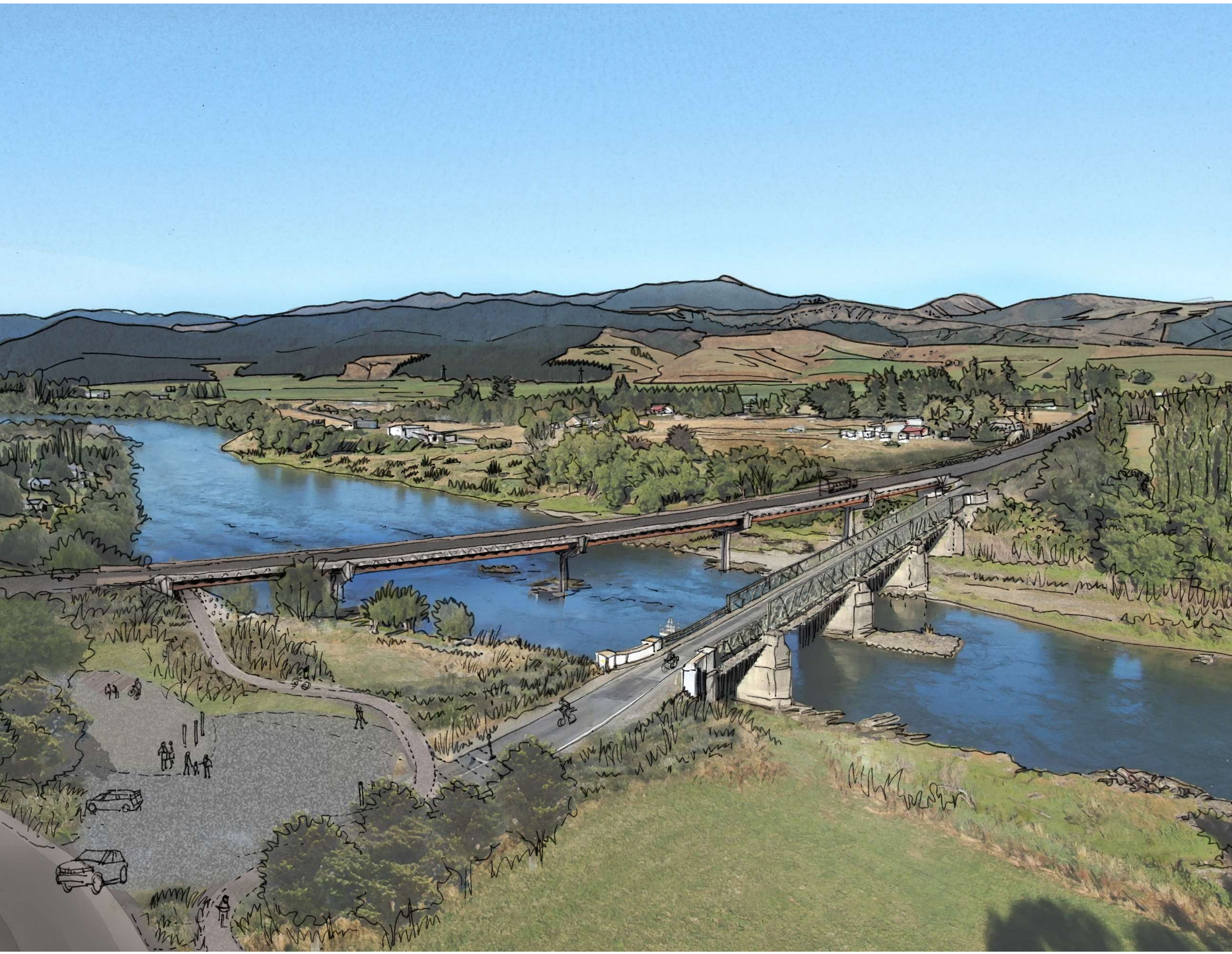
# New Beaumont Bridge

## Hydraulic Assessment & Design Information



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# Contents

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| 1 | First issue of the document |
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| | |

# 1 Introduction

## 1.1 Background

The SH8 Beaumont Bridge across the Clutha River is to be replaced by NZTA. The bridge will be located on a relatively straight section of the Clutha River just downstream of a river bend. The channel is incised with rock outcrops protruding the low flow water surface both in the centre of the channel and on the true right bank (see Figure 1-1). The location of the proposed replacement bridge is located immediately downstream of the existing SH8 Beaumont Bridge which is to be retained for pedestrian and cyclist use. Consequently, the hydraulic analysis will consider the effects of the existing bridge.

To support the design and construction of the replacement bridge, peak water levels and flow velocities for key design events have been estimated. These have then been used to assess the scour risk to the proposed bridge and to design appropriate scour protection

*Figure 1-1 View of the Clutha River looking downstream from the existing bridge towards the site of the proposed bridge*

## 1.2 Design Events

The two key design events for the bridge are detailed in the NZ Transport Agency Bridge Manual (NZTA, 2018). These are the Serviceability Limit State (SLS) flood event and the Ultimate Limit State (ULS) flood event. Based on the bridge being an importance level 3 bridge, these design events are the 1% annual exceedance probability (AEP) flood event including the effects of climate change to 2120 (+CC) and the 0.1% AEP +CC respectively.

The SLS design event is critical for setting the bridge soffit level to provide a specified freeboard, and to ensure scour during this flood event does not affect the bridge's serviceability; such as by eroding the approach road embankments. The ULS design event is critical to ensure the bridge structure is not undermined by scour at the piers or abutments during a flood event of this magnitude, and to ensure the bridge can withstand the hydrodynamic loading from the floodwaters.

# 2 Hydrological Assessment

A hydrological assessment was undertaken to derive design flood estimates at the location of the proposed bridge (WSP Opus, 2018). This hydrological assessment also provides a discussion on the data quality, flood frequency analysis methodology, implications of the 0.1% AEP design standard (equivalent to a 1,000-year average recurrence interval (ARI)), climate change scenarios, and on a low flow analysis to help facilitate the bridge construction.

The SLS and ULS flood estimates inclusive of climate change to 2120 are 5,250m$^3$/s and 7,100m$^3$/s respectively (highlighted in Table 2-1). The full range of flood estimates are provided, for both current and future climate conditions, in Table 2-1. Note that climate change projections have only been provided for floods greater than a 2% AEP.

*Table 2-1 Design flood estimates adjusted for climate change (m$^3$/s). Values rounded to the nearest 50m$^3$/s*

| ARI (yr.) | AEP (%) | 2018 | 2070[1] | 2120[2] |
|---|---|---|---|---|
| 2.33 | 50 | 1,580 | | |
| 5 | 20 | 2,130 | | |
| 10 | 10 | 2,630 | | |
| 20 | 5 | 3,140 | | |
| 25 | 4 | 3,280 | | |
| 50 | 2 | 3,810 | 4,200 | 4,600 |
| 100 | 1 | 4,320 | 4,750 | 5,250 |
| 1000 | 0.01 | 5,850 | 6,450 | 7,100 |
| 2500 | 0.04 | 6,500 | 7,150 | 7,850 |

# 3 Hydraulic Assessment

## 3.1 Methodology

A comprehensive hydraulic assessment for a significant structure such as the Beaumont Bridge would commonly include a coupled one-dimensional and two-dimensional computational hydraulic model, representing the river channel and floodplain respectively. Furthermore, this model would be run in an unsteady state to represent the effects of attenuation across the floodplain. However, given that there have been few flooding concerns with the existing SH8 bridge structure across its extensive lifespan (greater than 130 years), it is expected that the hydraulic risk to the proposed structure is sufficiently low and a more basic hydraulic assessment is appropriate. It is also important to note that the hydrological assessment (WSP Opus, 2018) found that there was minimal attenuation between Roxburgh Dam and Balclutha, and so the effects of using a steady-state model are also likely to be minimal and within the model's margin of error.

On this basis, the methodology adopted for the hydraulic assessment of the proposed bridge structure is based on a one-dimensional steady-state hydraulic model. This is more simplistic than a fully comprehensive hydraulic assessment outlined above and does not fully represent the

[1] The flows in this column are as a result of 50 years of climate change.
[2] The flows in this column are as a result of 100 years of climate change.

floodplain or any effects of attenuation. To enhance the model, a portion of the surveyed cross-sections were extended on the left and/or right banks in post-processing to better represent the floodplain.

The simplification of the hydraulic modelling can lead, and likely will have led, to conservative model results by some minor to moderate extent. The one-dimensional nature of the model means that the river channel, and the immediate floodplain (50-100m width) on each channel bank, is represented by a cross-section perpendicular to the flow of water. Where the water reaches the edge of the surveyed cross-section, the model assumes that the cross-section has vertical walls. Therefore, to convey the modelled flow down the channel, the modelling approach causes the water level to increase more than might be expected, rather than allow the flow to spread out across the floodplain and have a smaller increase in water level. Without undertaking a comprehensive assessment, it is not possible to quantify the magnitude of the model's conservativism. However, the conservative outcome within the model will be discussed qualitatively alongside the model results in Section 3.4. It is important to note that the model was calibrated to a flow of 3,250$m^3/s$, as discussed below. Overall, the methodology is considered appropriate given the level of risk to the bridge structure by floodwater action.

The one-dimensional computational hydraulic model used in this assessment to estimate the peak water levels and flow velocities at the location of the proposed bridge structure was constructed using HEC-RAS[3] computational hydraulic modelling software package. This is discussed further in Section 3.3.

## 3.2 Survey Data

To inform the hydraulic assessment, Eliot Sinclair was engaged to conduct a hydrographic survey of the Clutha River in a reach approximately 350m long around the existing and proposed bridge structures (Eliot Sinclair, 2018). This survey included six river cross-sections perpendicular to the flow of the river, existing bridge soffit levels across the bridge span, and a hydrographic survey of the river bed at 2m spacings. The survey was undertaken on 3 July 2018 in Dunedin Vertical Datum 1958 (DVD58) and in NZGD2000 / North Taieri Circuit 2000. All levels in this report are in terms of this vertical datum and projection.

Cross-sectional survey data supplied by Otago Regional Council (ORC) was used in addition to the Eliot Sinclair survey data to inform the hydraulic assessment. The locations of these cross-sections are shown below in Figure 3-1.

The soffit level of the existing bridge structure was surveyed to be 48.68m RL (Eliot Sinclair, 2018).

[3] HEC-RAS is a computational hydraulic modelling software package developed by the United States Army Corps of Engineers (USACE) that is commonly used within the industry to model the hydraulics of water flow through open channels and bridges.


*Figure 3-1 Cross-sections surveyed by Eliot Sinclair (shown in pink) and by ORC (shown in green)*

## 3.3 HEC-RAS Model

### 3.3.1 Overview

As discussed in Section 3.1, a HEC-RAS model was created to estimate the peak water levels and flow velocities for key design events. This model is based on twelve cross-sections over a reach of 4370m, with the majority focused around a 1520m reach of the Clutha River. Surveyed cross-sections, historical bridge drawings (PWD, 1885), and the latest design information was used to represent the existing and proposed bridge structures in the model. The spill through abutments of the proposed bridge were modelled as vertical structures located at the toe of the abutments to provide flexibility in the design of the abutments.

Boundary conditions were defined based on design flows and an assumed average energy slope at the upstream and downstream ends of the model. The average energy slopes were 0.27% and 0.13% at the upstream and downstream boundaries of the model respectively.

### 3.3.2 Calibration

Two calibration flood events were run based on the January 1994 and December 1995 flood events. These events are still relevant as the flows in this reach of the Clutha River have been modified by the Roxburgh Dam since 1956 (WSP Opus, 2018). These floods were estimated to have peak discharges of 2,700m$^3$/s and 3,250m$^3$/s respectively (WCS, 1996). Flood levels at Beaumont estimated from surveyed debris marks were 44.45m RL and 45.62m RL for the 1994 and 1995 flood events respectively. It was found that by applying a Manning's n value of 0.049 to represent the roughness of the river, the model predicted peak water levels closest to the measured water levels of the calibration flood events. The 1994 flood event was over-estimated by 0.38m, and the 1995 flood event was under-estimated by 0.04m. Given that the 1995 flood event was the larger of the two flood events, more emphasis was placed on this flood event. On average, increasing or decreasing the Manning's n value by 0.001 increased or decreased the water levels by 0.09m. A summary of the calibration results is provided below in Table 3-1.

*Table 3-1 Calibration results*

| Flood Event | Flow (m$^3$/s) | Surveyed Flood Level (m RL) | Modelled Flood Level (m RL) | Difference (m) |
|---|---|---|---|---|
| January 1994 | 2,700 | 44.45 | 44.83 | +0.38 |
| December 1995 | 3,250 | 45.62 | 45.58 | -0.04 |

## 3.4 Key Results

The HEC-RAS model was run as part of the hydraulic assessment to estimate the water level and average cross-sectional flow velocity upstream of the proposed bridge for the SLS and ULS design events. These results are presented below in Table 3-2.

