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6 December 2018

[redacted]

Dear [redacted]

Local Government Official Information and Meetings Act 1987 (LGOIMA) request for the completed flood modelling data

I am writing in response to your request received 23 November 2018 requesting the completed flood modelling data referred to by Dunedin City Council Transport Manager Richard Saunders on which the latest Mt. Ross-Sutton Bridge design is based.

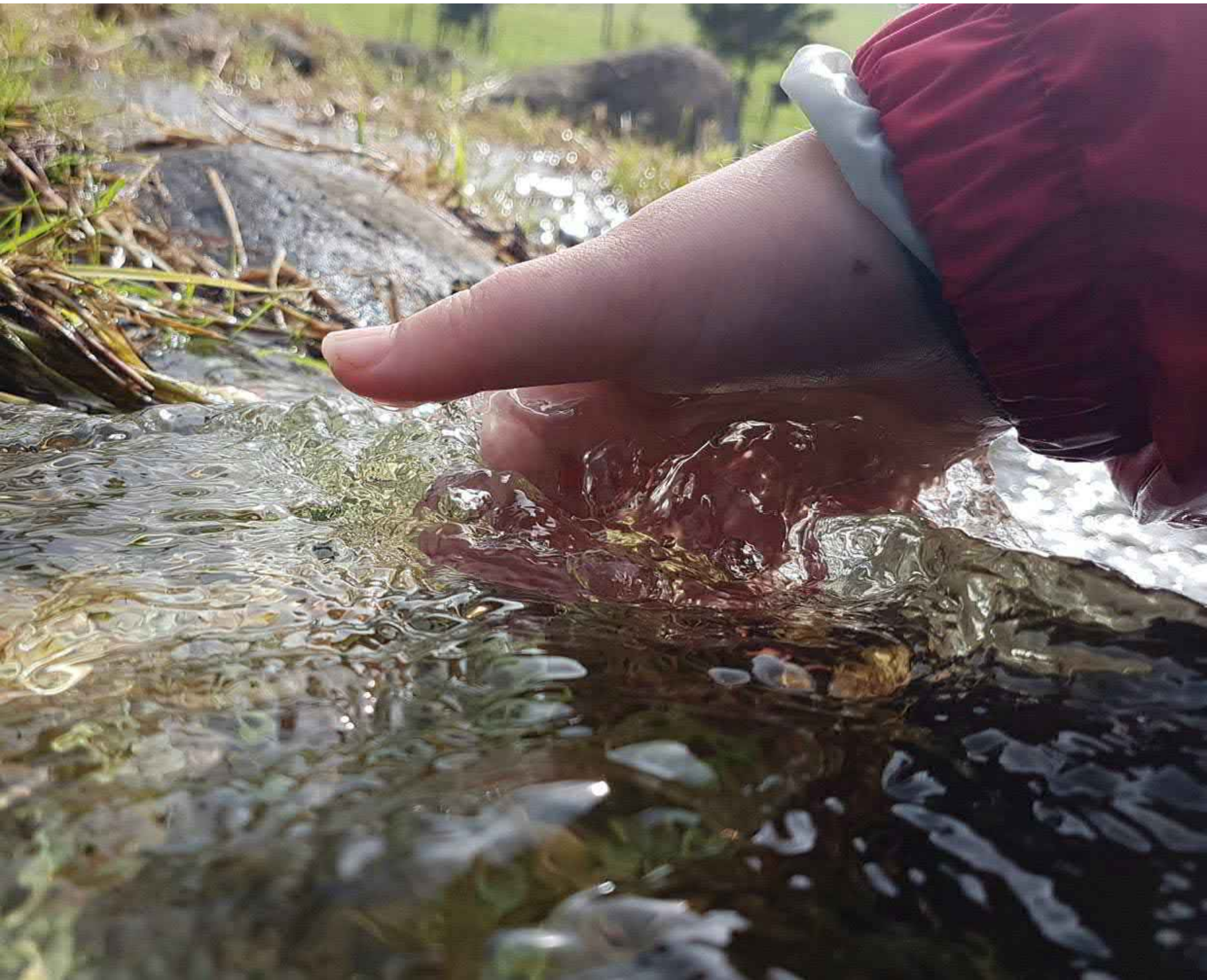
Please find attached the information requested.

Regards

Rebecca Murray
Governance Support Officer

DCC Bridge 77 Sutton-Mt Ross Rd

Hydrology - Hydraulics Analysis & Design



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Document History and Status

Revision	Date	Author	Reviewed by	Approved by	Status
1	15/11/2018	F Maas	L Algeo	A Martindale	Final

Revision Details

Revision	Details

1 Introduction

1.1 Background

Dunedin City Council (DCC) commissioned Opus International Consultants Ltd. (Opus) to design a replacement bridge for the Sutton - Mt. Ross Road Bridge which collapsed during a flood event in July 2017. The original bridge (Figure 1-1) was located 8km south of Middlemarch and approximately 50km northwest of Dunedin and was a vital link between the Mt. Ross and Mt Stoker areas and Dunedin. The bridge carried one lane of traffic across the Taieri River.

Figure 1-1 Location plan of the bridge

1.2 Design Standards

1.2.1 Design events

The proposed bridge will be designed in accordance with NZTA (2018). The bridge will be an Importance Level 2 structure (as defined by NZTA (2018)). As a result, the Serviceability Limit State (SLS) event should have been the 2% Annual Exceedance Probability Event (AEP) in the Taieri River. Given the expected flood levels, location of the bridge and the traffic volume a departure was sought for a reduction in the SLS design standard which was granted. Hence the hydraulic design is with respect to the following criteria:

- The SLS event will be the 4% AEP event in the Taieri River.
- The Ultimate Limit State (ULS) event will be the 0.1% AEP event in the Taieri River.

1.2.2 Freeboard

Table 2.4 in NZTA (2018) specifies that for an Importance Level 2 structure with a likelihood of woody debris being carried in the flood flow such as this bridge that a minimum freeboard of 1.2m above the peak water level resulting from 2% AEP, 2118 climate, event be applied to set the soffit level of the bridge. Given the expected flood levels, location of the bridge and the traffic volume a departure was sought for a reduction in the SLS design standard which was granted. Hence a freeboard of 0.6m above the SLS event for the 2018 climate was adopted for the design of this bridge.

1.2.3 Climate Change

To reflect the future life of this structure from when it is constructed, consideration was given to the effects of future climate change out to 2118 (i.e. 100 years from the date of completion of construction) as detailed in Section 2.3.2c of NZTA (2018).

1.3 Report Purpose

This report details the analysis carried out into the hydrology of the Taieri River and the hydraulic analysis carried out to support the design of the proposed new bridge. It provides predicted flood water levels and velocities following the construction of the bridge as well as details of scour protection required of the approach embankments.

1.4 Approach

Firstly, the hydrology of the Taieri River at the bridge was reviewed to provide the peak flows for the design events considered. Next a coupled one-dimensional & two-dimensional (1D-2D) MIKE FLOOD hydraulic computational model was developed of the Taieri River and flood plain. It was calibrated against the 14 December 1993 and 22 July 2017 flood events and then used to predict the water levels and velocities in the channel and flood plain for the chosen design events. These results were then used to assist in the design of the bridge primarily in terms of hydrodynamic loading and scour protection of the approach embankment and abutments.

2 Bridge Design Flows

2.1 Introduction

Otago Regional Council (ORC) maintain a flow site Taieri @ Sutton that is near the bridge site. It is less than 1km from the Sutton - Mt Ross Road Bridge (Figure 2-1). As there are no significant tributaries within this reach of the Taieri River, the flows at the Sutton flow recorder are likely to be the same as at the bridge site.

Figure 2-1 Location of flow site and the Sutton-Mount Ross Road Bridge.

2.2 Analysis

Appendix A details the analysis of the hydrometric data of the Taieri @ Sutton flow site and the adjustment for climate change to provide estimated design flood flows for various events including adjustments for climate change to 2068 and 2118. Table 2-1 summarises the relevant results used in the design.

Table 2-1 Design flood estimates adjusted for climate change (m³/s).

ARI[1] (yr.)	AEP (%)	Design Flood Flow Estimates (m^3/s)		
		2018	2068	2118
2.33	42.8	162	178	194
5	20	264	290	317
10	10	352	387	422
20	5	439	483	527
25	4	495	545	594
50	2	552	607	662
100	1	637	701	764
200	0.5	721	793	865
500	0.2	832	915	998
1000	0.1	915	1007	1098

2.3 Calibration Events

There is photographic information available of water levels at the bridge for two historic flood events that can be used for calibration purposes. Table 2-2 details the estimated values for these events at the bridge. There is greater uncertainty as to the actual water levels at the bridge during the 2017 event as the bridge was washed away during this event.

[1] ARI = Average Recurrence Interval

Table 2-2 Estimated peak flows and water levels at the bridge of calibration events

Event	Estimated Peak Flow (m^3/s)	Estimated Peak Water Level (m RL)	Estimated AEP (%)
14 December 1993	570	185.7	1.7%
22 July 2017	434	184.0	5.1%

2.4 Adopted Design Flows

While Table 2-1 lists the design flood estimates, only those for the SLS and ULS flood event (as defined in Section 1.2.1 are relevant for the design of the proposed bridge. The key values are summarised in Table 2-3.

Table 2-3 Adopted design flood estimates used for the design (m^3/s).

Event	AEP (%)	Design Flood Flow Estimates (m^3/s)	
		2018 Climate	2118 Climate
SLS	4	495	594
ULS	0.1	915	1098

3 Hydraulic Analysis

3.1 Introduction

Estimating the extent of the projected design level flood event is a critical design parameter for the new bridge structure. The deck height of the preferred option can only be set once the anticipated flood flow levels are known. Similarly, the geometric design and the required bridge auxiliary works can only progress once the deck height is confirmed.

