutment I	64.44	52.82
Depth at To	8.42	5.49
K1 Shape (	0.82 - Vert. with wing walls	
Degree of S	90	90
K2 Skew C	1	1
Projected I	64.44	52.82
Avg Depth	5.56	3.1
Flow Obstr	186.63	49.61
Area Obstr	358.48	163.59
Results		
Scour Dept	11.45	6.43
Qe/Ae = Vi	0.52	0.3
Froude #:	0.07	0.05
Equation:	Froehlich	Froehlich

2011 Spill		
Hydraulic Design Data		
Pier Scour		
All piers have the same scour depth		
Input Data		
Pier Shape:	Square nose	
Pier Width	1.07	
Grain Size	0.01	
Depth Upst	14.1	
Velocity Uj	1.45	
K1 Nose Sl	1.1	
Pier Angle:	0	
Pier Lengt	9	
K2 Angle C	1	
K3 Bed Co	1.1	
Grain Size	0.01	
K4 Armour	1	
Results		
Scour Dept	2.6	
Froude #:	0.12	
Equation:	CSU equation	
Abutment Scour		
	Left	Right
Input Data		
Station at T	64.44	147.18
Toe Sta at	64.44	147.18
Abutment I	64.44	52.82
Depth at To	8.42	5.49
K1 Shape (	0.55 - Spill-through abutment	
Degree of S	90	90
K2 Skew C	1	1
Projected I	64.44	52.82
Avg Depth	5.56	3.1
Flow Obstr	186.63	49.61
Area Obstr	358.48	163.59
Results		
Scour Dept	9.51	5.33
Qe/Ae = Vi	0.52	0.3
Froude #:	0.07	0.05
Equation:	Froehlich	Froehlich

2013 Wing		
Hydraulic Design Data		
Pier Scour		
All piers have the same scour depth		
Input Data		
Pier Shape:	Square nose	
Pier Width	1.07	
Grain Size	0.01	
Depth Upst	14.03	
Velocity Uj	1.64	
K1 Nose Sl	1.1	
Pier Angle:	0	
Pier Lengt	9	
K2 Angle C	1	
K3 Bed Co	1.1	
Grain Size	0.01	
K4 Armour	1	
Results		
Scour Dept	2.73	
Froude #:	0.14	
Equation:	CSU equation	
Abutment Scour		
	Left	Right
Input Data		
Station at T	64.44	147.18
Toe Sta at	64.44	147.18
Abutment I	64.44	52.82
Depth at To	8.35	5.42
K1 Shape (	0.82 - Vert. with wing walls	
Degree of S	90	90
K2 Skew C	1	1
Projected I	64.44	52.82
Avg Depth	5.42	2.95
Flow Obstr	224.65	57.33
Area Obstr	348.98	155.81
Results		
Scour Dept	12.07	6.65
Qe/Ae = Vi	0.64	0.37
Froude #:	0.09	0.07
Equation:	Froehlich	Froehlich

2013 Spill		
Hydraulic Design Data		
Pier Scour		
All piers have the same scour depth		
Input Data		
Pier Shape:	Square nose	
Pier Width	1.07	
Grain Size	0.01	
Depth Upst	14.03	
Velocity Uj	1.64	
K1 Nose Sl	1.1	
Pier Angle:	0	
Pier Lengt	9	
K2 Angle C	1	
K3 Bed Co	1.1	
Grain Size	0.01	
K4 Armour	1	
Results		
Scour Dept	2.73	
Froude #:	0.14	
Equation:	CSU equation	
Abutment Scour		
	Left	Right
Input Data		
Station at T	64.44	147.18
Toe Sta at	64.44	147.18
Abutment I	64.44	52.82
Depth at To	8.35	5.42
K1 Shape (	0.55 - Spill-through abutm	
Degree of S	90	90
K2 Skew C	1	1
Projected I	64.44	52.82
Avg Depth	5.42	2.95
Flow Obstr	224.65	57.33
Area Obstr	348.98	155.81
Results		
Scour Dept	9.88	5.43
Qe/Ae = Vi	0.64	0.37
Froude #:	0.09	0.07
Equation:	Froehlich	Froehlich