*Table 3-2 Hydraulic assessment results for key design events upstream of the proposed bridge*

| Design Event | Estimated Flow (m$^3$/s) | Water Level Upstream of Proposed Bridge (m RL) | Average Flow Velocity (m/s) |
|---|---|---|---|
| SLS | 5,250 | 47.93 | 3.4 |
| ULS | 7,100 | 49.91 | 3.6 |

To provide input into the design of the cycle pathways and other bridge features, the predicted water levels upstream of the proposed bridge for all flood events of current and future climate conditions have also been provided (see Table 3-3). The SLS and ULS flood events have been highlighted in orange.

*Table 3-3 Predicted water levels upstream of the proposed bridge for all flood events of current and future climate conditions (m RL)*

| *ARI (yr.)* | *AEP (%)* | 2018 | 2070 | 2120 |
|---|---|---|---|---|
| *2.33* | *50* | 43.01 | | |
| *5* | *20* | 43.91 | | |
| *10* | *10* | 44.62 | | |
| *20* | *5* | 45.31 | | |
| *25* | *4* | 45.53 | | |
| *50* | *2* | 46.21 | 46.70 | 47.22 |
| *100* | *1* | 46.91 | 47.39 | 47.93 |
| *1000* | *0.01* | 48.45 | 49.26 | 49.91 |
| *2500* | *0.04* | 49.31 | 49.95 | 50.55 |

## 3.5 Effects on Existing Flood Risk

The proposed bridge's abutments will reduce the flow of floodwaters across the floodplain, and its piers will impede the flow of water down the Clutha River channel. Consequently, it can be expected that the water levels upstream of the new bridge will increase. Table 3-4 details the existing flood levels, and the expected change in flood levels following the construction of the proposed bridge, for a range of flood events.

*Table 3-4 Predicted flood levels upstream of the existing SH8 Bridge in the existing and proposed situations, with the change in flood level shown in brackets. (m RL)*

| FLOOD EVENT | FLOW | EXISTING FLOOD LEVEL | FLOOD LEVEL IN PROPOSED SITUATION |
|---|---|---|---|
| 50% AEP | 1,580m³/s | 43.08 | 43.11 (+0.03) |
| 20% AEP | 2,130m³/s | 44.00 | 44.04 (+0.04) |
| 10% AEP | 2,630m³/s | 44.72 | 44.77 (+0.05) |
| 5% AEP | 3,140m³/s | 45.43 | 45.49 (+0.06) |
| 2% AEP | 3,810m³/s | 46.37 | 46.44 (+0.07) |
| 1% AEP | 4,320m³/s | 47.09 | 47.17 (+0.08) |
| 1% AEP +CC2120 | 5,250m³/s | 48.16 | 48.26 (+0.10) |

The results show that the effect of the proposed SH8 Bridge across the Clutha River will have a less than minor effect on the flood risk upstream of the existing SH8 Bridge. The maximum increase in flood level for the scenarios considered is 0.10m, which is within the model error range.

The existing bridge soffit level was surveyed to be 48.68m RL. This provides 0.52m of freeboard to the SLS flood event for the 2120 climate. This will reduce to 0.42m following the construction of the proposed bridge.

## 3.6 Sensitivity to Climate Change Scenarios

The hydraulic assessment results detailed above include the effects of climate change out to 2120 for climate change scenario RCP6.0. Further discussion on the various climate change scenarios and on the guidance from the Ministry for Environment is available in the hydrological assessment report (WSP Opus, 2018).

To quantify the effects of the uncertainty related to the expected amount of change to climate conditions, the SLS and ULS flood events have been run in the HEC-RAS model for the different climate change scenarios, taken out to 2120. The variation in water levels from RCP6.0 for each of these scenarios is presented in Table 3-5. It is important to note that the conservative approach inherent in the model still applies to each of the results in the table below.

The results below indicate that RCP6.0 is an appropriate climate change scenario to use for the design of the proposed SH8 Beaumont Bridge. The maximum increase in water level to RCP8.5 is only 0.30m. This is considered to be a minor risk to the bridge structure as designed should this climate change scenario occur. Alternatively, if climate change scenario RCP4.5 had been adopted, the difference in water level between RCP4.5 and RCP8.5 for the ULS event would be much more significant. This change in water level is estimated to be 0.73m, which would present a risk to the structure if designed to RCP4.5, should climate change scenario RCP8.5 eventuate.

*Table 3-5 Predicted water levels for various climate change scenarios taken out to 2120, with the change in water level from RCP6.0 shown in brackets. (m RL)*

| SCENARIO | SLS | ULS |
|---|---|---|
| *RCP2.6* | 47.17<br>(-0.76 | 48.79<br>(-1.12) |
| *RCP4.5* | 47.60<br>(-0.33) | 49.48<br>(-0.43) |
| *RCP6.0* | 47.93 | 49.91 |
| *RCP8.5* | 48.15<br>(+0.22) | 50.21<br>(+0.30) |

# 4 Scour Assessment

## 4.1 Setting

As discussed in Section 1.1, the proposed bridge structure is to be located across a straight section of an incised channel immediately downstream of a bend (Figure 4-1). Consequently, the flow velocity is likely to be slightly higher on the true left (i.e. east or Dunedin) side of the river. The channel is characterised by rock outcrops that protrude from the low flow water level.

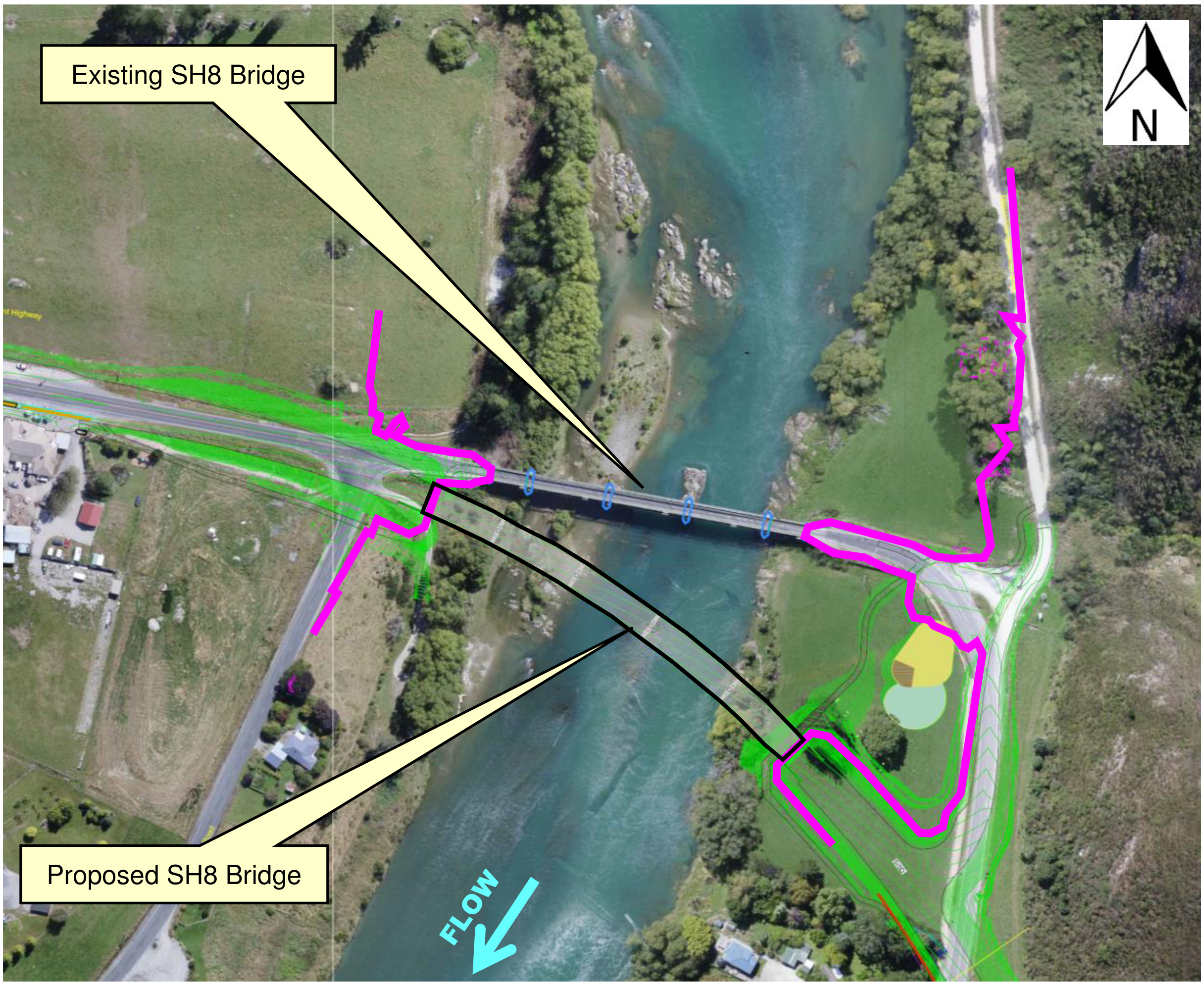


*Figure 4-1 Detail of Clutha River and the existing and proposed SH8 bridges showing channel alignment and SLS event flood extent near the bridges (pink).*

The proposed bridge structure (Figure 4-2) will have four single circular piers in between the abutments which are located on either side of the river channel. These piers will be founded upon the rock outcrops for ease of construction, to minimise the effects on the river flow, and to reduce the risk of scour undermining the bridge structure.

This scour assessment will focus on two key components; the scour risk to the piers located on the rock outcrops, and the scour risk to the abutments located on the floodplain either side of the river channel.

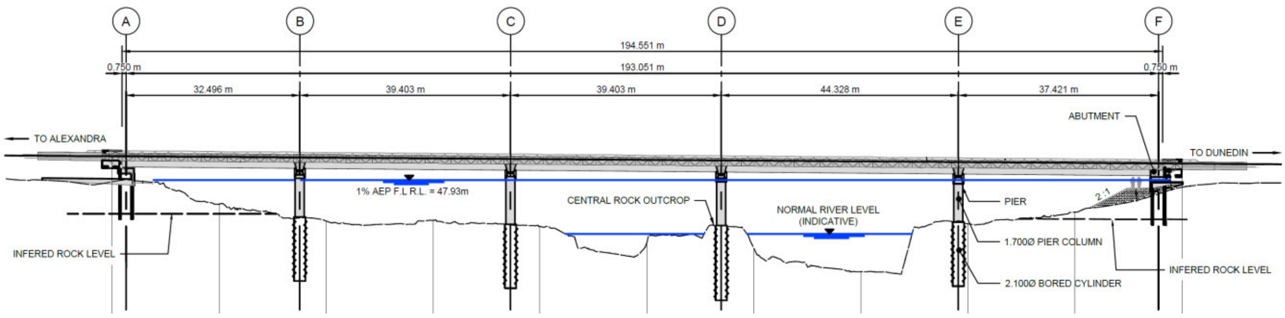


*Figure 4-2 Structural long section of the proposed bridge viewed from downstream*

## 4.2 Design Requirements

As discussed in Section 1.2, the two key design events are the SLS flood event (1% AEP for the 2120 climate), and the ULS flood event (0.1% AEP for the 2120 climate). It is critical that the road embankments are not washed out in the SLS flood event, and that the piers and abutments are not undermined during a ULS flood event.

## 4.3 Analysis of Potential Scour to Piers

The potential for scour to occur during a flood event is a result of the hydraulic forces applied by the floodwaters against the river bed material, and the ability of that river bed material to resist those hydraulic forces. Determining the magnitude of scour that could occur must be validated against observations of the surrounding river. In this case, it must be acknowledged that the existing SH8 bridge structure has had an asset life of approximately 135 years with minimal scour.

### *4.3.1 Existing Bridge*

The existing SH8 bridge structure (Figure 4-3) was well founded with concrete footings on rock outcrops that have protected its piers during flood events. The only scour known to have occurred to the existing SH8 bridge structure is underneath Pier D where the rock, on which the concrete footing sits, has been locally undercut by up to 1.8m (Figure 4-4). Only 1.0m of that undercutting is directly under the pier footing with the opening width being a very small proportion of the entire footing perimeter. The deepest point of this scour is located approximately 2.5m beneath the top of the concrete footing. No repairs have been made to date and consequently it must be assumed that this scour is being monitored and is not currently of concern.