3.2 Software

The 1D/2D computational hydraulic modelling package MIKE FLOOD version 2016, service pack 3 by the Danish Hydraulic Institute was used to analyse the flow past the bridge site including the adjacent floodplain. It is used extensively in New Zealand and throughout the world for modelling open-channel flow, overland flow and hydraulic structures. In addition to flow effects at the bridge site, the model considers upstream and downstream effects. It consisted of two coupled components:

- MIKE 11: a one-dimensional hydraulic computational modelling package to represent the flow in the main channel and past the bridge structure; and
- MIKE 21 FM[2]: a two-dimensional hydraulic computational modelling package to represent the flow across the floodplain and the bridge approach embankments.

3.3 Assumptions

The hydraulic analysis includes the following assumptions:

- The floodplain has a relatively uniform land use and can be represented by a constant roughness.
- The peak flow of the flood event simulated can be represented by a constant peak flow.

[2] Flexible Mesh (FM) consists of a flexible mesh consisting of triangles and/or rectangles of varying shape to analyse the two-dimensional flow across floodplains and in river or coastal channels.

- The flow in the channel at the downstream boundary can be represented by a stage-discharge relationship based on a Manning's calculation for normal (subcritical) flow.
- The design of the proposed bridge foundations is such that the piers and abutments are founded sufficiently deep in bed rock and consequently no scour analysis is required.

3.4 Projection & Datum

The surveyed data were captured in the following projection and datum:

- NZGD2000 New Zealand Transverse Mercator projection.
- New Zealand Vertical Datum 2016.

The bridge design, and consequently the hydraulic computational model including its results, are in the same projection and datum.

3.5 Model Extent

Figure 3-1 depicts the extent of the components of the MIKE FLOOD model. The model represents a 62ha floodplain bisected by a 1.4km reach of the Taieri River starting at a point approximately 600m upstream of the location of the bridge.

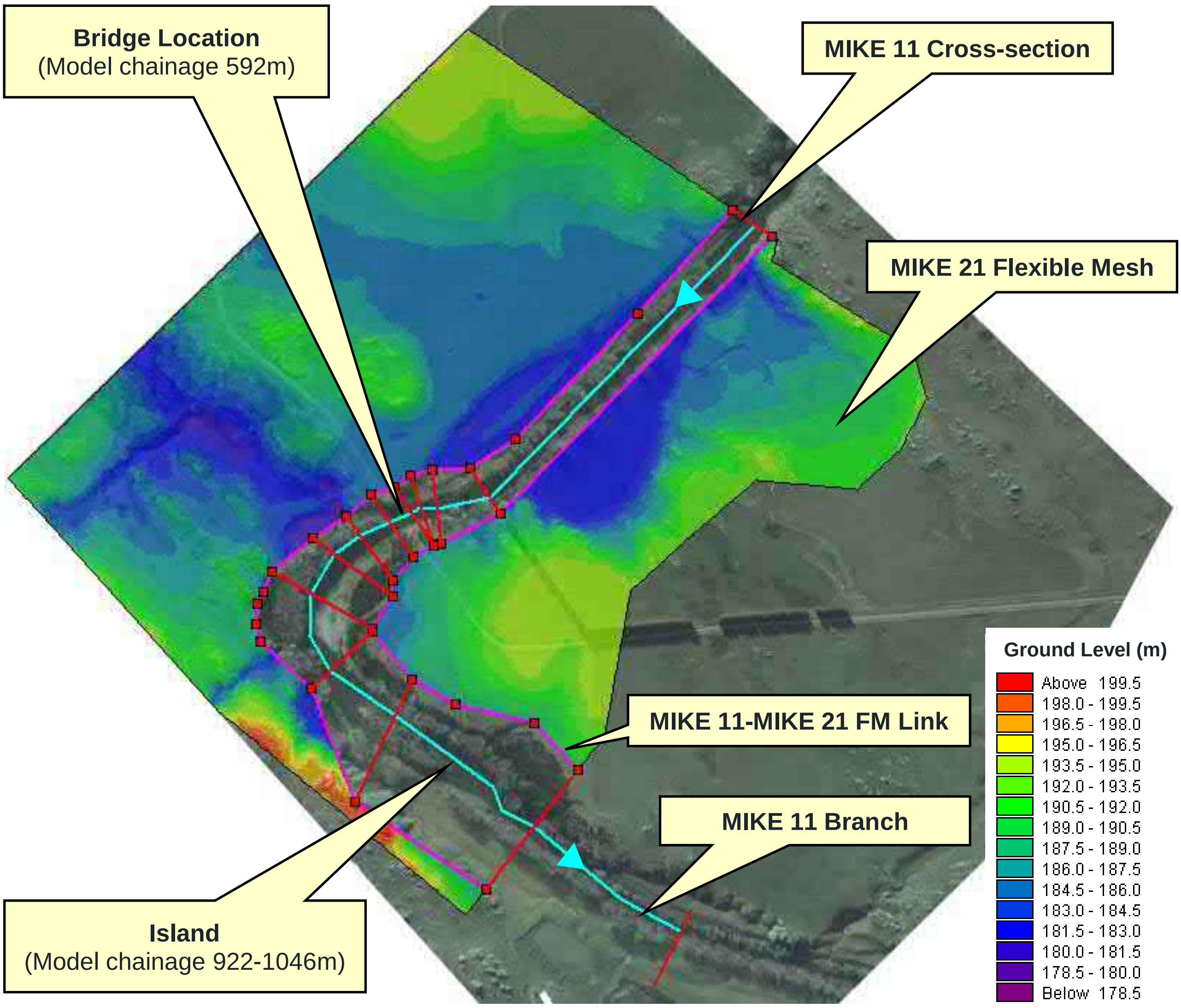

Figure 3-1 MIKE FLOOD hydraulic computational model extent

The river channel is represented in the MIKE 11 component while the floodplain is represented in the MIKE 21 FM component of the model.

Given the spacing of the cross-sections, the lateral links between the MIKE 11 and MIKE 21 have been set such that the levels in the MIKE 21 FM terrain determine what at what level the flow will spill onto the floodplain. This provides a more accurate representation in the model of the exchange of flow between the main channel and floodplain.

3.6 Terrain

A detailed topographical survey over a 1 km^2 area surrounding the original bridge site was completed in late July 2018 by Paterson Pitts, Dunedin.

3.6.1 River cross-sections

The captured data included 13 river cross-sections extending from the left bank floodplain to the right bank floodplain (true left and right banks are as viewed looking downstream). These were used for the existing (no bridge) scenario.

3.6.2 Flood plain mesh

The flood plain was surveyed using an unmanned aerial vehicle and augmented with detailed topographic information in the area surrounding the original bridge abutment and approaches for design purposes. This data was then interpolated onto a triangular mesh with a maximum triangle area of 20m^2.

3.7 Roughness

The banks of the Taieri River near the bridge are covered with grass and trees. Upstream of the bridge the trees are widely spaced. Downstream of the bridge site the density of the trees increases as well as being interspersed with bushes and scrub. Furthermore, approximately 330m downstream of the bridge there is a substantial bush and scrub covered island in the middle of the river that is likely to be submerged during a flood event. Hence upstream of the bridge the channel's resistance to flow is represented by a Manning's n value of 0.06 in MIKE 11 whereas downstream of the bridge this is much higher at a value of 0.10.

The floodplain in the MIKE21 FM portion of the model is largely covered by grasses and consequently it's resistance to flow has been represented by a uniform Manning's M roughness value of 20 (being equivalent to a Manning's n value of 0.05).

3.8 Boundary Conditions

There are two boundaries that have been connected to the MIKE 11 component of the MIKE FLOOD model. No boundaries were connected to the MIKE 21 FM component as all flow is expected to enter and leave the model domain in the main channel (i.e. the MIKE 11 component).

3.8.1 Upstream Boundary

A constant inflow at the upstream end of the MIKE 11 component of the model to represent the peak inflow during each flood event simulated. Section 2 describes the derivation of these design inflows.

3.8.2 Downstream

The downstream boundary of the MIKE 11 model allows water to flow out of the model domain. The flow in the channel is subcritical and hence the level can be determined from a Manning's normal flow calculation that is dependent on the shape of the cross-section, the channel roughness and the slope of the channel. The channel Manning's n roughness is 0.10 at this location (refer Section 3.7) and the channel slope is 0.18% (based on the thalweg of the surveyed cross-sections). Figure 3-2 shows the cross-section of the channel at this location and Figure 3-3 shows the resultant stage-discharge relationship used as the downstream boundary.