2011 Wide

Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width	1.07	
	Grain Size	0.01	
	Depth Ups	14.07	
	Velocity U	1.46	
	K1 Nose S	1.1	
	Pier Angle:	0	
	Pier Lengt	9	
	K2 Angle C	1	
	K3 Bed Co	1.1	
	Grain Size	0.01	
	K4 Armour	1	
Results			
	Scour Dept	2.6	
	Froude #:	0.12	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at T	37.32	147.18
	Toe Sta at	37.32	147.18
	Abutment L	37.32	52.82
	Depth at To	5.93	5.46
	K1 Shape C	1.00 - Vertical abutment	
	Degree of S	90	90
	K2 Skew C	1	1
	Projected L	37.32	52.82
	Avg Depth	4.28	3.06
	Flow Obstr	68.26	49
	Area Obstr	159.77	161.8
Results			
	Scour Dept	8.97	7.1
	Qe/Ae = V	0.43	0.3
	Froude #:	0.07	0.06
	Equation:	Froehlich	Froehlich

2013 Wide

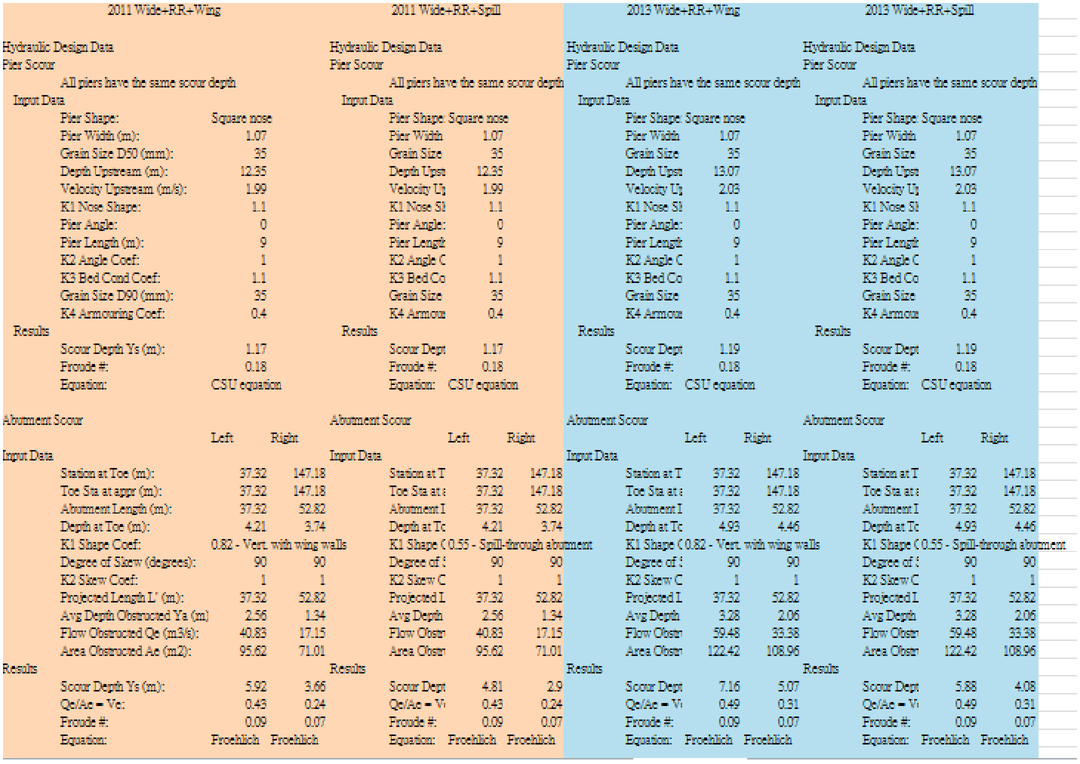
Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width	1.07	
	Grain Size	0.01	
	Depth Ups	13.98	
	Velocity U	1.65	
	K1 Nose S	1.1	
	Pier Angle:	0	
	Pier Lengt	9	
	K2 Angle C	1	
	K3 Bed Co	1.1	
	Grain Size	0.01	
	K4 Armour	1	
Results			
	Scour Dept	2.74	
	Froude #:	0.14	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at T	37.32	147.18
	Toe Sta at	37.32	147.18
	Abutment L	37.32	52.82
	Depth at To	5.84	5.37
	K1 Shape C	1.00 - Vertical abutment	
	Degree of S	90	90
	K2 Skew C	1	1
	Projected L	37.32	52.82
	Avg Depth	4.12	2.9
	Flow Obstr	80.84	56.19
	Area Obstr	153.59	153.06
Results			
	Scour Dept	9.4	7.38
	Qe/Ae = V	0.53	0.37
	Froude #:	0.08	0.07
	Equation:	Froehlich	Froehlich