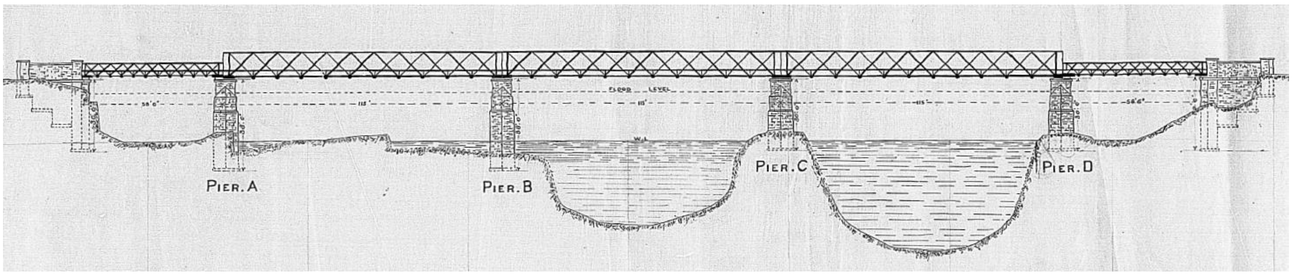


*Figure 4-3 Existing bridge viewed from downstream (Source: PWD, 1885)*

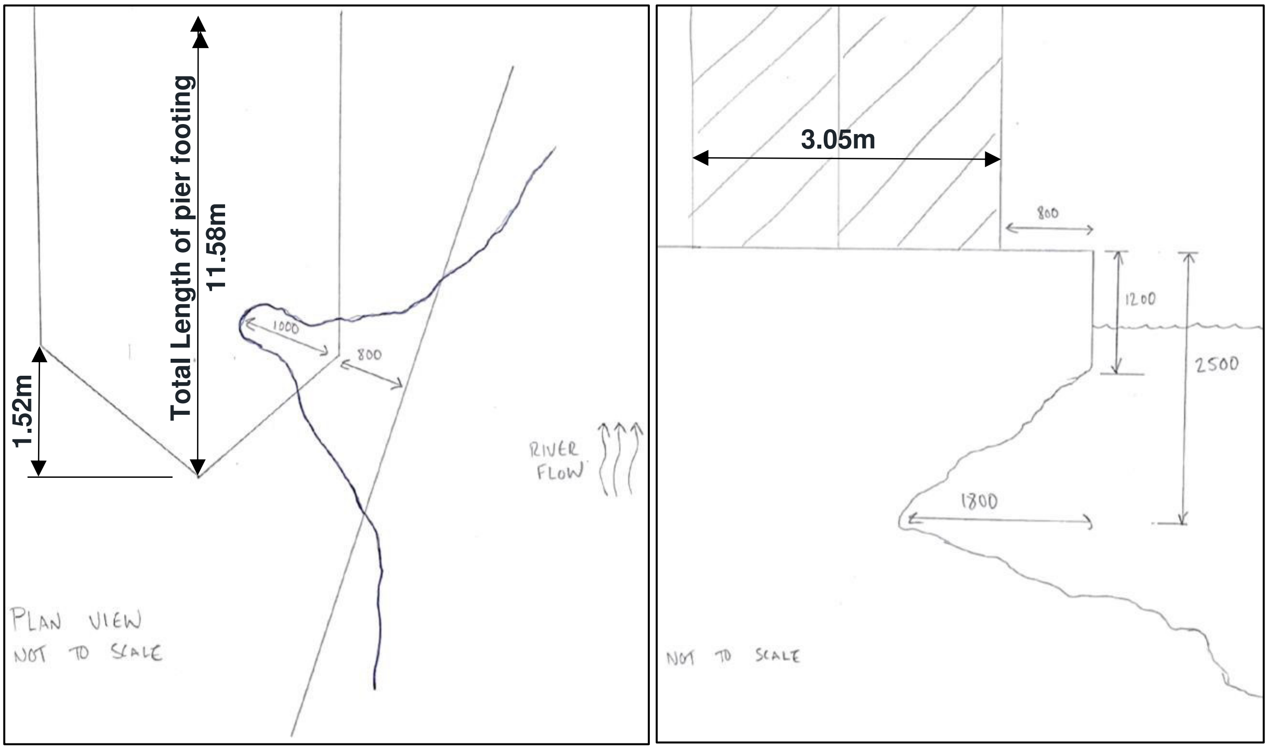


*Figure 4-4 Plan (left) and section (right) of scour undercutting of Pier D of the existing SH8 bridge (Source: Underwater Solutions Ltd, 2014 and PWD, 1885)*

#### *4.3.2 Proposed Bridge*

The proposed bridge pier piles will generally be founded on rock that is either phyllite or metasandstone. The geological properties of these rocks are discussed in detail in Grindley (2018). Consequently, significant scour is only likely to occur over multiple flood events over many years rather than during a single flood event as would occur for a river with non-cohesive bed material. A scour analysis of erodible rock was undertaken based on HEC-18 (FHWA, 2012). This analysis showed that only the weaker phyllite rock has the potential for some scour, such as at Pier E. This correlates well with a recent dive inspection of the key pier locations (Underwater Solutions Ltd, 2018). This dive inspection highlighted up to 2m of undercutting near the proposed location of Pier E, and up to 0.5m of undercutting at some of the other proposed pier locations. The proposed piles of Pier E are expected to be embedded deeper than the weaker phyllite rock into the harder metasandstone rock below.

#### *4.3.3 Pier Scour Summary*

The existing bridge is had a service life of approximately 135 years with minimal scour. There is however some evidence of minor long-term scour. The pier piles of the proposed bridge will be founded on the same rock material as that of the existing bridge. Hence scour is not a significant risk to the proposed bridge structure. It is however recommended that the potential for long-term scour (i.e. over many years) is considered during the design of the pier piles such that they can withstand some undercutting in areas of weaker rock material.

### 4.4 Analysis of Potential Scour to Abutments

The proposed abutments are located outside of the main channel on the true left and right floodplains. The proposed bridge will be constructed downstream of the existing bridge with the abutments set back slightly further from the main channel than the existing bridge. The average flow velocity in the main river channel in the SLS flood event is approximately 3.4m/s. The flow velocities on the floodplain are expected to be significantly lower. However, the south corner of the eastern abutment does protrude close to the main river channel where the flow velocities will be much closer to the average flow velocity. Figure 4-5 shows a plan image of this abutment.

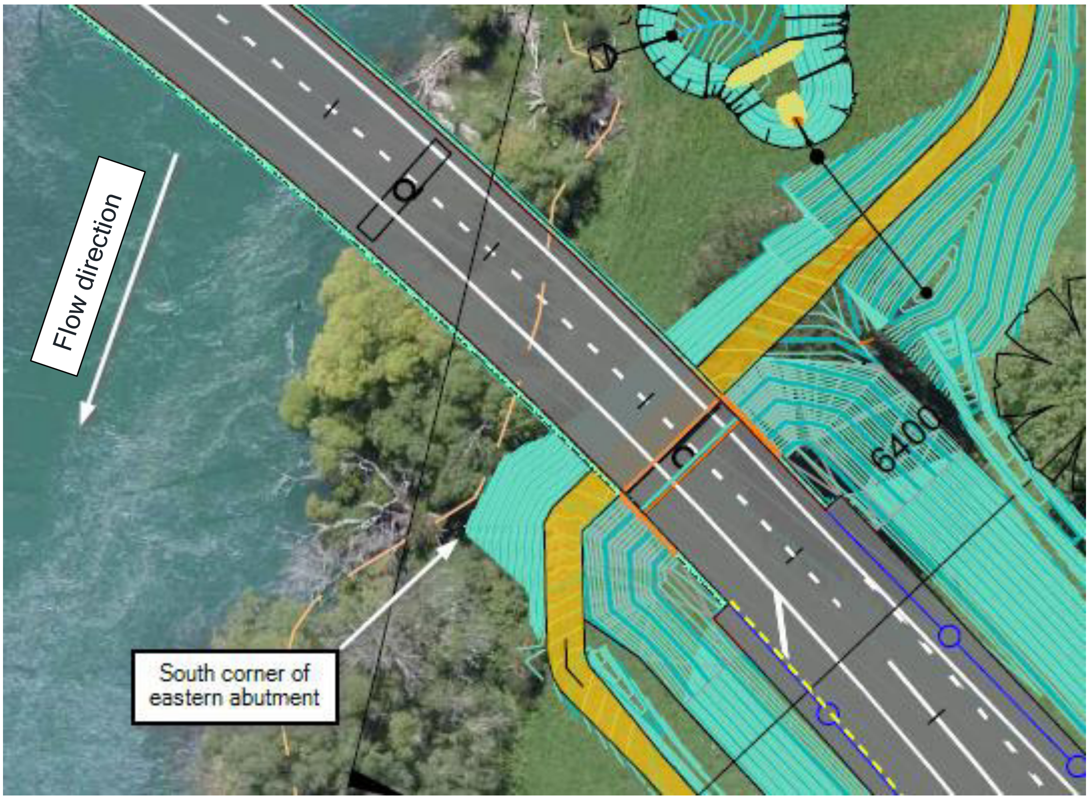


*Figure 4-5 Location of eastern abutment in relation to the river channel*

Both abutments will be founded in the metamorphic rock that, as noted previously, does not have a significant risk of scour. Boreholes BH02 and BH05 are close to the west and east abutments respectively and show that the metamorphic rock is found at RL 44m and RL 40m respectively. The material above the metamorphic rock consists of approximately 2.6m to 3m of sandy gravels and silty sand. The road embankments will be approximately 3m to 4m above existing ground level.

The only risk to the structure is therefore that the road embankment and the material that this is placed on will be scoured away during the SLS event. This can be mitigated by the provision of suitably designed scour protection.

# 5 Scour Protection Design

Scour protection for the proposed SH8 Beaumont Bridge has been designed to meet the design requirements outlined in Section 4.2.

## 5.1 Piers

Scour protection for the piers has not been designed as it is anticipated that the piles will be founded sufficiently deep into the scour-resistant metamorphic rock to structurally survive scour to the rock material in the channel.

## 5.2 Abutments and Embankments

### 5.2.1 *Rock Size*

Scour protection has been designed for the bridge embankments at the abutments for the SLS flood event in the form of a rock revetment based on methods described in Melville and Coleman (2000). This publication lists a large number of methods to determine the rock size required. Those based on Austroads (1994), Croad (1989) Richardson and Davis (1995) and Pagan-Ortiz (1991) were used as they are the most appropriate to use in the selection of rock for abutment protection. The results of these methods show that a median rock size (i.e. $D_{50}$) of 0.60m will be required. Table 5-1 summarises key parameters of the rock revetment design including the riprap rock layer.

*Table 5-1 Rock revetment design parameters*

| PARAMETER | VALUE |
|---|---|
| Design approach flow velocity | 3.4m/s |
| Design flow depth at toe | 8.5m |
| Rock Revetment<br>• crest level<br>• slope | <br>Up to cycle path level<br>2H:1V;<br>1.5H:1V at south corner of eastern abutment |
| Riprap Rock Layer<br>• Median rock size ($D_{50}$)<br>• Layer thickness | <br>0.60m<br>1.20m |
| Granular Protection Layer<br>• Median rock size ($D_{50}$)<br>• Layer thickness | <br>63mm<br>200mm |
| Geotextile | Bidim A64 Geotextile or equivalent |

### 5.2.2 Geotextile and Granular Protection Layer

A geotextile is required to prevent fill material from passing through the rock revetment. This geotextile will wrap around the toe of the rock revetment and extend to the crest of the rock revetment. A granular geotextile protection layer is proposed to protect the geotextile from damage, particularly during the construction phase. Table 5-1 summarises the key parameters of the geotextile and granular protection layer.

### 5.2.3 Revetment Extent

The rock revetment will wrap around the full abutment in line with the abutment face to minimise risk of undermining as per the minimum recommended extent in Figure 5-1. The bridge embankments will have batter slopes of 2H:1V, however the south corner of the eastern abutment may require a steeper batter slope of 1.5H:1V due to the limited space available and has been considered when selecting the size of the riprap rocks.

### 5.2.4 Revetment Toe

The toe of the rock revetment must be positioned such that it is not at significant risk of undermining by scour. This would typically be achieved by locating the toe of the rock revetment beneath the scour depth, or by including a launching pad in the rock revetment so that natural scour processes shift the rock material down to the scour depth (as shown in Figure 5-1). However, these approaches are not suitable for this location due to the nature of the deep incised channel at the toe of the rock revetment and the shallow depth of the natural rock. It is therefore recommended that the toe of the rock revetment is keyed into existing natural rock outcrop capable of withstanding the scour. This can be achieved through thickening the toe, use of larger rocks in the toe, trenching into the natural rock material or a combination of these and other methods. Figure 5-2 shows an indicative sketch of the proposed rock revetment at the south corner of the eastern abutment.