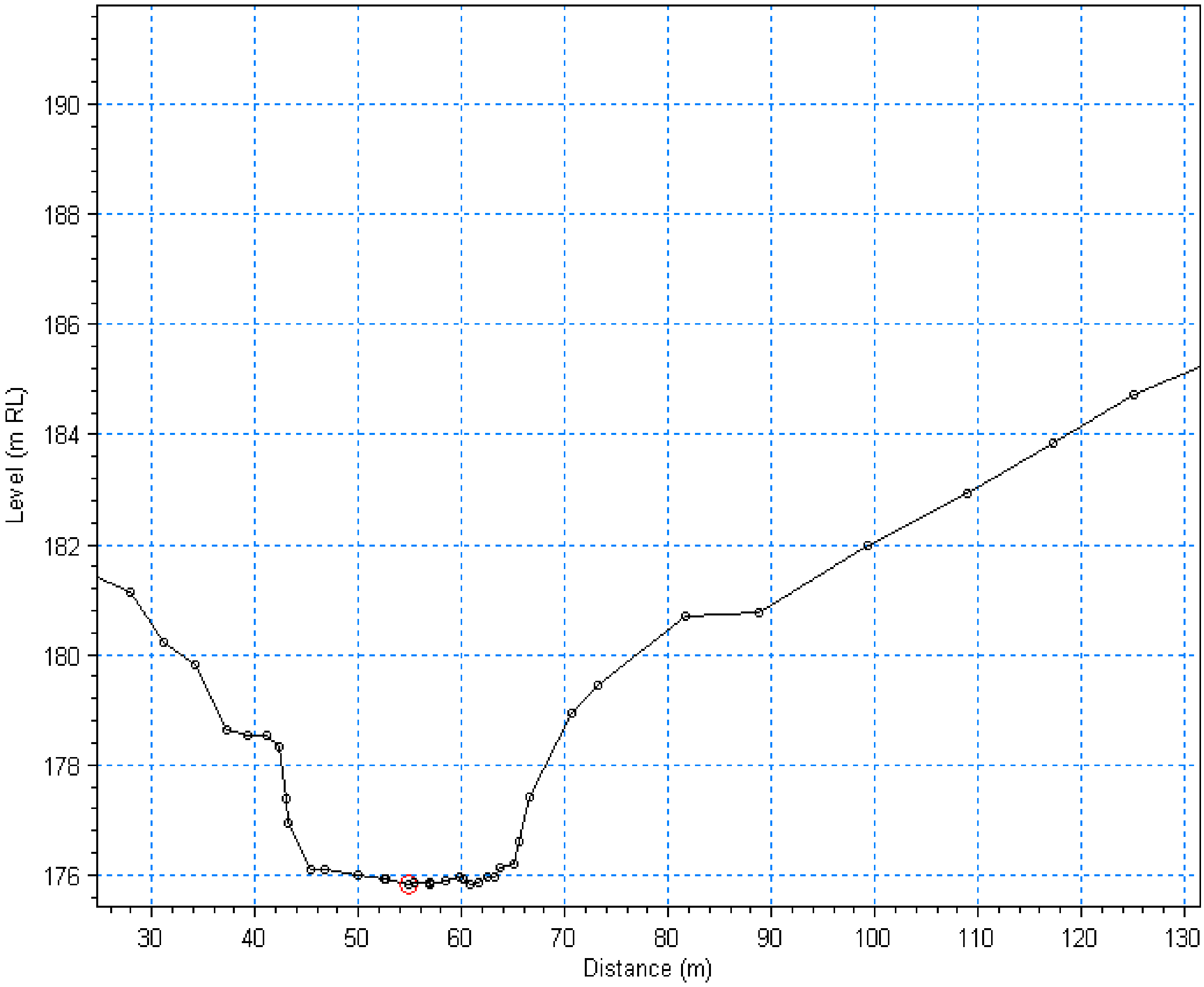

Figure 3-2 MIKE 11 cross-section at downstream boundary of the model.

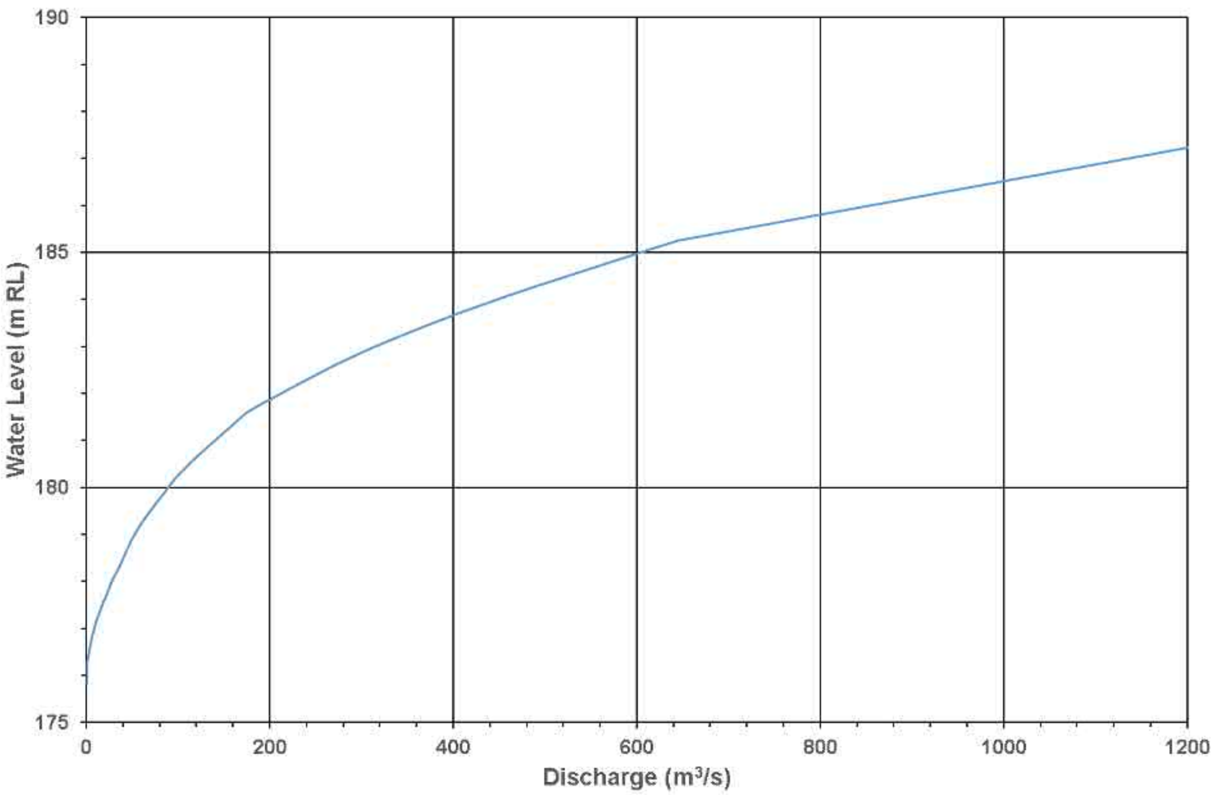

Figure 3-3 MIKE 11 downstream boundary stage-discharge relationship.

3.9 Calibration

Table 3-1 details the results of the calibration of the model. There is greater certainty as to the actual recorded water level during the December 1993 event and hence the model calibration is acceptable for the design of the proposed bridge.

Table 3-1 Estimated peak flows and water levels of calibration events.

Event	Peak Water Level (m RL)		
	Estimated	Modelled	Difference
14 December 1993	185.7	185.5	-0.2
22 July 2017	184.0	184.6	+0.6

3.10 Existing Situation

Eight events were analysed using the MIKE FLOOD model of the current (no-bridge) scenario to assist in the design process. Four events were included in addition to those listed in Table 2-3 to provide information on the likely water levels that could be expected during the construction period. Table 3-2 details the peak water levels predicted by the model results. Figure B - 1 to Figure B - 3 show the predicted peak depth and velocity of the flow near the proposed bridge across the floodplain for the 5% AEP (2018 climate), SLS (4% AEP, 2118 climate) and ULS (0.1% AEP, 2118 climate) event. The result of the first of these is useful for the construction methodology as events with a lower probability of occurrence (i.e. a higher ARI or larger flood flow) affect the likely construction work site.

Table 3-2 Peak water levels at the bridge site for the current (no-bridge) scenario.

AEP (%)	Peak Water Level (m RL)	
	2018 Climate	2118 Climate
42.8	182.1	-
20	183.2	-
10	183.8	-
5	184.7	-
4 (SLS)	185.1	185.7
0.1 (ULS)	187.1	187.9

3.11 Sensitivity Testing

3.11.1 Discharge

Comparing the results in Table 3-2 for a 0.1% AEP, 2018 climate, event with that for the 2118 climate shows that a 20% increase in flow results in only a 0.8m rise in water level. The invert of the channel is approximately RL 176.7m and consequently the water depth increases by 8% from 10.4m to 11.2m. Hence the model results are not very sensitive to errors or changes in discharge.

3.11.2 Channel Roughness

To investigate the sensitivity of the model with respect to the channel and floodplain roughness the model was re-run with a 5% increase in the channel and floodplain roughness for the 4% AEP, 2018 climate, event. As a result, the predicted peak water levels at the bridge site increased from RL 185.1m to RL 185.7m. The invert of the channel is approximately RL 176.7m and consequently the water depth increases by 8% from 8.4m to 9.1m. Hence the model results are sensitive to errors or changes in channel and floodplain roughness.

4 Impact of Proposed Bridge

4.1 Proposed Bridge Schematisation

The proposed bridge consists of a long approach embankment across the north (true right) flood plain and a bridge deck spanning the main channel supported by two piers (Figure 4-1). Consequently, it is represented in both components of the model.

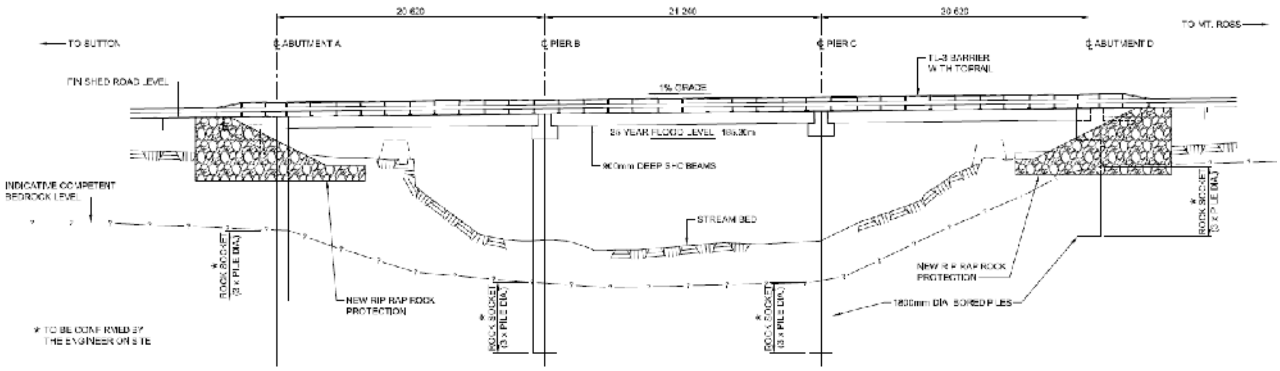

Figure 4-1 Elevation of proposed bridge structure.