2011 RR+Wide

Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width	1.07	
	Grain Size	35	
	Depth Ups	12.35	
	Velocity U	1.99	
	K1 Nose S	1.1	
	Pier Angle:	0	
	Pier Lengt	9	
	K2 Angle C	1	
	K3 Bed Co	1.1	
	Grain Size	35	
	K4 Armour	0.4	
Results			
	Scour Dept	1.17	
	Froude #:	0.18	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at T	37.32	147.18
	Toe Sta at	37.32	147.18
	Abutment L	37.32	52.82
	Depth at To	4.21	3.74
	K1 Shape C	1.00 - Vertical abutment	
	Degree of S	90	90
	K2 Skew C	1	1
	Projected L	37.32	52.82
	Avg Depth	2.56	1.34
	Flow Obstr	40.83	17.15
	Area Obstr	95.62	71.01
Results			
	Scour Dept	6.65	4.17
	Qe/Ae = V	0.43	0.24
	Froude #:	0.09	0.07
	Equation:	Froehlich	Froehlich

2013 RR+Wide

Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width	1.07	
	Grain Size	35	
	Depth Ups	13.07	
	Velocity U	2.03	
	K1 Nose S	1.1	
	Pier Angle:	0	
	Pier Lengt	9	
	K2 Angle C	1	
	K3 Bed Co	1.1	
	Grain Size	35	
	K4 Armour	0.4	
Results			
	Scour Dept	1.19	
	Froude #:	0.18	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at T	37.32	147.18
	Toe Sta at	37.32	147.18
	Abutment L	37.32	52.82
	Depth at To	4.93	4.46
	K1 Shape C	1.00 - Vertical abutment	
	Degree of S	90	90
	K2 Skew C	1	1
	Projected L	37.32	52.82
	Avg Depth	3.28	2.06
	Flow Obstr	59.48	33.38
	Area Obstr	122.42	108.96
Results			
	Scour Dept	8.01	5.73
	Qe/Ae = V	0.49	0.31
	Froude #:	0.09	0.07
	Equation:	Froehlich	Froehlich



2011 Wide+RR+Wing

Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width (m):	1.07	
	Grain Size D50 (mm):	35	
	Depth Upstream (m):	12.35	
	Velocity Upstream (m/s):	1.99	
	K1 Nose Shape:	1.1	
	Pier Angle:	0	
	Pier Length (m):	9	
	K2 Angle Coef:	1	
	K3 Bed Cond Coef:	1.1	
	Grain Size D90 (mm):	35	
	K4 Armouring Coef:	0.4	
Results			
	Scour Depth Ys (m):	1.17	
	Froude #:	0.18	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at Toe (m):	37.32	147.18
	Toe Sta at appr (m):	37.32	147.18
	Abutment Length (m):	37.32	52.82
	Depth at Toe (m):	4.21	3.74
	K1 Shape Coef:	0.82 - Vert. with wing walls	
	Degree of Skew (degrees):	90	90
	K2 Skew Coef:	1	1
	Projected Length L' (m):	37.32	52.82
	Avg Depth Obstructed Ya (m)	2.56	1.34
	Flow Obstructed Qe (m3/s):	40.83	17.15
	Area Obstructed Ae (m2):	95.62	71.01
Results			
	Scour Depth Ys (m):	5.92	3.66
	Qe/Ae = Ve:	0.43	0.24
	Froude #:	0.09	0.07
	Equation:	Froehlich	Froehlich