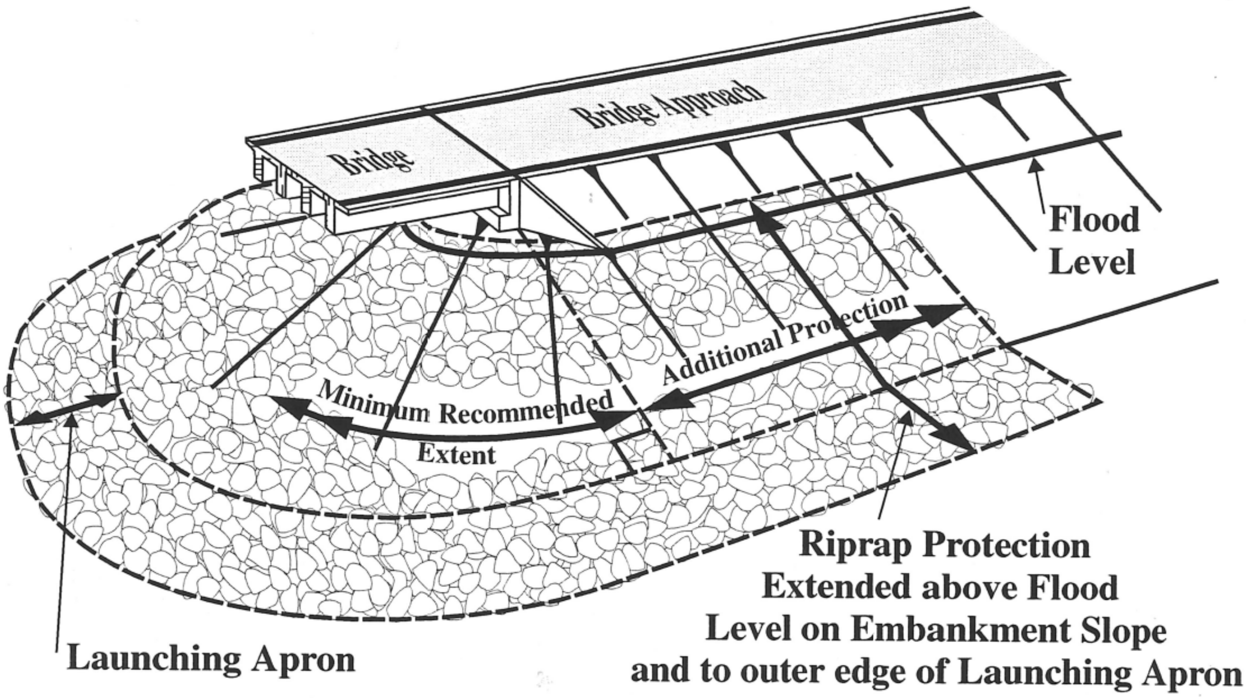


*Figure 5-1 Recommended practice for the placement of riprap protection at bridge abutments (Source: Fig 9.29 in Melville and Coleman (2000)).*

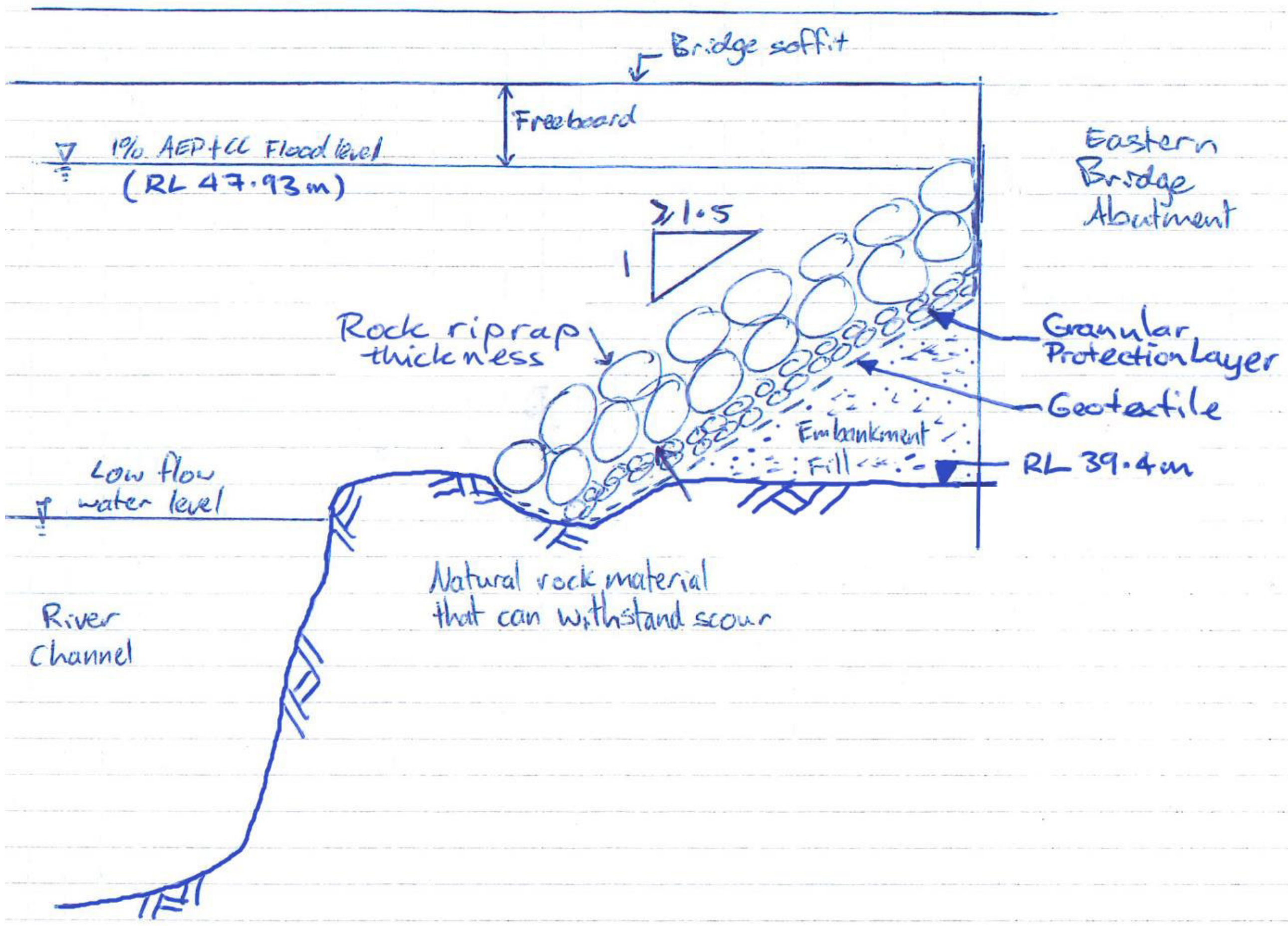


*Figure 5-2 Sketch of rock revetment design at south corner of eastern abutment*

### 5.2.5 Rock Grading Envelopes

Table 5-2 details the riprap armour layer grading. The rock grading has been specified to BS EN 13383:1-2002 based on design undertaken in accordance with The Rock Manual (CIRIA, 2007). The riprap grading selected is a "non-standard" Category A (facing) grading. Table 5-3 details the grading of the granular protection layer.

*Table 5-2 Riprap armour stone grading*

| CLASS DESIGNATION | CLASS LIMIT DEFINITION BY WEIGHT | | | | $M_{em}$ |
|---|---|---|---|---|---|
| | ELL | NLL | NUL | EUL | |
| Passing requirements | <2% | <10% | >70% | >97% | Lower to upper limit |
| Category A Light Mass Riprap | 80kg ($D_s$~390mm) | 170kg ($D_s$~500mm) | 650kg ($D_s$~780mm) | 1,040kg ($D_s$~910mm) | 300 to 420 kg ($D_s$ ~ 600 to 670mm) |

*Table 5-3 Granular protection layer grading*

| CLASS DESIGNATION | CLASS LIMIT DEFINITION BY WEIGHT | | | | $M_{em}$ |
|---|---|---|---|---|---|
| | ELL | NLL | NUL | EUL | |
| Passing requirements | <5% | <15% | >90% | >98% | <50% |
| Type 1 granular geotextile protection layer | 16mm | 30mm | 118mm | 156mm | 63mm |

# 6 Conclusions and Recommendations

In conclusion, the hydraulics at the site of the proposed SH8 Beaumont Bridge have been assessed. A HEC-RAS model was developed with survey data, a hydrological assessment, historical flood data and bridge drawings to provide a robust tool for assessing water levels and flow velocities at the bridge structure.

The water levels upstream of the proposed bridge structure for the SLS and ULS flood events are estimated to be 47.93m RL and 49.91m RL respectively.

The effects on the existing flood risk are shown to be minor, with a maximum increase in water level of 0.10m for the 1% AEP flood event including the effects of climate change out to 2120. The sensitivity of the design levels to different climate change scenarios were also presented and discussed. This shows that the approach adopted has limited risks.

The existing SH8 bridge structure has had an asset life of approximately 135 years with minimal scour. Like the existing bridge, the proposed bridge pier piles will generally be founded on rock that is either phyllite or metasandstone. The scour analysis shows that only the weaker phyllite rock has the potential for some scour, such as at Pier E. However, the proposed piles of Pier E are expected to be embedded deeper than the weaker phyllite rock into the harder metasandstone rock. Nevertheless, it is recommended that the potential for long-term scour (i.e. over many years) is considered during the design of the pier piles such that they can withstand some undercutting in areas of weaker rock material.

Both abutments will be founded in the metamorphic rock that does not have a significant risk of scour. The only risk to the structure is therefore that the road embankment and the material that this is placed on will be scoured away during the SLS event. This can be mitigated by the provision of suitably designed scour protection.

Scour protection will be provided by a rock revetment that will wrap around the full abutment on both banks in line with the abutment face to minimise risk of undermining. The facing riprap layer will be 1.2m thick and consist of rock with a median rock size (i.e. $D_{50}$) of 0.6m. It is recommended that the toe of the rock revetment is keyed into existing natural rock outcrop capable of withstanding the scour. This can be achieved through thickening the toe, use of larger rocks in the toe, trenching into the natural rock material or a combination of these and other methods.

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# Appendix A – Erodible Rock Scour Calculations

## SH8 Beaumont Bridge - Erodible Rock Scour - Critical Stream Power

*Ref: FHWA (2012)*

Approach flow for ULS event

| | | | | |
|---|---|---|---|---|
| WL | 49.91 | m RL | Water level | |
| Bed | 36.41 | m RL | Bed level | *(offset 70.07m of XS upstream of proposed bridge - in theory the most conservative location)* |
| $K_b$ | 1 | | Bend coefficient | (=1 for straight, Equation 4.4 otherwise) |
| ρ | 1000 | $kg/m^3$ | density of water | |
| ϒ | 9810 | $N/m^3$ | Unit weight of water | |
| y | 13.5 | m | Depth | |
| s | 0.0016 | | slope | *(slope taken from EG slope upstream of proposed bridge in HEC-RAS)* |

Approach shear stress:

τ = 208.7 $N/m^2$ (Equation 4.3)

Approach stream power:

$P_a$ = 0.75 $KW/m^2$ (Equation 7.39)

Determine scour depth

| | | | |
|---|---|---|---|
| b = | 2.1 | m | Pile diameter |
| $P_c$ = | 4.95 | $KW/m^2$ | (Original) |
| | 10.27 | $KW/m^3$ | (Foliation) |
| | 51.61 | $KW/m^4$ | (J1) |
| | 30.69 | $KW/m^5$ | (J2) |
| | 30.69 | $KW/m^6$ | (J3) |