4.1.1 Approach Embankment

The levels in the MIKE 21 FM mesh were updated with the levels of the approach embankment of the proposed bridge (Figure 4-2) to show the impact of raising the ground on the floodplain flow.

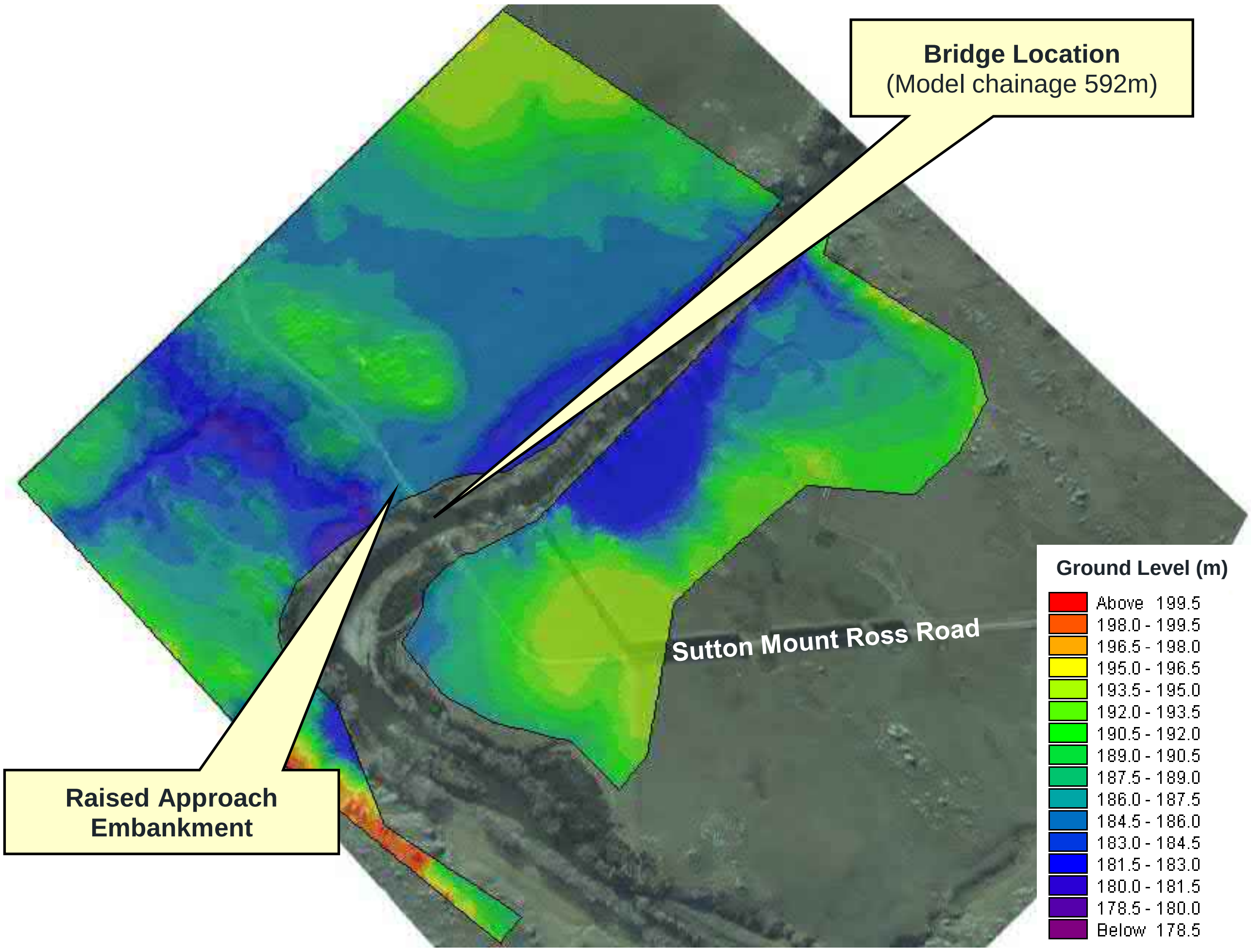

Figure 4-2 Schematisation of the approach embankment in the MIKE 21 FM mesh

4.1.2 Bridge Structure

The proposed bridge is located over the main channel and is consequently included in the MIKE 11 model of the main channel. It is likely to be submerged during flood events with flows greater than that of the SLS (4% AEP, 2118 climate) event and is represented as follows:

- An 88m-wide broad-crested weir with a crest level of RL 187.3m representing the average deck level of the proposed bridge; and
- A 6m-long culvert with an irregular cross-section (Figure 4-3) representing the opening under the bridge structure.

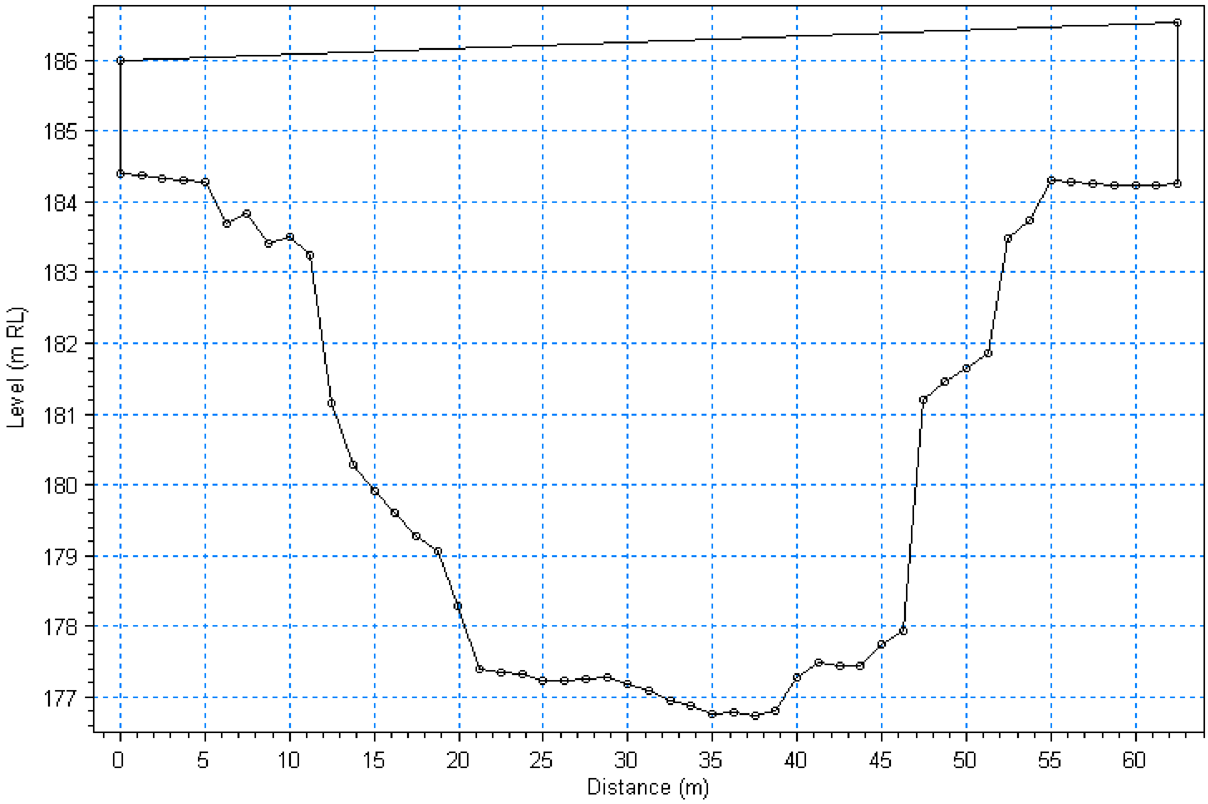

Figure 4-3 Cross-section representing the main channel under the proposed bridge

4.2 Results

The updated model was run for the six design events listed in Table 2-3 and results analysed to determine the predicted peak water levels and flow velocities following the construction of the proposed bridge.

4.2.1 Predicted Peak Water Levels

Table 4-1 provides the predicted water level immediately upstream of the bridge. They provide an insight into the freeboard of the soffit of the bridge and the afflux of the proposed bridge. Figure B - 4 and Figure B - 5 show the predicted peak flow depths and velocities across the floodplain near the bridge for the SLS and ULS events respectively for the 2118 climate.

Table 4-1 Predicted peak water levels upstream of the proposed bridge.

AEP (%)	Peak Water Level (m RL)	
	2018 Climate	2118 Climate
4 (SLS)	185.3	185.9
0.1 (ULS)	187.4	188.2

The results show that:

- The bridge has 0.6m freeboard for the 4% AEP, 2018 climate, event;
- The approach embankment and bridge deck are not overtopped during the SLS (4% AEP, 2118 climate) event; and
- During the ULS (0.1% AEP, 2118 climate) event the approach embankment is overtopped by up to 1.8m and the bridge deck by up to 1m.

4.2.2 Predicted Flow Velocities

Three different flow velocities are key to the design of the bridge. These are the approach flow velocity in the channel, the flow velocity in the bridge opening and the flow velocity across or along the approach embankment. Table 4-2 details the peak cross-section averaged approach flow velocity taken 14m upstream of the bridge centre-line and Table 4-3 those for the bridge opening. Figure B - 6 shows the magnitude and direction of the predicted peak flow velocities across the floodplain near the bridge for the ULS, 2118 climate, event with a peak flow velocity of 1.7m/s across the approach embankment.

Table 4-2 Predicted peak cross-section averaged approach velocities.

AEP (%)	Peak Flow Velocity (m/s)	
	2018 Climate	2118 Climate
4 (SLS)	1.4	1.5
0.1 (ULS)	1.5	1.5

Table 4-3 Predicted peak cross-section averaged velocities in the bridge opening.