2011 Wide+RR+Spill

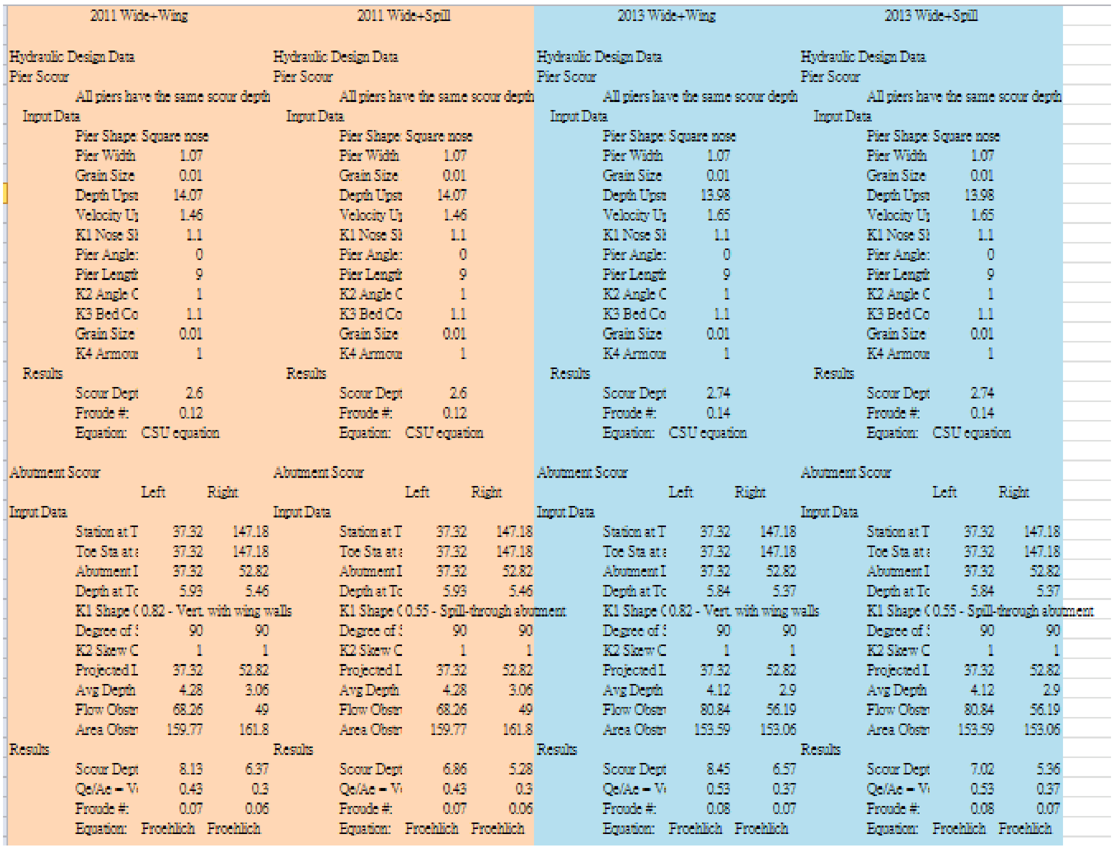
Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width	1.07	
	Grain Size	35	
	Depth Ups	12.35	
	Velocity U	1.99	
	K1 Nose S	1.1	
	Pier Angle:	0	
	Pier Lengt	9	
	K2 Angle C	1	
	K3 Bed Co	1.1	
	Grain Size	35	
	K4 Armour	0.4	
Results			
	Scour Dept	1.17	
	Froude #:	0.18	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at T	37.32	147.18
	Toe Sta at	37.32	147.18
	Abutment L	37.32	52.82
	Depth at To	4.21	3.74
	K1 Shape C	0.55 - Spill-through abutment	
	Degree of S	90	90
	K2 Skew C	1	1
	Projected L	37.32	52.82
	Avg Depth	2.56	1.34
	Flow Obstr	40.83	17.15
	Area Obstr	95.62	71.01
Results			
	Scour Dept	4.81	2.9
	Qe/Ae = V	0.43	0.24
	Froude #:	0.09	0.07
	Equation:	Froehlich	Froehlich