The bend coefficient $K_b$ is used to calculate the increased shear stress on the outside of a bend. This coefficient ranges from 1.05 to 2.0, depending on the severity of the bend. The bend coefficient is a function of the radius of curvature $R_c$ divided by the top width of the channel T, as follows:

$$K_b = 2.0 \qquad \text{for } 2 \geq R_c/T$$

$$K_b = 2.38 - 0.206\left(\frac{R_c}{T}\right) + 0.0073\left(\frac{R_c}{T}\right)^2 \qquad \text{for } 10 > R_c/T > 2 \qquad (4.4)$$

$$K_b = 1.05 \qquad \text{for } R_c/T \geq 10$$

| ys/b | $y_s$ | $P/P_a$ | P | Stream Power greater than critical ($P > P_c$)? | | | | |
|---|---|---|---|---|---|---|---|---|
| | (m) | (Eq 7.40) | ($kW/m^2$) | Initial | Foliation | J1 | J2 | J3 |
| 0.01 | 0.02 | 8.36 | 6.26 | **yes** | no | no | no | no |
| 0.1 | 0.21 | 7.84 | 5.87 | **yes** | no | no | no | no |
| 0.2 | 0.42 | 7.30 | 5.47 | **yes** | no | no | no | no |
| 0.3 | 0.63 | 6.80 | 5.09 | **yes** | no | no | no | no |
| 0.4 | 0.84 | 6.33 | 4.74 | no | no | no | no | no |
| 0.5 | 1.05 | 5.90 | 4.42 | no | no | no | no | no |
| 0.6 | 1.26 | 5.49 | 4.11 | no | no | no | no | no |
| 0.7 | 1.47 | 5.12 | 3.83 | no | no | no | no | no |
| 0.8 | 1.68 | 4.76 | 3.57 | no | no | no | no | no |
| 0.9 | 1.89 | 4.44 | 3.32 | no | no | no | no | no |
| 1.0 | 2.10 | 4.13 | 3.09 | no | no | no | no | no |
| 1.1 | 2.31 | 3.85 | 2.88 | no | no | no | no | no |
| 1.2 | 2.52 | 3.58 | 2.68 | no | no | no | no | no |
| 1.3 | 2.73 | 3.34 | 2.50 | no | no | no | no | no |

**Results:**

- Only initial assessment based on conservative engineering assessmentshows up to 0.63m of scour
- Subsequent assessment of measured properties of 4 joints shows negligible scour

## SH8 Beaumont Bridge - Erodible Rock Scour - Critical Stream Power - Initial Assessment

*Ref: FHWA (2012)*

| | | | |
|---|---|---|---|
| UCS = | 17.5 | Mpa | Unconfined compressive strength |
| RQD = | 13 | | Rock Quality Designation |
| $J_n$ | 2.73 | | Assumed (Table 4.23) |
| $J_r$ | 1 | | Assumed (Table 4.24) |
| $J_a$ | 10 | | Assumed (Table 4.25) |

| | |
|---|---|
| 11 - 20 | phyllitte |
| 40 - 100 | meta sandstones |

| | | |
|---|---|---|
| $M_s$ = | 17.7 | (Table 4.22) |
| $K_b$ = | 4.761905 | (Equation 4.18) |
| $K_d$ = | 0.1 | (Equation 4.19) |
| $J_s$ = | 1 | Assumed (Table 4.26) |

Erodibility Index

| | | |
|---|---|---|
| K = | 8.428571 | (Equation 4.17) |

Critical stream power required to initiate scouring:

| | | |
|---|---|---|
| $P_c$ = | 4.947 KW/m$^2$ | (Equation 7.38) |

**Table 4.22. Values of the Rock Mass Strength Parameter $M_s$.**

| Hardness | Identification in Profile | Unconfined Compressive Strength (MPa) | Mass Strength Number (Ms) |
|---|---|---|---|
| Very soft rock | Material crumbles under firm (moderate) blows with sharp end of geological pick and | Less than 1.7 | 0.87 |
| | can be peeled off with a knife; is too hard to cut triaxial sample by hand. | 1.7 – 3.3 | 1.86 |
| Soft rock | Can just be scraped and peeled with a knife; indentations 1 mm to 3-mm show in the | 3.3 – 6.6 | 3.95 |
| | specimen with firm (moderate) blows of the pick point. | 6.6 – 13.2 | 8.39 |
| Hard rock | Cannot be scraped or peeled with a knife; hand-held specimen can be broken with hammer end of geological pick with a single firm (moderate) blow. | 13.2 – 26.4 | 17.70 |
| Very hard rock | Hand-held specimen breaks with hammer end of pick under more than one blow. | 26.4 – 53.0<br>53.00 – 106.0 | 35.0<br>70.0 |
| Extremely hard rock | Specimen requires many blows with geological pick to break through intact material. | Larger than 212.0 | 280.0 |

**Table 4.23. Rock Joint Set Number $J_n$.**

| Number of Joint Sets | Joint Set Number ($J_n$) |
|---|---|
| Intact, no or few joints/fissures | 1.00 |
| One joint/fissure set | 1.22 |
| One joint/fissure set plus random | 1.50 |
| Two joint/fissure sets | 1.83 |
| Two joint/fissure sets plus random | 2.24 |
| Three joint/fissure sets | 2.73 |
| Three joint/fissure sets plus random | 3.34 |
| Four joint/fissure sets | 4.09 |
| Multiple joint/fissure sets | 5.00 |

**Table 4.24. Joint Roughness Number $J_r$.**

| Condition of Joint | Joint Roughness Number $J_r$ |
|---|---|
| Stepped joints/fissures | 4.0 |
| Rough or irregular, undulating | 3.0 |
| Smooth undulating | 2.0 |
| Slickensided undulating | 1.5 |
| Rough or irregular, planar | 1.5 |
| Smooth planar | 1.0 |
| Slickensided planar | 0.5 |
| Joints/fissures either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation | 1.0 |
| Shattered or micro-shattered clays | 1.0 |

**Table 4.25. Joint Alteration Number $J_a$.**

| Description of Gouge | Joint Alteration Number ($J_a$) for Joint Separation (mm) | | |
|---|---|---|---|
| | 1.0 (1) | 1.0 –5.0(2) | 5.0(3) |
| Tightly healed, hard, non-softening impermeable filling | 0.75 | - | - |
| Unaltered joint walls, surface staining only | 1.0 | - | - |
| Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling | 2.0 | 2.0 | 4.0 |
| Non-softening, slightly clayey non-cohesive filling | 3.0 | 6.0 | 10.0 |
| Non-softening, strongly over-consolidated clay mineral filling, with or without crushed rock | 3.0 | 6.0** | 10.0 |
| Softening or low friction clay mineral coatings and small quantities of swelling clays | 4.0 | 8.0 | 13.0 |
| Softening moderately over-consolidated clay mineral filling, with or without crushed rock | 4.0 | 8.00** | 13.0 |
| Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock | 5.0 | 10.0** | 18.0 |

Note:
(1) Joint walls effectively in contact.
(2) Joint walls come into contact after approximately 100-mm shear.
(3) Joint walls do not come into contact at all upon shear.
**Also applies when crushed rock occurs in clay gouge without rock wall contact.

**Table 4.26. Relative Orientation Parameter $J_s$.**

| Dip Direction of Closer Spaced Joint Set (degrees) | Dip Angle of Closer Spaced Joint Set (degrees) | Ratio of Joint Spacing, r | | | |
|---|---|---|---|---|---|
| Dip Direction | Dip Angle | Ratio 1:1 | Ratio 1:2 | Ratio 1:4 | Ratio 1:8 |
| 180/0 | 90 | 1.14 | 1.20 | 1.24 | 1.26 |
| In direction of stream flow | 89 | 0.78 | 0.71 | 0.65 | 0.61 |
| In direction of stream flow | 85 | 0.73 | 0.66 | 0.61 | 0.57 |
| In direction of stream flow | 80 | 0.67 | 0.60 | 0.55 | 0.52 |
| In direction of stream flow | 70 | 0.56 | 0.50 | 0.46 | 0.43 |
| In direction of stream flow | 60 | 0.50 | 0.46 | 0.42 | 0.40 |
| In direction of stream flow | 50 | 0.49 | 0.46 | 0.43 | 0.41 |
| In direction of stream flow | 40 | 0.53 | 0.49 | 0.46 | 0.45 |
| In direction of stream flow | 30 | 0.63 | 0.59 | 0.55 | 0.53 |
| In direction of stream flow | 20 | 0.84 | 0.77 | 0.71 | 0.67 |
| In direction of stream flow | 10 | 1.25 | 1.10 | 0.98 | 0.90 |
| In direction of stream flow | 5 | 1.39 | 1.23 | 1.09 | 1.01 |
| In direction of stream flow | 1 | 1.50 | 1.33 | 1.19 | 1.10 |
| 0/180 | 0 | 1.14 | 1.09 | 1.05 | 1.02 |
| Against direction of stream flow | -1 | 0.78 | 0.85 | 0.90 | 0.94 |
| Against direction of stream flow | -5 | 0.73 | 0.79 | 0.84 | 0.88 |
| Against direction of stream flow | -10 | 0.67 | 0.72 | 0.78 | 0.81 |
| Against direction of stream flow | -20 | 0.56 | 0.62 | 0.66 | 0.69 |
| Against direction of stream flow | -30 | 0.50 | 0.55 | 0.58 | 0.60 |
| Against direction of stream flow | -40 | 0.49 | 0.52 | 0.55 | 0.57 |
| Against direction of stream flow | -50 | 0.53 | 0.56 | 0.59 | 0.61 |
| Against direction of stream flow | -60 | 0.63 | 0.68 | 0.71 | 0.73 |
| Against direction of stream flow | -70 | 0.84 | 0.91 | 0.97 | 1.01 |
| Against direction of stream flow | -80 | 1.26 | 1.41 | 1.53 | 1.61 |
| Against direction of stream flow | -85 | 1.39 | 1.55 | 1.69 | 1.77 |
| Against direction of stream flow | -89 | 1.50 | 1.68 | 1.82 | 1.91 |
| 180/0 | -90 | 1.14 | 1.20 | 1.24 | 1.26 |

Notes:
1. For intact material take $J_s$ = 1.0.
2. For values of r greater than 8 take $J_s$ as for r = 8.
3. If the flow direction FD is not in the direction of the true dip TD, the effective dip ED is determined by adding the ground slope to the apparent dip AD: ED = AD + GS

## SH8 Beaumont Bridge - Erodible Rock Scour - Critical Stream Power - Foliation Joint

*Ref: FHWA (2012)*

| | | | |
|---|---|---|---|
| UCS = | 17.5 | Mpa | Unconfined compressive strength |
| RQD = | 13 | | Measured |
| $J_n$ | 2.73 | | Assumed (Table 4.23) |
| $J_r$ | 1 | | Measured (Table 4.24) |
| $J_a$ | 2 | | Measured (Table 4.25) |

| | |
|---|---|
| 11 - 20 | phyllitte |
| 40 - 100 | meta sandstones |

$M_s$ = 17.7 (Table 4.22)

| | | |
|---|---|---|
| $K_b$ = | 4.761905 | (Equation 4.18) |
| $K_d$ = | 0.5 | (Equation 4.19) |
| $J_s$ = | 0.53 | Measured (Table 4.26) |
| | | Dip = 38°, Direction = 227° |

Erodibility Index

K = 22.33571 (Equation 4.17)

Critical stream power required to initiate scouring:

$P_c$ = 10.274 KW/m$^2$ (Equation 7.38)

Table 4.22. Values of the Rock Mass Strength Parameter $M_s$.

| Hardness | Identification in Profile | Unconfined Compressive Strength (MPa) | Mass Strength Number (Ms) |
|---|---|---|---|
| Very soft rock | Material crumbles under firm (moderate) blows with sharp end of geological pick and | Less than 1.7 | 0.87 |
| | can be peeled off with a knife; is too hard to cut triaxial sample by hand. | 1.7 – 3.3 | 1.86 |
| Soft rock | Can just be scraped and peeled with a knife; indentations 1 mm to 3-mm show in the | 3.3 – 6.6 | 3.95 |
| | specimen with firm (moderate) blows of the pick point. | 6.6 – 13.2 | 8.39 |
| Hard rock | Cannot be scraped or peeled with a knife; hand-held specimen can be broken with hammer end of geological pick with a single firm (moderate) blow. | 13.2 – 26.4 | 17.70 |
| Very hard rock | Hand-held specimen breaks with hammer end of pick under more than one blow. | 26.4 – 53.0<br>53.00 – 106.0 | 35.0<br>70.0 |
| Extremely hard rock | Specimen requires many blows with geological pick to break through intact material. | Larger than 212.0 | 280.0 |

Table 4.25. Joint Alteration Number $J_a$.

| Description of Gouge | Joint Alteration Number ($J_a$) for Joint Separation (mm) 1.0 [(1)] | 1.0 –5.0 [(2)] | 5.0 [(3)] |
|---|---|---|---|
| Tightly healed, hard, non-softening impermeable filling | 0.75 | - | - |
| Unaltered joint walls, surface staining only | 1.0 | - | - |
| Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling | 2.0 | 2.0 | 4.0 |
| Non-softening, slightly clayey non-cohesive filling | 3.0 | 6.0 | 10.0 |
| Non-softening, strongly over-consolidated clay mineral filling, with or without crushed rock | 3.0 | 6.0** | 10.0 |
| Softening or low friction clay mineral coatings and small quantities of swelling clays | 4.0 | 8.0 | 13.0 |
| Softening moderately over-consolidated clay mineral filling, with or without crushed rock | 4.0 | 8.00** | 13.0 |
| Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock | 5.0 | 10.0** | 18.0 |

Note:
(1) Joint walls effectively in contact.
(2) Joint walls come into contact after approximately 100-mm shear.
(3) Joint walls do not come into contact at all upon shear.
**Also applies when crushed rock occurs in clay gouge without rock wall contact.