AEP (%)	Peak Flow Velocity (m/s)	
	2018 Climate	2118 Climate
4 (SLS)	2.1	2.2
0.1 (ULS)	2.2	2.3

The velocity results show that the highest velocities can be expected underneath the bridge deck through the bridge opening. For the structural design a factor of safety should be applied to the values in Table 4-2 and Table 4-3.

5 Hydraulic Design

5.1 Introduction

The abutments and piers of the bridge structure are to be founded sufficiently deep in bed rock that no scour calculation or scour protection is required for the ULS (0.1% AEP, 2118 climate) event. Hence the only scour protection that is required is to ensure that the bridge abutments and approach embankments are not scoured away during an SLS (4% AEP, 2118 climate) event.

Hydrodynamic loading has been calculated as part of the structural design of the bridge and is not included here.

5.2 Abutment and Approach Embankment Scour Protection

The design of the scour protection followed the approach outlined in Melville and Coleman (2000) as specified by NZTA (2018). At this stage the analysis is limited to providing the median rock size required to protect against the effects of scour. The full grading envelopes, layer thicknesses, embedment depths and protection extents can be developed from these rock sizes.

The north (right bank) abutment encroaches approximately 96m into the floodplain flow for the SLS event, and therefore velocities near the abutment are likely to be larger than the average approach flow. Melville and Coleman (2000) lists several methods to use in the selection of rock for abutment protection. Those based on Austroads (1994), Croad (1989) Richardson and Davis (1995) and Pagan-Ortiz (1991) were used as they are considered the most appropriate for spill-through abutments.

The results from the scour protection analysis show that rock riprap with a median rock size (i.e. D_{50}) of at least 380mm will be required.

6 Summary

To assist in the design of the proposed Sutton – Mount Ross Road Bridge the hydrological and hydraulic aspects were investigated. The outputs of this work are essential in setting the soffit level (and hence the deck level) of the bridge, the geometric road design and the structural design of the bridge.

The proposed bridge will be an Importance Level 2 bridge in terms of NZTA (2018). Departures have been granted and consequently the design standards are as follows:

- The SLS event is the 4% AEP, 2118 climate, event;
- The ULS event is the 0.1%, 2118 climate, event; and
- A minimum soffit freeboard of 0.6m above the 4% AEP, 2018 climate.

The hydrology of the Taieri River was analysed by reviewing the record of the Taieri @ Sutton flow gauge. These were then adjusted for climate change to provide the full range of design flows.

A computational hydraulic MIKE FLOOD model was set up and calibrated against water levels for the 14 December 1993 and 22 July 2017 flood events. Following sensitivity testing it was used to predict peak water levels and flow velocities for the design events for the existing (no-bridge) scenario and following bridge construction. The results showed that the proposed bridge meets the specified agreed design standards in terms of water level. During a ULS event the peak flow velocity through the bridge opening under the deck is 2.3m/s. For the structural design a factor of safety should be applied to this value.

The abutments and piers of the bridge structure are to be founded sufficiently deep in bed rock that they do not require scour protection for a ULS (0.1% AEP, 2118 climate) event. The approach embankment will require scour protection against the flow velocities during an SLS (4%AEP, 2118 climate) event. The north (right bank) abutment encroaches approximately 96m into the floodplain flow for the SLS event, and therefore velocities near the abutment are likely to be larger than the average approach flow. The results from the scour protection analysis show that rock riprap with a median rock size (i.e. D_{50}) of at least 380mm will be required.

7 Glossary

Soffit	The soffit of a bridge is the lowest elevation of the underside of the bridge deck structure, including any supporting beams.
Thalweg	The thalweg of a river is a line drawn to join the lowest points along the entire length of a stream bed, defining its deepest channel.

8 References

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Appendix A – Hydrology – Design Flood Estimation

Memorandum

To	Kos Maas
Copy	Jack McConchie
From	Lizzie Fox
Office	Wellington Office
Date	3 September 2018
File	6-CD102.00/106GD
Subject	Design Flood estimation

1 Introduction

WSP Opus have been commissioned to design a new bridge over the Taieri River at Sutton. This bridge will replace the current Mount Ross Bridge at the same location. To assist with the hydraulic analysis and design, a range of design flows and their mean velocities are required. These can be derived by analysing the available hydrometric flow records from the Taieri River near the proposed bridge i.e. the Taieri River @ Sutton flow recorder.

2 Hydrometric data

Although there is some uncertainty regarding the exact location of the Taieri River @ Sutton flow recorder, it is less than 1km from the Mt Ross Bridge (Figure 1). As there are no significant tributaries within this reach of the Taieri River, the flows at the Sutton flow recorder are likely to be the same as at the Mount Ross Bridge. The Taieri River @ Sutton flow site is maintained by Otago Regional Council (ORC), and the flow record is described in Table 1.

Figure 1: Location of flow site and the Mount Ross Bridge.

Table 1: Summary details of the Taieri River @ Sutton flow site.

SITE	START	END	RECORD LENGTH	NO. OF GAPS	DURATION OF GAPS (%)
Taieri River @ Sutton	Aug-1960	Jul-2018	~58 years	123	22%

The summary flow statistics for Taieri River @ Sutton are listed in Table 2.

Table 2: Summary statistics of the Taieri River @ Sutton flow site (1960-2018). Flows are in m³/s.

SITE	MIN.	MEAN	MEDIAN	MAX.	U.Q.	L.Q.
Taieri River @ Sutton	0.6	18.0	12.7	560	23.5	5.2

2.1 Gap and quality analysis

Because of the desktop nature of this study, no independent quality assurance of the data has been undertaken. However, since the data have been collected and quality assured using industry best practice, they are assumed accurate; and to reflect flows likely to be experienced at the Mt Ross Bridge. A brief gap analysis, and comparison of the gauging data with the rated flows, were undertaken to ensure that the flow data were robust and fit for purpose.

Over the ~58-year flow record, there is 22% of missing record (Table 1); however, over half of this missing data occurred prior to 1970. A~4-year gap between 1964 to 1968 comprised much of this missing data (Figure 2).

Since 1990, there is only 0.6% of missing record i.e. over the last ~28 years. The quality of the data series has therefore improved significantly over the past three decades. It is considered that the quality of the data, and the length of record (i.e. ~45-years), are sufficient for robust frequency analysis.

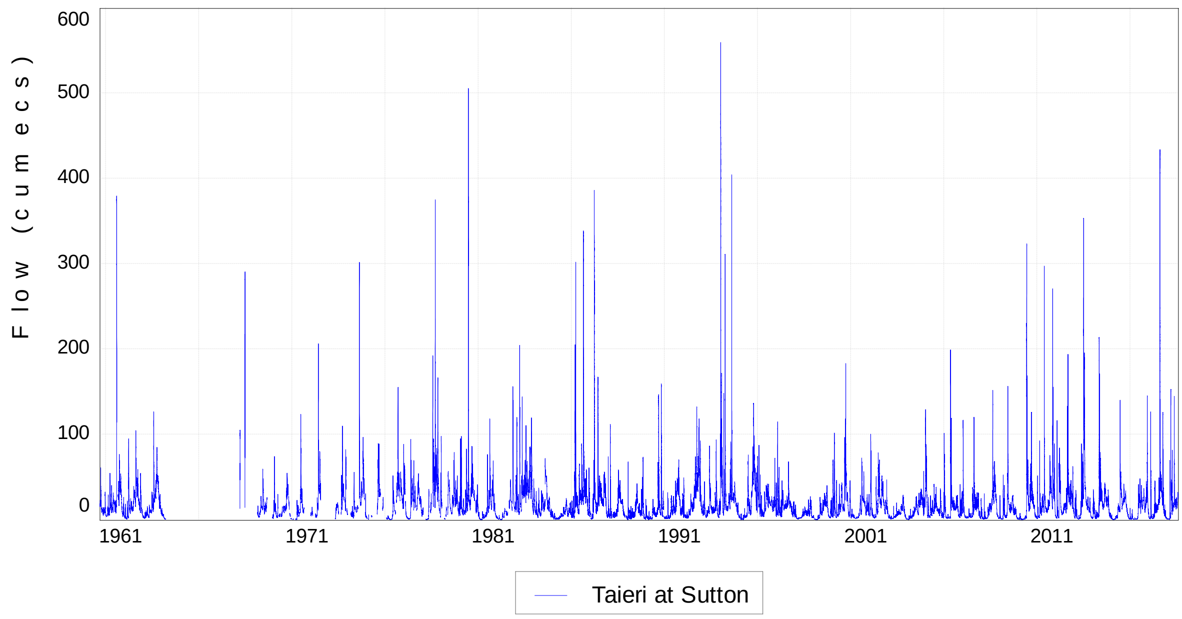


Figure 2: Taieri River @ Sutton flow record (1960-2018). Large gap from 1964 to 1968, and subsequent gaps of up to 4 months up until the late 1970s.

244 gaugings have been conducted of the Taieri River at Sutton flow site to generate multiple stage-discharge relationships i.e. rating curves (Figure 3). Six percent of the gaugings have been carried out at flows greater than 94m³/s; flows at or greater than this represent the highest 1% of discharges measured at the site. These gaugings define the 'top end' of the rating relationship between flow and water level (stage). These high flow gaugings reduce the uncertainty inherent in discharge estimation during large flood events; those critical when defining the expected magnitudes of larger design events.