2013 Wide+RR+Wing

Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width	1.07	
	Grain Size	35	
	Depth Ups	13.07	
	Velocity U	2.03	
	K1 Nose S	1.1	
	Pier Angle:	0	
	Pier Lengt	9	
	K2 Angle C	1	
	K3 Bed Co	1.1	
	Grain Size	35	
	K4 Armour	0.4	
Results			
	Scour Dept	1.19	
	Froude #:	0.18	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at T	37.32	147.18
	Toe Sta at	37.32	147.18
	Abutment L	37.32	52.82
	Depth at To	4.93	4.46
	K1 Shape C	0.82 - Vert. with wing walls	
	Degree of S	90	90
	K2 Skew C	1	1
	Projected L	37.32	52.82
	Avg Depth	3.28	2.06
	Flow Obstr	59.48	33.38
	Area Obstr	122.42	108.96
Results			
	Scour Dept	7.16	5.07
	Qe/Ae = V	0.49	0.31
	Froude #:	0.09	0.07
	Equation:	Froehlich	Froehlich

2013 Wide+RR+Spill

Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width	1.07	
	Grain Size	35	
	Depth Ups	13.07	
	Velocity U	2.03	
	K1 Nose S	1.1	
	Pier Angle:	0	
	Pier Lengt	9	
	K2 Angle C	1	
	K3 Bed Co	1.1	
	Grain Size	35	
	K4 Armour	0.4	
Results			
	Scour Dept	1.19	
	Froude #:	0.18	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at T	37.32	147.18
	Toe Sta at	37.32	147.18
	Abutment L	37.32	52.82
	Depth at To	4.93	4.46
	K1 Shape C	0.55 - Spill-through abutment	
	Degree of S	90	90
	K2 Skew C	1	1
	Projected L	37.32	52.82
	Avg Depth	3.28	2.06
	Flow Obstr	59.48	33.38
	Area Obstr	122.42	108.96
Results			
	Scour Dept	5.88	4.08
	Qe/Ae = V	0.49	0.31
	Froude #:	0.09	0.07
	Equation:	Froehlich	Froehlich



2011 Wide+Wing			
Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width	1.07	
	Grain Size	0.01	
	Depth Upst	14.07	
	Velocity U	1.46	
	K1 Nose S	1.1	
	Pier Angle:	0	
	Pier Lengt	9	
	K2 Angle C	1	
	K3 Bed Co	1.1	
	Grain Size	0.01	
	K4 Armou	1	
Results			
	Scour Dept	2.6	
	Froude #:	0.12	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at T	37.32	147.18
	Toe Sta at	37.32	147.18
	Abutment	37.32	52.82
	Depth at T	5.93	5.46
	K1 Shape	0.82 - Vert. with wing walls	
	Degree of	90	90
	K2 Skew C	1	1
	Projected	37.32	52.82
	Avg Depth	4.28	3.06
	Flow Obstr	68.26	49
	Area Obstr	159.77	161.8
Results			
	Scour Dept	8.13	6.37
	Qe/Ae = V	0.43	0.3
	Froude #:	0.07	0.06
	Equation:	Froehlich	Froehlich