Table 4.23. Rock Joint Set Number $J_n$.

| Number of Joint Sets | Joint Set Number ($J_n$) |
|---|---|
| Intact, no or few joints/fissures | 1.00 |
| One joint/fissure set | 1.22 |
| One joint/fissure set plus random | 1.50 |
| Two joint/fissure sets | 1.83 |
| Two joint/fissure sets plus random | 2.24 |
| Three joint/fissure sets | 2.73 |
| Three joint/fissure sets plus random | 3.34 |
| Four joint/fissure sets | 4.09 |
| Multiple joint/fissure sets | 5.00 |

Table 4.24. Joint Roughness Number $J_r$.

| Condition of Joint | Joint Roughness Number $J_r$ |
|---|---|
| Stepped joints/fissures | 4.0 |
| Rough or irregular, undulating | 3.0 |
| Smooth undulating | 2.0 |
| Slickensided undulating | 1.5 |
| Rough or irregular, planar | 1.5 |
| Smooth planar | 1.0 |
| Slickensided planar | 0.5 |
| Joints/fissures either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation | 1.0 |
| Shattered or micro-shattered clays | 1.0 |

Table 4.26. Relative Orientation Parameter $J_s$.

| Dip Direction of Closer Spaced Joint Set (degrees) | Dip Angle of Closer Spaced Joint Set (degrees) | Ratio of Joint Spacing, r | | | |
|---|---|---|---|---|---|
| Dip Direction | Dip Angle | Ratio 1:1 | Ratio 1:2 | Ratio 1:4 | Ratio 1:8 |
| 180/0 | 90 | 1.14 | 1.20 | 1.24 | 1.26 |
| In direction of stream flow | 89 | 0.78 | 0.71 | 0.65 | 0.61 |
| In direction of stream flow | 85 | 0.73 | 0.66 | 0.61 | 0.57 |
| In direction of stream flow | 80 | 0.67 | 0.60 | 0.55 | 0.52 |
| In direction of stream flow | 70 | 0.56 | 0.50 | 0.46 | 0.43 |
| In direction of stream flow | 60 | 0.50 | 0.46 | 0.42 | 0.40 |
| In direction of stream flow | 50 | 0.49 | 0.46 | 0.43 | 0.41 |
| In direction of stream flow | 40 | 0.53 | 0.49 | 0.46 | 0.45 |
| In direction of stream flow | 30 | 0.63 | 0.59 | 0.55 | 0.53 |
| In direction of stream flow | 20 | 0.84 | 0.77 | 0.71 | 0.67 |
| In direction of stream flow | 10 | 1.25 | 1.10 | 0.98 | 0.90 |
| In direction of stream flow | 5 | 1.39 | 1.23 | 1.09 | 1.01 |
| In direction of stream flow | 1 | 1.50 | 1.33 | 1.19 | 1.10 |
| 0/180 | 0 | 1.14 | 1.09 | 1.05 | 1.02 |
| Against direction of stream flow | -1 | 0.78 | 0.85 | 0.90 | 0.94 |
| Against direction of stream flow | -5 | 0.73 | 0.79 | 0.84 | 0.88 |
| Against direction of stream flow | -10 | 0.67 | 0.72 | 0.78 | 0.81 |
| Against direction of stream flow | -20 | 0.56 | 0.62 | 0.66 | 0.69 |
| Against direction of stream flow | -30 | 0.50 | 0.55 | 0.58 | 0.60 |
| Against direction of stream flow | -40 | 0.49 | 0.52 | 0.55 | 0.57 |
| Against direction of stream flow | -50 | 0.53 | 0.56 | 0.59 | 0.61 |
| Against direction of stream flow | -60 | 0.63 | 0.68 | 0.71 | 0.73 |
| Against direction of stream flow | -70 | 0.84 | 0.91 | 0.97 | 1.01 |
| Against direction of stream flow | -80 | 1.26 | 1.41 | 1.53 | 1.61 |
| Against direction of stream flow | -85 | 1.39 | 1.55 | 1.69 | 1.77 |
| Against direction of stream flow | -89 | 1.50 | 1.68 | 1.82 | 1.91 |
| 180/0 | -90 | 1.14 | 1.20 | 1.24 | 1.26 |

Notes:
1. For intact material take $J_s$ = 1.0.
2. For values of r greater than 8 take $J_s$ as for r = 8.
3. If the flow direction FD is not in the direction of the true dip TD, the effective dip ED is determined by adding the ground slope to the apparent dip AD: ED = AD + GS

## SH8 Beaumont Bridge - Erodible Rock Scour - Critical Stream Power - Joint J1

*Ref: FHWA (2012)*

| | | | |
|---|---|---|---|
| UCS = | 17.5 | Mpa | Unconfined compressive strength |
| RQD = | 13 | | Measured |
| $J_n$ | 2.73 | | Assumed (Table 4.23) |
| $J_r$ | 2 | | Measured (Table 4.24) |
| $J_a$ | 1 | | Measured (Table 4.25) |

| | |
|---|---|
| 11 - 20 | phyllitte |
| 40 - 100 | meta sandstones |

| | | |
|---|---|---|
| $M_s$ = | 17.7 | (Table 4.22) |
| $K_b$ = | 4.761905 | (Equation 4.18) |
| $K_d$ = | 2 | (Equation 4.19) |
| $J_s$ = | 1.14 | Measured (Table 4.26) |
| | | Dip = 87°, Direction = 110° |

Erodibility Index

K = 192.1714 (Equation 4.17)

Critical stream power required to initiate scouring:

$P_c$ = 51.614 KW/m$^2$ (Equation 7.38)

Table 4.22. Values of the Rock Mass Strength Parameter $M_s$.

| Hardness | Identification in Profile | Unconfined Compressive Strength (MPa) | Mass Strength Number (Ms) |
|---|---|---|---|
| Very soft rock | Material crumbles under firm (moderate) blows with sharp end of geological pick and | Less than 1.7 | 0.87 |
| | can be peeled off with a knife; is too hard to cut triaxial sample by hand. | 1.7 – 3.3 | 1.86 |
| Soft rock | Can just be scraped and peeled with a knife; indentations 1 mm to 3-mm show in the | 3.3 – 6.6 | 3.95 |
| | specimen with firm (moderate) blows of the pick point. | 6.6 – 13.2 | 8.39 |
| Hard rock | Cannot be scraped or peeled with a knife; hand-held specimen can be broken with hammer end of geological pick with a single firm (moderate) blow. | 13.2 – 26.4 | 17.70 |
| Very hard rock | Hand-held specimen breaks with hammer end of pick under more than one blow. | 26.4 – 53.0<br>53.00 – 106.0 | 35.0<br>70.0 |
| Extremely hard rock | Specimen requires many blows with geological pick to break through intact material. | Larger than 212.0 | 280.0 |

Table 4.23. Rock Joint Set Number $J_n$.

| Number of Joint Sets | Joint Set Number ($J_n$) |
|---|---|
| Intact, no or few joints/fissures | 1.00 |
| One joint/fissure set | 1.22 |
| One joint/fissure set plus random | 1.50 |
| Two joint/fissure sets | 1.83 |
| Two joint/fissure sets plus random | 2.24 |
| Three joint/fissure sets | 2.73 |
| Three joint/fissure sets plus random | 3.34 |
| Four joint/fissure sets | 4.09 |
| Multiple joint/fissure sets | 5.00 |

Table 4.24. Joint Roughness Number $J_r$.

| Condition of Joint | Joint Roughness Number $J_r$ |
|---|---|
| Stepped joints/fissures | 4.0 |
| Rough or irregular, undulating | 3.0 |
| Smooth undulating | 2.0 |
| Slickensided undulating | 1.5 |
| Rough or irregular, planar | 1.5 |
| Smooth planar | 1.0 |
| Slickensided planar | 0.5 |
| Joints/fissures either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation | 1.0 |
| Shattered or micro-shattered clays | 1.0 |

Table 4.25. Joint Alteration Number $J_a$.

| Description of Gouge | Joint Alteration Number ($J_a$) for Joint Separation (mm) | | |
|---|---|---|---|
| | 1.0 (1) | 1.0 –5.0 (2) | 5.0 (3) |
| Tightly healed, hard, non-softening impermeable filling | 0.75 | - | - |
| Unaltered joint walls, surface staining only | 1.0 | - | - |
| Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling | 2.0 | 2.0 | 4.0 |
| Non-softening, slightly clayey non-cohesive filling | 3.0 | 6.0 | 10.0 |
| Non-softening, strongly over-consolidated clay mineral filling, with or without crushed rock | 3.0 | 6.0** | 10.0 |
| Softening or low friction clay mineral coatings and small quantities of swelling clays | 4.0 | 8.0 | 13.0 |
| Softening moderately over-consolidated clay mineral filling, with or without crushed rock | 4.0 | 8.00** | 13.0 |
| Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock | 5.0 | 10.0** | 18.0 |

Note:
(1) Joint walls effectively in contact.
(2) Joint walls come into contact after approximately 100-mm shear.
(3) Joint walls do not come into contact at all upon shear.
**Also applies when crushed rock occurs in clay gouge without rock wall contact.

Table 4.26. Relative Orientation Parameter $J_s$.

| Dip Direction of Closer Spaced Joint Set (degrees) | Dip Angle of Closer Spaced Joint Set (degrees) | Ratio of Joint Spacing, r | | | |
|---|---|---|---|---|---|
| Dip Direction | Dip Angle | Ratio 1:1 | Ratio 1:2 | Ratio 1:4 | Ratio 1:8 |
| 180/0 | 90 | 1.14 | 1.20 | 1.24 | 1.26 |
| In direction of stream flow | 89 | 0.78 | 0.71 | 0.65 | 0.61 |
| In direction of stream flow | 85 | 0.73 | 0.66 | 0.61 | 0.57 |
| In direction of stream flow | 80 | 0.67 | 0.60 | 0.55 | 0.52 |
| In direction of stream flow | 70 | 0.56 | 0.50 | 0.46 | 0.43 |
| In direction of stream flow | 60 | 0.50 | 0.46 | 0.42 | 0.40 |
| In direction of stream flow | 50 | 0.49 | 0.46 | 0.43 | 0.41 |
| In direction of stream flow | 40 | 0.53 | 0.49 | 0.46 | 0.45 |
| In direction of stream flow | 30 | 0.63 | 0.59 | 0.55 | 0.53 |
| In direction of stream flow | 20 | 0.84 | 0.77 | 0.71 | 0.67 |
| In direction of stream flow | 10 | 1.25 | 1.10 | 0.98 | 0.90 |
| In direction of stream flow | 5 | 1.39 | 1.23 | 1.09 | 1.01 |
| In direction of stream flow | 1 | 1.50 | 1.33 | 1.19 | 1.10 |
| 0/180 | 0 | 1.14 | 1.09 | 1.05 | 1.02 |
| Against direction of stream flow | -1 | 0.78 | 0.85 | 0.90 | 0.94 |
| Against direction of stream flow | -5 | 0.73 | 0.79 | 0.84 | 0.88 |
| Against direction of stream flow | -10 | 0.67 | 0.72 | 0.78 | 0.81 |
| Against direction of stream flow | -20 | 0.56 | 0.62 | 0.66 | 0.69 |
| Against direction of stream flow | -30 | 0.50 | 0.55 | 0.58 | 0.60 |
| Against direction of stream flow | -40 | 0.49 | 0.52 | 0.55 | 0.57 |
| Against direction of stream flow | -50 | 0.53 | 0.56 | 0.59 | 0.61 |
| Against direction of stream flow | -60 | 0.63 | 0.68 | 0.71 | 0.73 |
| Against direction of stream flow | -70 | 0.84 | 0.91 | 0.97 | 1.01 |
| Against direction of stream flow | -80 | 1.26 | 1.41 | 1.53 | 1.61 |
| Against direction of stream flow | -85 | 1.39 | 1.55 | 1.69 | 1.77 |
| Against direction of stream flow | -89 | 1.50 | 1.68 | 1.82 | 1.91 |
| 180/0 | -90 | 1.14 | 1.20 | 1.24 | 1.26 |