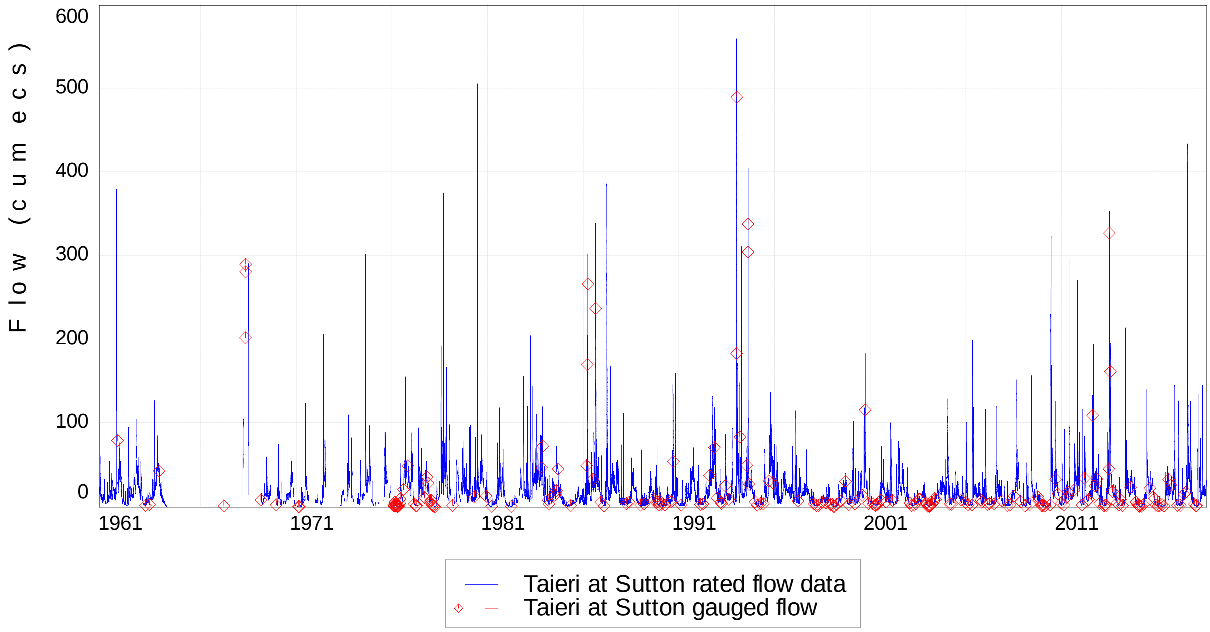


Figure 3: Flow gaugings measured at Taieri River @ Sutton compared with the rated flow series.

The largest gauged flow was on the 24 December 1993; 490m³/s. This gauging was in good agreement with the rated flow at the time (504m³/s) i.e. a difference of only 3% (Figure 4). The highest rated flow has been 560m³/s, which occurred on the same day, although approximately 12-hours earlier than the gauging. The gauging data therefore suggests the ratings applied to the site are reliable, and are suitable for defining the magnitudes of large design flood events.

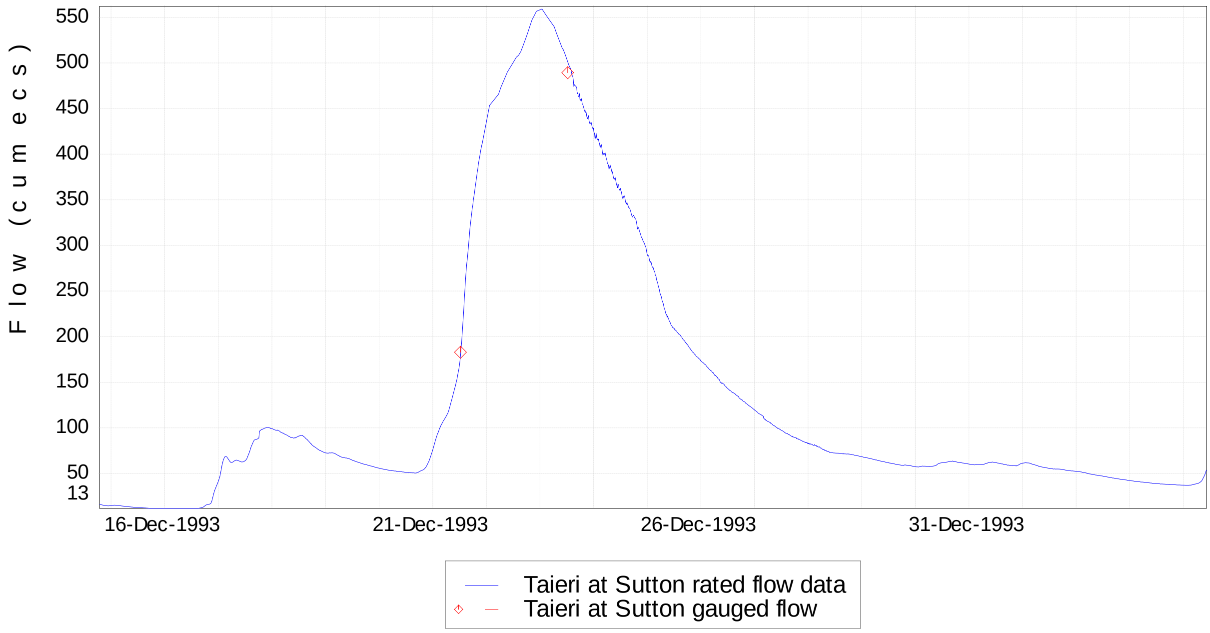


Figure 4: The largest rated flow event in Taieri River @ Sutton, compared with the gauged data.

3 Design Flood Flows

A frequency analysis was undertaken of the annual flood maxima series from the Taieri River @ Sutton flow site.

Three types of statistical distribution were assessed for how well they modelled the actual annual flood maxima series (i.e. Gumbel, Pearson 3 (PE3) and GEV). The distribution which provided the best fit to the annual maxima series was then used to estimate peak flows for flood events of specific annual exceedance probabilities (i.e. AEPs). The criteria adopted in this study were:

- The distribution that provided the best-fit through all the annual flood maxima;
- The distribution with the most realistic shape; and
- The distribution that provides the closest approximation to the extreme flood maxima.

As is standard practice, the frequency analyses were performed on a 12-month partition. That is, only the largest flood in each complete year was plotted, and the most appropriate statistical distribution fitted to those annual values (Figure 5).

While the Gumbel distribution provides the best fit to the largest flood on record, the PE3 statistical distribution provides a 'more balanced' fit to all those events with AEPs less than 20% (i.e. a 5-year ARI event). Furthermore, the assumption of a PE3 distribution produces slightly more conservative estimates of the magnitudes of larger design events. Given the significance of the Mt Ross Bridge, it is considered that the more conservative approach is appropriate. Consequently, the magnitudes and frequencies of a range of design events were estimated assuming that the annual flood maxima series approximate a PE3 statistical distribution.

The magnitudes and frequencies of a range of design flood events are listed in Table 3.

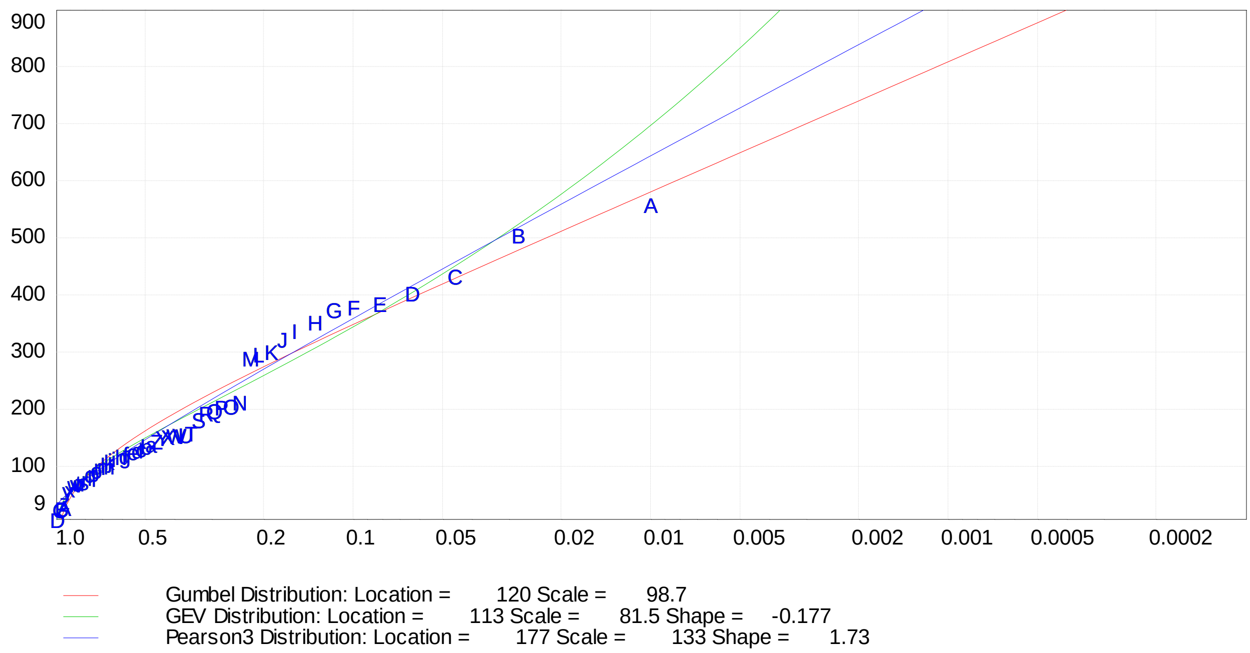


Figure 5: Frequency analysis of the annual flood maxima series from the Taieri River @ Sutton flow record.

Table 3: Annual exceedance probabilities (AEPs) for Taieri River @ Sutton (1961-2017) assuming a PE3 distribution. Design flows are rounded to the nearest whole number.

ARI (YEARS)	AEP (%)	FLOW (m^3/s)
2.33	43.5	162
5	20	264
10	10	352
20	5	439
25	4	495
50	2	552
100	1	637
200	0.5	721
500	0.2	832
1000	0.1	915

The Taieri @ Sutton flow series provides ~45 years of data for analysis. The reliability of estimates of the magnitudes of design floods is a function of the length of flow record used, and the appropriateness of the flow record for a particular location. As a general rule of thumb, AEPs should not be extrapolated beyond twice the length of the record (Davie, 2008). NIWA, however, use a general rule of thumb of five times the length of record.