2011 Wide+Spill			
Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width	1.07	
	Grain Size	0.01	
	Depth Upst	14.07	
	Velocity U	1.46	
	K1 Nose S	1.1	
	Pier Angle:	0	
	Pier Lengt	9	
	K2 Angle C	1	
	K3 Bed Co	1.1	
	Grain Size	0.01	
	K4 Armou	1	
Results			
	Scour Dept	2.6	
	Froude #:	0.12	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at T	37.32	147.18
	Toe Sta at	37.32	147.18
	Abutment	37.32	52.82
	Depth at T	5.93	5.46
	K1 Shape	0.55 - Spill-through abutment	
	Degree of	90	90
	K2 Skew C	1	1
	Projected	37.32	52.82
	Avg Depth	4.28	3.06
	Flow Obstr	68.26	49
	Area Obstr	159.77	161.8
Results			
	Scour Dept	6.86	5.28
	Qe/Ae = V	0.43	0.3
	Froude #:	0.07	0.06
	Equation:	Froehlich	Froehlich

2013 Wide+Wing			
Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width	1.07	
	Grain Size	0.01	
	Depth Upst	13.98	
	Velocity U	1.65	
	K1 Nose S	1.1	
	Pier Angle:	0	
	Pier Lengt	9	
	K2 Angle C	1	
	K3 Bed Co	1.1	
	Grain Size	0.01	
	K4 Armou	1	
Results			
	Scour Dept	2.74	
	Froude #:	0.14	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at T	37.32	147.18
	Toe Sta at	37.32	147.18
	Abutment	37.32	52.82
	Depth at T	5.84	5.37
	K1 Shape	0.82 - Vert. with wing walls	
	Degree of	90	90
	K2 Skew C	1	1
	Projected	37.32	52.82
	Avg Depth	4.12	2.9
	Flow Obstr	80.84	56.19
	Area Obstr	153.59	153.06
Results			
	Scour Dept	8.45	6.57
	Qe/Ae = V	0.53	0.37
	Froude #:	0.08	0.07
	Equation:	Froehlich	Froehlich

2013 Wide+Spill			
Hydraulic Design Data			
Pier Scour			
	All piers have the same scour depth		
Input Data			
	Pier Shape:	Square nose	
	Pier Width	1.07	
	Grain Size	0.01	
	Depth Upst	13.98	
	Velocity U	1.65	
	K1 Nose S	1.1	
	Pier Angle:	0	
	Pier Lengt	9	
	K2 Angle C	1	
	K3 Bed Co	1.1	
	Grain Size	0.01	
	K4 Armou	1	
Results			
	Scour Dept	2.74	
	Froude #:	0.14	
	Equation:	CSU equation	
Abutment Scour			
		Left	Right
Input Data			
	Station at T	37.32	147.18
	Toe Sta at	37.32	147.18
	Abutment	37.32	52.82
	Depth at T	5.84	5.37
	K1 Shape	0.55 - Spill-through abutment	
	Degree of	90	90
	K2 Skew C	1	1
	Projected	37.32	52.82
	Avg Depth	4.12	2.9
	Flow Obstr	80.84	56.19
	Area Obstr	153.59	153.06
Results			
	Scour Dept	7.02	5.36
	Qe/Ae = V	0.53	0.37
	Froude #:	0.08	0.07
	Equation:	Froehlich	Froehlich

Appendix F – Manning's n Values from HEC-RAS Hydraulic Manual

Channels

Type of Channel and Description	Minimum	Normal	Maximum
Natural streams - minor streams (top width at floodstage < 100 ft)			
1. Main Channels			
a. clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
b. same as above, but more stones and weeds	0.030	0.035	0.040
c. clean, winding, some pools and shoals	0.033	0.040	0.045
d. same as above, but some weeds and stones	0.035	0.045	0.050
e. same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. same as "d" with more stones	0.045	0.050	0.060
g. sluggish reaches, weedy, deep pools	0.050	0.070	0.080
h. very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
2. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
a. bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
b. bottom: cobbles with large boulders	0.040	0.050	0.070
3. Floodplains			
a. Pasture, no brush			
1.short grass	0.025	0.030	0.035
2. high grass	0.030	0.035	0.050
b. Cultivated areas			
1. no crop	0.020	0.030	0.040
2. mature row crops	0.025	0.035	0.045
3. mature field crops	0.030	0.040	0.050
c. Brush			
1. scattered brush, heavy weeds	0.035	0.050	0.070