Notes:
1. For intact material take $J_s$ = 1.0.
2. For values of r greater than 8 take $J_s$ as for r = 8.
3. If the flow direction FD is not in the direction of the true dip TD, the effective dip ED is determined by adding the ground slope to the apparent dip AD: ED = AD + GS

## SH8 Beaumont Bridge - Erodible Rock Scour - Critical Stream Power - Joint J2

*Ref: FHWA (2012)*

| | | | |
|---|---|---|---|
| UCS = | 17.5 | Mpa | Unconfined compressive strength |
| RQD = | 13 | | Measured |
| $J_n$ | 2.73 | | Assumed (Table 4.23) |
| $J_r$ | 2 | | Measured (Table 4.24) |
| $J_a$ | 2 | | Measured (Table 4.25) |

| | |
|---|---|
| 11 - 20 | phyllitte |
| 40 - 100 | meta sandstones |

| | | |
|---|---|---|
| $M_s$ = | 17.7 | (Table 4.22) |
| $K_b$ = | 4.761905 | (Equation 4.18) |
| $K_d$ = | 1 | (Equation 4.19) |
| $J_s$ = | 1.14 | Measured (Table 4.26) |
| | | Dip = 73°, Direction = 006° |

Erodibility Index

| | | |
|---|---|---|
| K = | 96.08571 | (Equation 4.17) |

Critical stream power required to initiate scouring:

| | | |
|---|---|---|
| $P_c$ = | 30.690 $KW/m^2$ | (Equation 7.38) |

Table 4.22. Values of the Rock Mass Strength Parameter $M_s$.

| Hardness | Identification in Profile | Unconfined Compressive Strength (MPa) | Mass Strength Number (Ms) |
|---|---|---|---|
| Very soft rock | Material crumbles under firm (moderate) blows with sharp end of geological pick and | Less than 1.7 | 0.87 |
| | can be peeled off with a knife; is too hard to cut triaxial sample by hand. | 1.7 – 3.3 | 1.86 |
| Soft rock | Can just be scraped and peeled with a knife; indentations 1 mm to 3-mm show in the | 3.3 – 6.6 | 3.95 |
| | specimen with firm (moderate) blows of the pick point. | 6.6 – 13.2 | 8.39 |
| Hard rock | Cannot be scraped or peeled with a knife; hand-held specimen can be broken with hammer end of geological pick with a single firm (moderate) blow. | 13.2 – 26.4 | 17.70 |
| Very hard rock | Hand-held specimen breaks with hammer end of pick under more than one blow. | 26.4 – 53.0<br>53.00 – 106.0 | 35.0<br>70.0 |
| Extremely hard rock | Specimen requires many blows with geological pick to break through intact material. | Larger than 212.0 | 280.0 |

Table 4.23. Rock Joint Set Number $J_n$.

| Number of Joint Sets | Joint Set Number ($J_n$) |
|---|---|
| Intact, no or few joints/fissures | 1.00 |
| One joint/fissure set | 1.22 |
| One joint/fissure set plus random | 1.50 |
| Two joint/fissure sets | 1.83 |
| Two joint/fissure sets plus random | 2.24 |
| Three joint/fissure sets | 2.73 |
| Three joint/fissure sets plus random | 3.34 |
| Four joint/fissure sets | 4.09 |
| Multiple joint/fissure sets | 5.00 |

Table 4.24. Joint Roughness Number $J_r$.

| Condition of Joint | Joint Roughness Number $J_r$ |
|---|---|
| Stepped joints/fissures | 4.0 |
| Rough or irregular, undulating | 3.0 |
| Smooth undulating | 2.0 |
| Slickensided undulating | 1.5 |
| Rough or irregular, planar | 1.5 |
| Smooth planar | 1.0 |
| Slickensided planar | 0.5 |
| Joints/fissures either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation | 1.0 |
| Shattered or micro-shattered clays | 1.0 |

Table 4.25. Joint Alteration Number $J_a$.

| Description of Gouge | Joint Alteration Number ($J_a$) for Joint Separation (mm) | | |
|---|---|---|---|
| | 1.0 [(1)] | 1.0 –5.0 [(2)] | 5.0 [(3)] |
| Tightly healed, hard, non-softening impermeable filling | 0.75 | - | - |
| Unaltered joint walls, surface staining only | 1.0 | - | - |
| Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling | 2.0 | 2.0 | 4.0 |
| Non-softening, slightly clayey non-cohesive filling | 3.0 | 6.0 | 10.0 |
| Non-softening, strongly over-consolidated clay mineral filling, with or without crushed rock | 3.0 | 6.0** | 10.0 |
| Softening or low friction clay mineral coatings and small quantities of swelling clays | 4.0 | 8.0 | 13.0 |
| Softening moderately over-consolidated clay mineral filling, with or without crushed rock | 4.0 | 8.00** | 13.0 |
| Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock | 5.0 | 10.0** | 18.0 |

Note:
(1) Joint walls effectively in contact.
(2) Joint walls come into contact after approximately 100-mm shear.
(3) Joint walls do not come into contact at all upon shear.
**Also applies when crushed rock occurs in clay gouge without rock wall contact.

Table 4.26. Relative Orientation Parameter $J_s$.

| Dip Direction of Closer Spaced Joint Set (degrees) | Dip Angle of Closer Spaced Joint Set (degrees) | Ratio of Joint Spacing, r | | | |
|---|---|---|---|---|---|
| Dip Direction | Dip Angle | Ratio 1:1 | Ratio 1:2 | Ratio 1:4 | Ratio 1:8 |
| 180/0 | 90 | 1.14 | 1.20 | 1.24 | 1.26 |
| In direction of stream flow | 89 | 0.78 | 0.71 | 0.65 | 0.61 |
| In direction of stream flow | 85 | 0.73 | 0.66 | 0.61 | 0.57 |
| In direction of stream flow | 80 | 0.67 | 0.60 | 0.55 | 0.52 |
| In direction of stream flow | 70 | 0.56 | 0.50 | 0.46 | 0.43 |
| In direction of stream flow | 60 | 0.50 | 0.46 | 0.42 | 0.40 |
| In direction of stream flow | 50 | 0.49 | 0.46 | 0.43 | 0.41 |
| In direction of stream flow | 40 | 0.53 | 0.49 | 0.46 | 0.45 |
| In direction of stream flow | 30 | 0.63 | 0.59 | 0.55 | 0.53 |
| In direction of stream flow | 20 | 0.84 | 0.77 | 0.71 | 0.67 |
| In direction of stream flow | 10 | 1.25 | 1.10 | 0.98 | 0.90 |
| In direction of stream flow | 5 | 1.39 | 1.23 | 1.09 | 1.01 |
| In direction of stream flow | 1 | 1.50 | 1.33 | 1.19 | 1.10 |
| 0/180 | 0 | 1.14 | 1.09 | 1.05 | 1.02 |
| Against direction of stream flow | -1 | 0.78 | 0.85 | 0.90 | 0.94 |
| Against direction of stream flow | -5 | 0.73 | 0.79 | 0.84 | 0.88 |
| Against direction of stream flow | -10 | 0.67 | 0.72 | 0.78 | 0.81 |
| Against direction of stream flow | -20 | 0.56 | 0.62 | 0.66 | 0.69 |
| Against direction of stream flow | -30 | 0.50 | 0.55 | 0.58 | 0.60 |
| Against direction of stream flow | -40 | 0.49 | 0.52 | 0.55 | 0.57 |
| Against direction of stream flow | -50 | 0.53 | 0.56 | 0.59 | 0.61 |
| Against direction of stream flow | -60 | 0.63 | 0.68 | 0.71 | 0.73 |
| Against direction of stream flow | -70 | 0.84 | 0.91 | 0.97 | 1.01 |
| Against direction of stream flow | -80 | 1.26 | 1.41 | 1.53 | 1.61 |
| Against direction of stream flow | -85 | 1.39 | 1.55 | 1.69 | 1.77 |
| Against direction of stream flow | -89 | 1.50 | 1.68 | 1.82 | 1.91 |
| 180/0 | -90 | 1.14 | 1.20 | 1.24 | 1.26 |

Notes:
1. For intact material take $J_s$ = 1.0.
2. For values of r greater than 8 take $J_s$ as for r = 8.
3. If the flow direction FD is not in the direction of the true dip TD, the effective dip ED is determined by adding the ground slope to the apparent dip AD: ED = AD + GS

## SH8 Beaumont Bridge - Erodible Rock Scour - Critical Stream Power - Joint J3

*Ref: FHWA (2012)*

| | | | |
|---|---|---|---|
| UCS = | 17.5 | Mpa | Unconfined compressive strength |
| RQD = | 13 | | Measured |
| $J_n$ | 2.73 | | Assumed (Table 4.23) |
| $J_r$ | 2 | | Measured (Table 4.24) |
| $J_a$ | 2 | | Measured (Table 4.25) |

| | |
|---|---|
| 11 - 20 | phyllitte |
| 40 - 100 | meta sandstones |

| | | |
|---|---|---|
| $M_s$ = | 17.7 | (Table 4.22) |
| $K_b$ = | 4.761905 | (Equation 4.18) |
| $K_d$ = | 1 | (Equation 4.19) |
| $J_s$ = | 1.14 | Measured (Table 4.26) |
| | | Dip = 65°, Direction = 048° |

Erodibility Index

K = 96.08571 (Equation 4.17)

Critical stream power required to initiate scouring:

$P_c$ = 30.690 KW/m$^2$ (Equation 7.38)

**Table 4.22. Values of the Rock Mass Strength Parameter $M_s$.**

| Hardness | Identification in Profile | Unconfined Compressive Strength (MPa) | Mass Strength Number (Ms) |
|---|---|---|---|
| Very soft rock | Material crumbles under firm (moderate) blows with sharp end of geological pick and | Less than 1.7 | 0.87 |
| | can be peeled off with a knife; is too hard to cut triaxial sample by hand. | 1.7 – 3.3 | 1.86 |
| Soft rock | Can just be scraped and peeled with a knife; indentations 1 mm to 3-mm show in the | 3.3 – 6.6 | 3.95 |
| | specimen with firm (moderate) blows of the pick point. | 6.6 – 13.2 | 8.39 |
| Hard rock | Cannot be scraped or peeled with a knife; hand-held specimen can be broken with hammer end of geological pick with a single firm (moderate) blow. | 13.2 – 26.4 | 17.70 |
| Very hard rock | Hand-held specimen breaks with hammer end of pick under more than one blow. | 26.4 – 53.0<br>53.00 – 106.0 | 35.0<br>70.0 |
| Extremely hard rock | Specimen requires many blows with geological pick to break through intact material. | Larger than 212.0 | 280.0 |

**Table 4.25. Joint Alteration Number $J_a$.**

| Description of Gouge | Joint Alteration Number ($J_a$) for Joint Separation (mm) | | |
|---|---|---|---|
| | 1.0 (1) | 1.0 –5.0(2) | 5.0(3) |
| Tightly healed, hard, non-softening impermeable filling | 0.75 | - | - |
| Unaltered joint walls, surface staining only | 1.0 | - | - |
| Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling | 2.0 | 2.0 | 4.0 |
| Non-softening, slightly clayey non-cohesive filling | 3.0 | 6.0 | 10.0 |
| Non-softening, strongly over-consolidated clay mineral filling, with or without crushed rock | 3.0 | 6.0** | 10.0 |
| Softening or low friction clay mineral coatings and small quantities of swelling clays | 4.0 | 8.0 | 13.0 |
| Softening moderately over-consolidated clay mineral filling, with or without crushed rock | 4.0 | 8.00** | 13.0 |
| Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock | 5.0 | 10.0** | 18.0 |

Note:
(1) Joint walls effectively in contact.
(2) Joint walls come into contact after approximately 100-mm shear.
(3) Joint walls do not come into contact at all upon shear.
**Also applies when crushed rock occurs in clay gouge without rock wall contact.