Using either assumption, the uncertainty of flow estimates increases rapidly with more extreme events. Therefore, there is greater uncertainty inherent in the estimates of the magnitudes of flood events greater than 1% AEP. The adoption of a PE3 statistical distribution for the annual flood maxima series, however, ensures conservative design flows.

4 Velocity derivation

To assist with hydraulic design, an estimate of the flow velocity during design floods is required. The velocity affects the loading on any piers, the scour adjacent to the piers, and the risk of erosion of the abutments.

Using the available gauging information from the Taieri @ Sutton flow recorder, a relationship was derived between flow and mean velocity. This relationship can be extrapolated to determine the mean velocity at any given flow, including the design events described in Table 3.

While it is likely that the maximum velocity is the critical design parameter, these data are not available. However, there is likely to be a relationship between the mean and maximum velocities, and so the analysis of mean velocities should be indicative of flow behaviour during larger flood events.

Of the 244 gaugings, available for the Taieri @ Sutton flow recorder, 192 recorded the mean velocity. Gaugings prior to 1976 did not have the mean velocity stored with the derived flow, nor did the period of gaugings from February 1993 to March 1995. The relationship between the gauged flow and mean velocity is displayed in Figure 6, along with the fitted trendline.

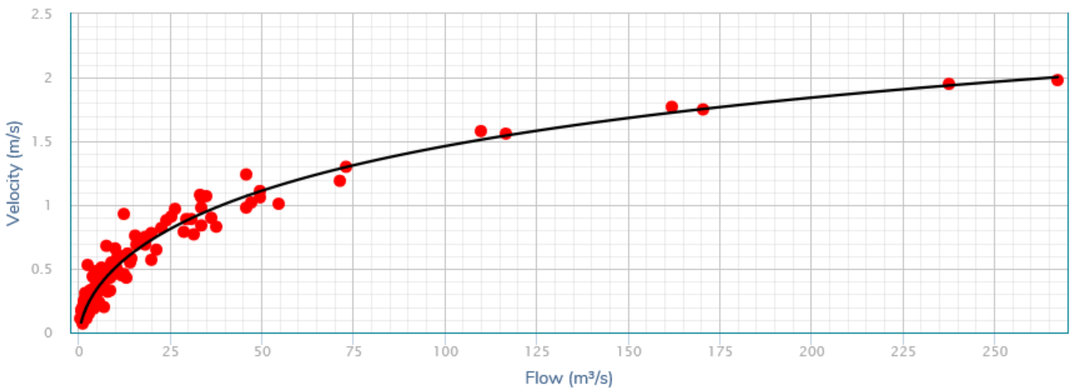


Figure 6: *Relationship between mean flow and velocity during gaugings carried out between 1976 to 2018 (excluding 1993-1995). R^2=0.96*

One gauging was excluded from the analysis; a flow of 327.7m^3/s with a mean velocity of 1.54m/s recorded on the 18 June 2013. Although this was the largest gauged flow for which there is also a velocity measurement, it plotted significantly lower than all the other gaugings.

When flows exceed bankfull discharge, it is possible that the mean velocity decreases despite the increase in discharge. This might explain the apparently anomalous gauging result.

There is a strong relationship between the mean velocity and flow, with an r^2 of 0.96 (Figure 6). This relationship was extrapolated to determine the mean velocity for various design flows (Figure 7 & Table 4).

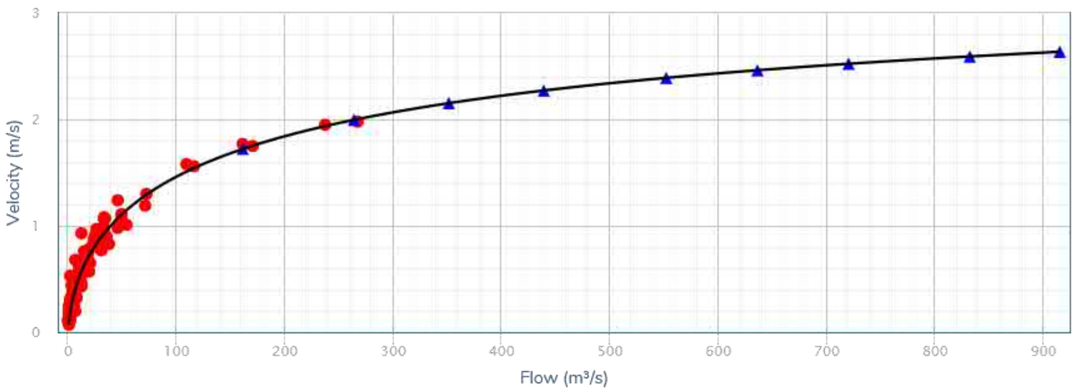


Figure 7: *Extrapolated mean velocities of various design flows using the derived relationship between gauged flow and mean velocity.*

Table 4: *Peak discharge and mean velocities during a range of design events.*

ARI (YEARS)	AEP (%)	FLOW (m^3/s)	VELOCITY (m/s)
2.33	43.5	162	1.7
5	20	264	2.0
10	10	352	2.2
20	5	439	2.3
25	4	495	2.4
50	2	552	2.4
100	1	637	2.5
200	0.5	721	2.5
500	0.2	832	2.6
1000	0.1	915	2.6

5 Climate Change

If predicted climate change eventuates, it may cause more than just a rise in the world's temperature. Warmer temperatures mean that more water vapour will enter the atmosphere while also increasing the air's ability to hold moisture. Furthermore, sensitivity analysis has indicated that changes in rainfall are often amplified in runoff.

The Ministry for the Environment have released the climate change predictions for New Zealand based on the IPCC 5th assessment (Ministry for the Environment, 2016). However, these projections have not been transformed into a methodology for determining effects on rainfall or flood flows. Therefore, the methodology that was developed in 2010 for determining the projected increase in rainfall because of climate change in New Zealand has been adopted (Ministry for the Environment, 2010).

The mean annual temperature for Otago is predicted to increase by between 0.1 and 1.9˚C by the 2040s, and 0.8 and 4.6˚C by the 2090s (Table 5).

Table 5: Projected increases in mean annual temperature by 2040 and 2090 for the Otago Region (Ministry for the Environment, 2010).

SCENARIO	2040 (˚C)	2090 (˚C)
Lower limit	0.1	0.8
Average	0.9	2.0
Upper limit	1.9	4.6

Note: These data are from Tables 2 and 3 in Ministry for the Environment (2010). The original tables cover the period from 1990 (1980-1999) to 2040 (2030-2049) and 2090 (2080-2099) based on downscaled temperature changes for 12 global climate models, re-scaled to match the IPCC global warming range for six illustrative emission scenarios.

The MfE methodology recommends a percentage adjustment per degree of warming that should be applied to the high intensity rainfall totals to account for the effect of global warming. For example, rainfall during any event with an average recurrence interval exceeding about 30 years is expected to increase by 8 percent per degree of projected warming. Consequently, rainfall within the Taieri catchment might be expected to increase by an average of 16% by the 2090s.

Currently, the direct effect of climate change on stream runoff, and particularly flooding, has not been quantified. Since this study is particularly concerned with extreme events, when catchment storage is approaching saturation, it has been assumed that an increase in rainfall will produce a similar increase in runoff. A predicted average increase in temperature of 0.9°C by the 2040s and 2.0°C by the 2090s can then be used to adjust the peak design flows derived for the Taieri at Sutton. This approach should provide some conservatism and resilience to the design flows.

However, the percentage increases in MfE (2010) are relative to the base period being the 1990s. With the current level of service being estimated, the 50-year and 100-year adjustments for climate change are required to 2068 and 2118 respectively. To allow for any climate change that has occurred since the 1990s, the peak discharges in 2068 have been increased by 10% and those in 2118 by 20%. These slightly higher adjustments for the potential effects of climate change i.e. 10% as opposed to 7.2% and 20% as opposed to 16.8%, are to allow for the potential effects of climate change over the next 50 and 100 years i.e. to 2068 and 2118 rather than to the 2040s and 2090s respectively (Table 6).

Table 6 Design flood estimates adjusted for climate change (m^3/s).

ARI (yr.)	AEP (%)	2018	2068	2118
2.33	50	162	178	194
5	20	264	290	317
10	10	352	387	422
20	5	439	483	527
25	4	495	545	594
50	2	552	607	662
100	1	637	701	764
200	0.5	721	793	865
500	0.2	832	915	998
1000	0.1	915	1007	1098

Having adjusted the peak flow during a range of design events for the potential effects of climate change, the mean velocities could also be adjusted (Table 7). It is assumed that the same relationship between flow and mean velocity will persist under the climate change scenarios. This is realistic since the relationship is a function of flow and the channel characteristics, which will not change.

Table 7: Peak discharge and mean velocities during a range of design events, with 50-years and 100-years of predicted climate change applied.