2. light brush and trees, in winter	0.035	0.050	0.060
3. light brush and trees, in summer	0.040	0.060	0.080
4. medium to dense brush, in winter	0.045	0.070	0.110
5. medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. dense willows, summer, straight	0.110	0.150	0.200
2. cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. same as 4. with flood stage reaching branches	0.100	0.120	0.160
4. Excavated or Dredged Channels			
a. Earth, straight, and uniform			
1. clean, recently completed	0.016	0.018	0.020
2. clean, after weathering	0.018	0.022	0.025
3. gravel, uniform section, clean	0.022	0.025	0.030
4. with short grass, few weeds	0.022	0.027	0.033
b. Earth winding and sluggish			
1. no vegetation	0.023	0.025	0.030
2. grass, some weeds	0.025	0.030	0.033
3. dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. earth bottom and rubble sides	0.028	0.030	0.035
5. stony bottom and weedy banks	0.025	0.035	0.040
6. cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. no vegetation	0.025	0.028	0.033
2. light brush on banks	0.035	0.050	0.060
d. Rock cuts			

1. smooth and uniform	0.025	0.035	0.040
2. jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. dense weeds, high as flow depth	0.050	0.080	0.120
2. clean bottom, brush on sides	0.040	0.050	0.080
3. same as above, highest stage of flow	0.045	0.070	0.110
4. dense brush, high stage	0.080	0.100	0.140
5. Lined or Constructed Channels			
a. Cement			
1. neat surface	0.010	0.011	0.013
2. mortar	0.011	0.013	0.015
b. Wood			
1. planed, untreated	0.010	0.012	0.014
2. planed, creosoted	0.011	0.012	0.015
3. unplaned	0.011	0.013	0.015
4. plank with battens	0.012	0.015	0.018
5. lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. trowel finish	0.011	0.013	0.015
2. float finish	0.013	0.015	0.016
3. finished, with gravel on bottom	0.015	0.017	0.020
4. unfinished	0.014	0.017	0.020
5. gunite, good section	0.016	0.019	0.023
6. gunite, wavy section	0.018	0.022	0.025
7. on good excavated rock	0.017	0.020	
8. on irregular excavated rock	0.022	0.027	
d. Concrete bottom float finish with sides of:			
1. dressed stone in mortar	0.015	0.017	0.020
2. random stone in mortar	0.017	0.020	0.024
3. cement rubble masonry, plastered	0.016	0.020	0.024
4. cement rubble masonry	0.020	0.025	0.030
5. dry rubble or riprap	0.020	0.030	0.035

e. Gravel bottom with sides of:			
1. formed concrete	0.017	0.020	0.025
2. random stone mortar	0.020	0.023	0.026
3. dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. glazed	0.011	0.013	0.015
2. in cement mortar	0.012	0.015	0.018
g. Masonry			
1. cemented rubble	0.017	0.025	0.030
2. dry rubble	0.023	0.032	0.035
h. Dressed ashlar/stone paving	0.013	0.015	0.017
i. Asphalt			
1. smooth	0.013	0.013	
2. rough	0.016	0.016	
j. Vegetal lining	0.030		0.500

Appendix G – Risk Assessment

Risk Identification

The probability of encountering the risks during the project activities are:

1. Eye fatigue from computer operations: significant
2. Car accident when travelling to floodway site: very slight

Risk Evaluation

The frequency of exposure and level of consequences of the risks during the project activities are:

1. Eye fatigue from computer operations: frequently and minor damage
2. Car accident when travelling to floodway site: very rarely and major injury

Risk Control

In order to minimise these risks:

1. Eye fatigue from computer operations:
   - Take a five minute break from computer operation every hour
   - Adjust monitor brightness when operating in a dark environment
   - Avoid long periods of operation
2. Car accident when travelling to floodway site:
   - Obey road rules
   - Avoid driving when fatigued
   - Avoid road hazards

Ethical Responsibility

Referring to the Code of Ethics provided by Engineers Australia (2010), the project activities do not breach the code's guidelines and will be carried out with the well-informed conscience of the student.