**Table 4.23. Rock Joint Set Number $J_n$.**

| Number of Joint Sets | Joint Set Number ($J_n$) |
|---|---|
| Intact, no or few joints/fissures | 1.00 |
| One joint/fissure set | 1.22 |
| One joint/fissure set plus random | 1.50 |
| Two joint/fissure sets | 1.83 |
| Two joint/fissure sets plus random | 2.24 |
| Three joint/fissure sets | 2.73 |
| Three joint/fissure sets plus random | 3.34 |
| Four joint/fissure sets | 4.09 |
| Multiple joint/fissure sets | 5.00 |

**Table 4.24. Joint Roughness Number $J_r$.**

| Condition of Joint | Joint Roughness Number $J_r$ |
|---|---|
| Stepped joints/fissures | 4.0 |
| Rough or irregular, undulating | 3.0 |
| Smooth undulating | 2.0 |
| Slickensided undulating | 1.5 |
| Rough or irregular, planar | 1.5 |
| Smooth planar | 1.0 |
| Slickensided planar | 0.5 |
| Joints/fissures either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation | 1.0 |
| Shattered or micro-shattered clays | 1.0 |

**Table 4.26. Relative Orientation Parameter $J_s$.**

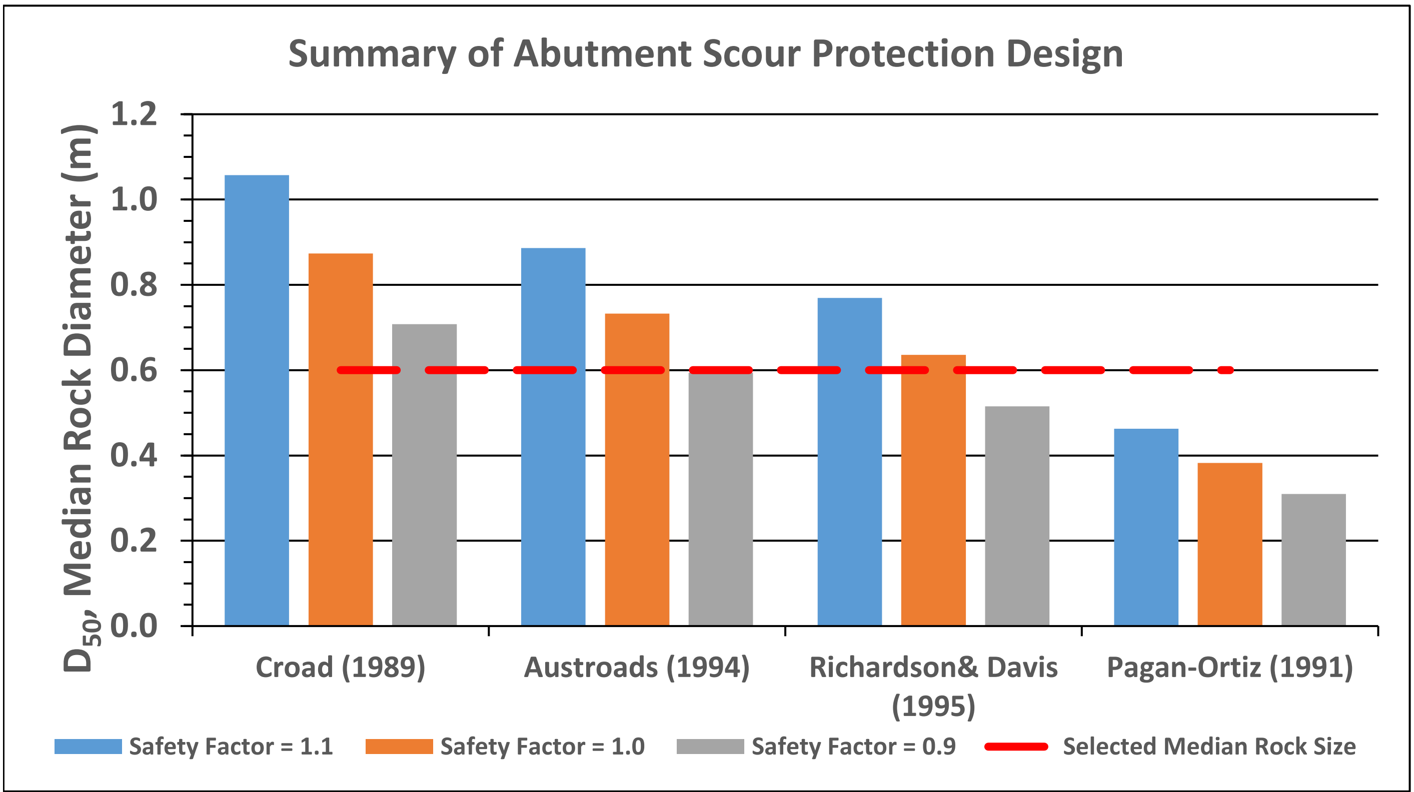
| Dip Direction of Closer Spaced Joint Set (degrees) | Dip Angle of Closer Spaced Joint Set (degrees) | Ratio of Joint Spacing, r | | | |
|---|---|---|---|---|---|
| Dip Direction | Dip Angle | Ratio 1:1 | Ratio 1:2 | Ratio 1:4 | Ratio 1:8 |
| 180/0 | 90 | 1.14 | 1.20 | 1.24 | 1.26 |
| In direction of stream flow | 89 | 0.78 | 0.71 | 0.65 | 0.61 |
| In direction of stream flow | 85 | 0.73 | 0.66 | 0.61 | 0.57 |
| In direction of stream flow | 80 | 0.67 | 0.60 | 0.55 | 0.52 |
| In direction of stream flow | 70 | 0.56 | 0.50 | 0.46 | 0.43 |
| In direction of stream flow | 60 | 0.50 | 0.46 | 0.42 | 0.40 |
| In direction of stream flow | 50 | 0.49 | 0.46 | 0.43 | 0.41 |
| In direction of stream flow | 40 | 0.53 | 0.49 | 0.46 | 0.45 |
| In direction of stream flow | 30 | 0.63 | 0.59 | 0.55 | 0.53 |
| In direction of stream flow | 20 | 0.84 | 0.77 | 0.71 | 0.67 |
| In direction of stream flow | 10 | 1.25 | 1.10 | 0.98 | 0.90 |
| In direction of stream flow | 5 | 1.39 | 1.23 | 1.09 | 1.01 |
| In direction of stream flow | 1 | 1.50 | 1.33 | 1.19 | 1.10 |
| 0/180 | 0 | 1.14 | 1.09 | 1.05 | 1.02 |
| Against direction of stream flow | -1 | 0.78 | 0.85 | 0.90 | 0.94 |
| Against direction of stream flow | -5 | 0.73 | 0.79 | 0.84 | 0.88 |
| Against direction of stream flow | -10 | 0.67 | 0.72 | 0.78 | 0.81 |
| Against direction of stream flow | -20 | 0.56 | 0.62 | 0.66 | 0.69 |
| Against direction of stream flow | -30 | 0.50 | 0.55 | 0.58 | 0.60 |
| Against direction of stream flow | -40 | 0.49 | 0.52 | 0.55 | 0.57 |
| Against direction of stream flow | -50 | 0.53 | 0.56 | 0.59 | 0.61 |
| Against direction of stream flow | -60 | 0.63 | 0.68 | 0.71 | 0.73 |
| Against direction of stream flow | -70 | 0.84 | 0.91 | 0.97 | 1.01 |
| Against direction of stream flow | -80 | 1.26 | 1.41 | 1.53 | 1.61 |
| Against direction of stream flow | -85 | 1.39 | 1.55 | 1.69 | 1.77 |
| Against direction of stream flow | -89 | 1.50 | 1.68 | 1.82 | 1.91 |
| 180/0 | -90 | 1.14 | 1.20 | 1.24 | 1.26 |

Notes:
1. For intact material take $J_s$ = 1.0.
2. For values of r greater than 8 take $J_s$ as for r = 8.
3. If the flow direction FD is not in the direction of the true dip TD, the effective dip ED is determined by adding the ground slope to the apparent dip AD: ED = AD + GS

# Appendix B – Abutment Scour Protection Calculations

| | | | |
|---|---|---|---|
| Project: | SH8 Beaumont Bridge (Clutha River) | Date: | 13/06/2019 |
| Project No: | 6-CT102.00 | | |
| Created by | Daniel McMullan, Franciscus Maas | | |

| Safety Factor | Method | Croad (1989) | Austroads (1994) | Richardson& Davis (1995) | Pagan-Ortiz (1991) | Average | Max |
|---|---|---|---|---|---|---|---|
| 1.1 | $d_{r50}$ | 1.057 | 0.887 | 0.769 | 0.462 | **0.972** | **1.057** |
| 1 | $d_{r50}$ | 0.874 | 0.733 | 0.636 | 0.382 | **0.803** | **0.874** |
| 0.9 | $d_{r50}$ | 0.708 | 0.594 | 0.515 | 0.309 | **0.651** | **0.708** |



| | | | |
|---|---|---|---|
| Project: | SH8 Beaumont Bridge (Clutha River) | Date: | 13/06/2019 |
| Project No: | 6-CT102.00 | | |
| Created by | Daniel McMullan, Franciscus Maas | | |
| Method: | Croad (1989) | | |

**Input data:**

| | | | |
|---|---|---|---|
| g | 9.81 | $m/s^2$ | |
| $S_s$ | 2.65 | | specific gravity of rock |
| $V_{max}$ | 3.40 | m/s | Flow velocity |
| y | 8.50 | m | Water depth at toe |

$$\frac{d_{r50}}{y} = \frac{0.91}{(S_s - 1)K_{sl}} Fr^2$$

$$Fr = \frac{V}{\sqrt{gy}}$$

| | | |
|---|---|---|
| α | 26.5651 | Slope angle |
| β | 42 | Angle of repose of riprap stone |
| $K_{sl}$ | 0.7438 | embankment slop factor |

$$K_{sl} = \sqrt{1 - \frac{\sin^2 \alpha}{\sin^2 \theta}}$$

**Output data:**

| Safety Factor | $V_b$ | Fr | $dr_{50}$ |
|---|---|---|---|
| 1.1 | 3.74 | 0.41 | 1.06 |
| 1 | 3.40 | 0.37 | 0.87 |
| 0.9 | 3.06 | 0.34 | 0.71 |

| | | | |
|---|---|---|---|
| Project: | SH8 Beaumont Bridge (Clutha River) | Date: | 13/06/2019 |
| Project No: | 6-CT102.00 | | |

| | |
|---|---|
| Created by | Daniel McMullan, Franciscus Maas |
| Method: | Austroads (1994) |

**Input data:**

| | | | |
|---|---|---|---|
| $V_{max}$ | 3.40 | m/s | Flow velocity |
| y | 8.50 | m | Water depth at toe |
| g | 9.81 | $m/s^2$ | |
| $S_s$ | 2.65 | Specific gravity of rock | |

$$\frac{d_{r50}}{y} = \frac{1.026}{(S_s - 1)} Fr^2$$

**Output data:**

| Safety Factor | $V_b$ | Fr | $dr_{50}$ |
|---|---|---|---|
| 1.1 | 3.74 | 0.41 | 0.89 |
| 1 | 3.4 | 0.37 | 0.73 |
| 0.9 | 3.06 | 0.34 | 0.59 |

| | | | |
|---|---|---|---|
| Project: | SH8 Beaumont Bridge (Clutha River) | Date: | 13/06/2019 |
| Project No: | 6-CT102.00 | | |

| | |
|---|---|
| Created by | Daniel McMullan, Franciscus Maas |
| Method: | Richardson& Davis (1995) |

**Input data:**

| | | | |
|---|---|---|---|
| $V_{max}$ | 3.40 | m/s | Flow velocity |
| y | 8.50 | m | Water depth at toe |
| g | 9.81 | $m/s^2$ | |
| $S_s$ | 2.65 | Specific gravity of rock | |
| Ks | 0.89 | Shape Factor | |

$$\frac{d_{r50}}{y_2} = \frac{K_s}{(S_s - 1)} Fr_2^{\ 2}$$

$K_s$ = shape factor
= 0.89 for spill-through abutments
= 1.02 for vertical wall abutments

**Output data:**

| Safety Factor | $V_b$ | Fr | $dr_{50}$ |
|---|---|---|---|
| 1.1 | 3.74 | 0.41 | 0.77 |
| 1 | 3.40 | 0.37 | 0.64 |
| 0.9 | 3.06 | 0.34 | 0.51 |

| | | | |
|---|---|---|---|
| Project: | SH8 Beaumont Bridge (Clutha River) | Date: | 13/06/2019 |
| Project No: | 6-CT102.00 | | |

| | |
|---|---|
| Created by | Daniel McMullan, Franciscus Maas |
| Method: | Pagan-Ortiz (1991) |

**Input data:**

| | | | |
|---|---|---|---|
| $V_{max}$ | 3.40 | m/s | Flow velocity |
| y | 8.50 | m | Water depth at toe |
| g | 9.81 | $m/s^2$ | |
| $S_s$ | 2.65 | Specific gravity of rock | |

**Spill-through abutment:**

$$\frac{d_{r50}}{y_2} = \frac{0.535}{(S_s - 1)} Fr_2^{\ 2}$$

**Output data:**

| Safety Factor | $V_b$ | Fr | $dr_{50}$ |
|---|---|---|---|
| 1.1 | 3.74 | 0.41 | 0.46 |
| 1 | 3.40 | 0.37 | 0.38 |
| 0.9 | 3.06 | 0.34 | 0.31 |