ARI (yr.)	AEP (%)	2068		2118	
		Flow (m^3/s)	Velocity (m/s)	Flow (m^3/s)	Velocity (m/s)
2.33	50	178	1.8	194	1.8
5	20	290	2.0	317	2.1
10	10	387	2.2	422	2.3
20	5	483	2.3	527	2.4
25	4	545	2.4	594	2.4
50	2	607	2.4	662	2
100	1	701	2.5	764	2.6
200	0.5	793	2.6	865	2.6
500	0.2	915	2.6	998	2.7
1000	0.1	1007	2.7	1098	2.7

6 Summary

The above analysis, and review of the available hydrometric data, allow the following conclusions:

- There is a flow recorder on the Taieri River within ~1km of the Mt Ross Bridge. Because of its proximity, similar catchment characteristics, and lack of any significant tributaries, flows measured in the Taieri @ Sutton can be used to derive the magnitudes and frequencies of a range of design events likely to affect the Mt Ross Bridge.
- A gap analysis and brief quality assurance indicates that the Taieri @ Sutton flow record provides ~45-years of reliable annual flood maxima. There is less than 1% of missing record over the past 30-years;

- 244 flow gaugings have been carried out at the site, with 6% of these at flows within the top 1-percentile. The 'top end' of the rating curve is therefore well-defined. This allows confidence in the estimated magnitudes of large flood events;

- The annual flood maxima series approximates a PE3 statistical distribution. Using this distribution, the magnitudes and frequencies of a range of design events, from 43.5. through 0.1% AEP (1000-year ARI), were estimated. It should be noted that extrapolating beyond 2 times the length of record significantly increases the uncertainty of design flow estimates;

- The available gauging data were used to define a relationship between flow and mean velocity. This relationship was then used to estimate the mean velocity during a range of design events. The 0.1% AEP (i.e. 1000-year ARI event) is estimated to have a peak discharge of $915m^3/s$, and a mean velocity of 2.6m/s. This seems to be realistic, although the maximum velocity is likely to be the critical control, rather than the mean velocity.

- When applying future climate change predictions to the various design events, the peak discharge during a 1000-year ARI event will increase to $1007m^3/s$ and $1098m^3/s$ in 50-years and 100-years respectively.

- Using the current relationship between flow and mean velocity, these increased discharges would result in the mean velocity increasing to 2.7m/s.

7 References

Davie, T. (2008). *Fundamentals of hydrology* (2nd ed.). New Zealand: Taylor and Francis.

Ministry for the Environment. (2016). Climate change projections for New Zealand: Atmospheric projections based on simulations undertaken for the IPC fifth assessment.

Ministry for the Environment. (2010). Tools for estimating the effects of climate change on flood flow: A guidance manual for local government in New Zealand.

Appendix B – Model 2D Result Plots

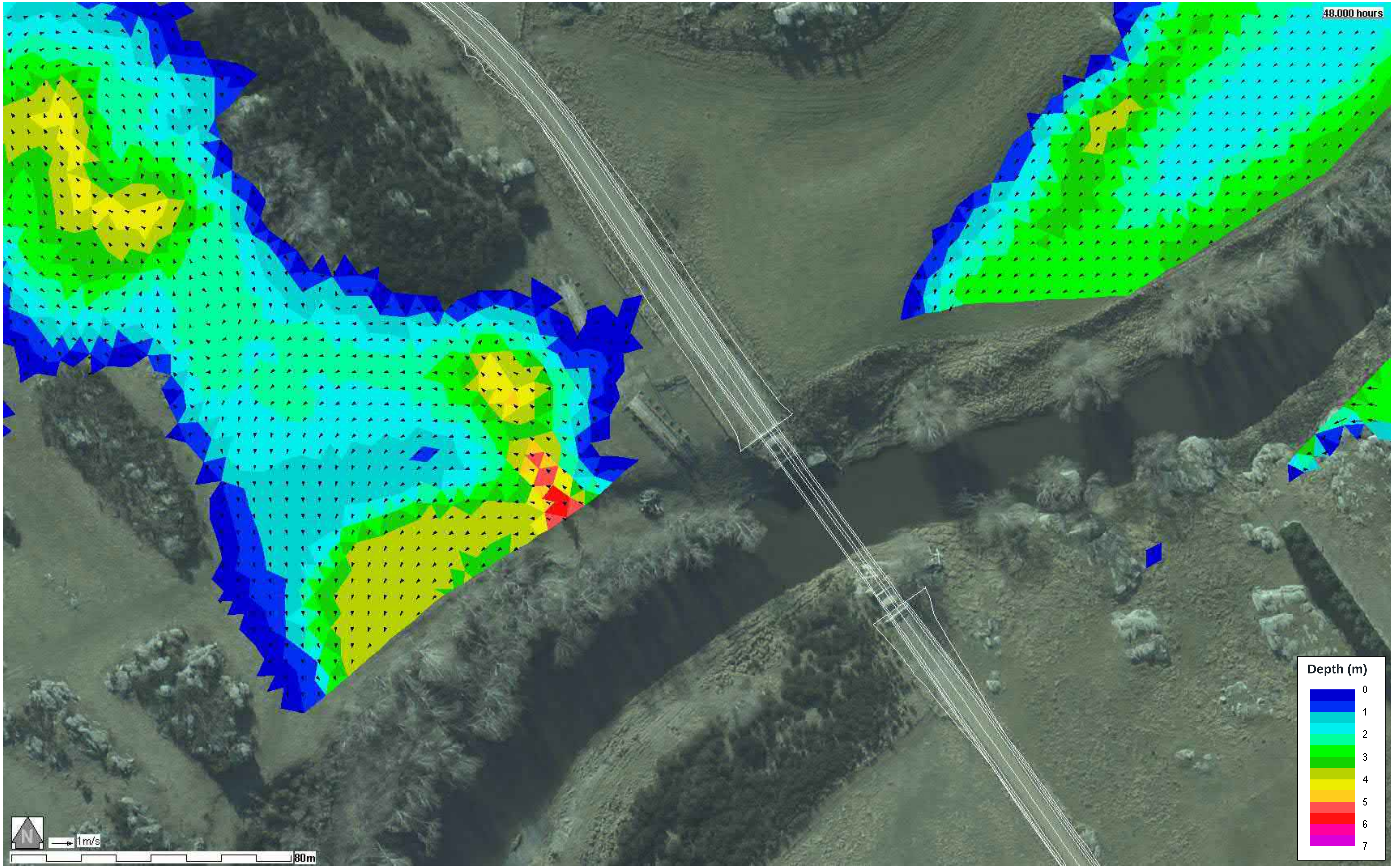


Figure B - 1 Predicted peak depth and velocity vectors near the proposed bridge (white lines) during a 5% AEP, 2018 climate, event for the existing (no-bridge) scenario.

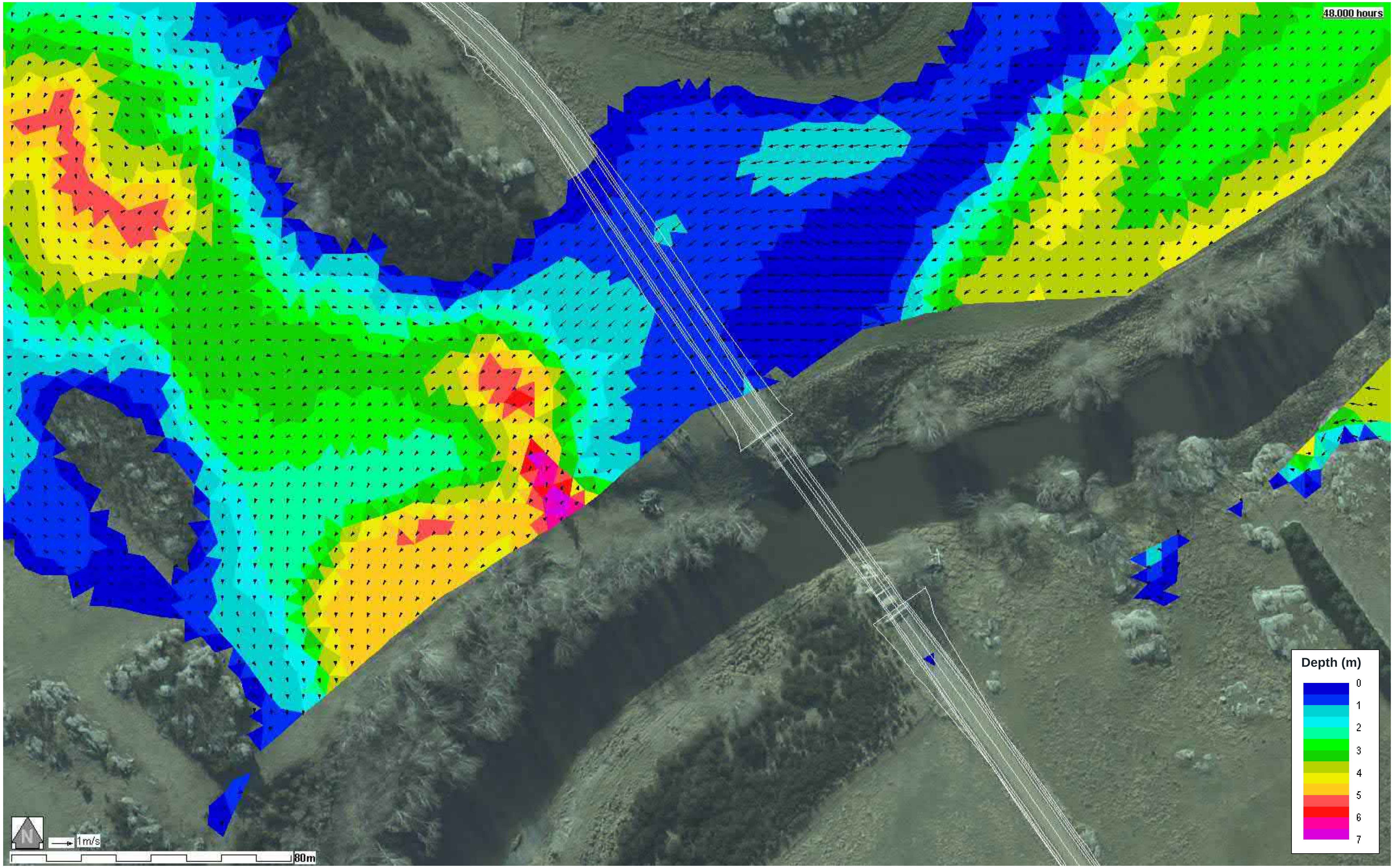


Figure B - 2 Predicted peak depth and velocity vectors near the proposed bridge (white lines) during an SLS (4% AEP, 2118 climate) event for the existing (no-bridge) scenario.

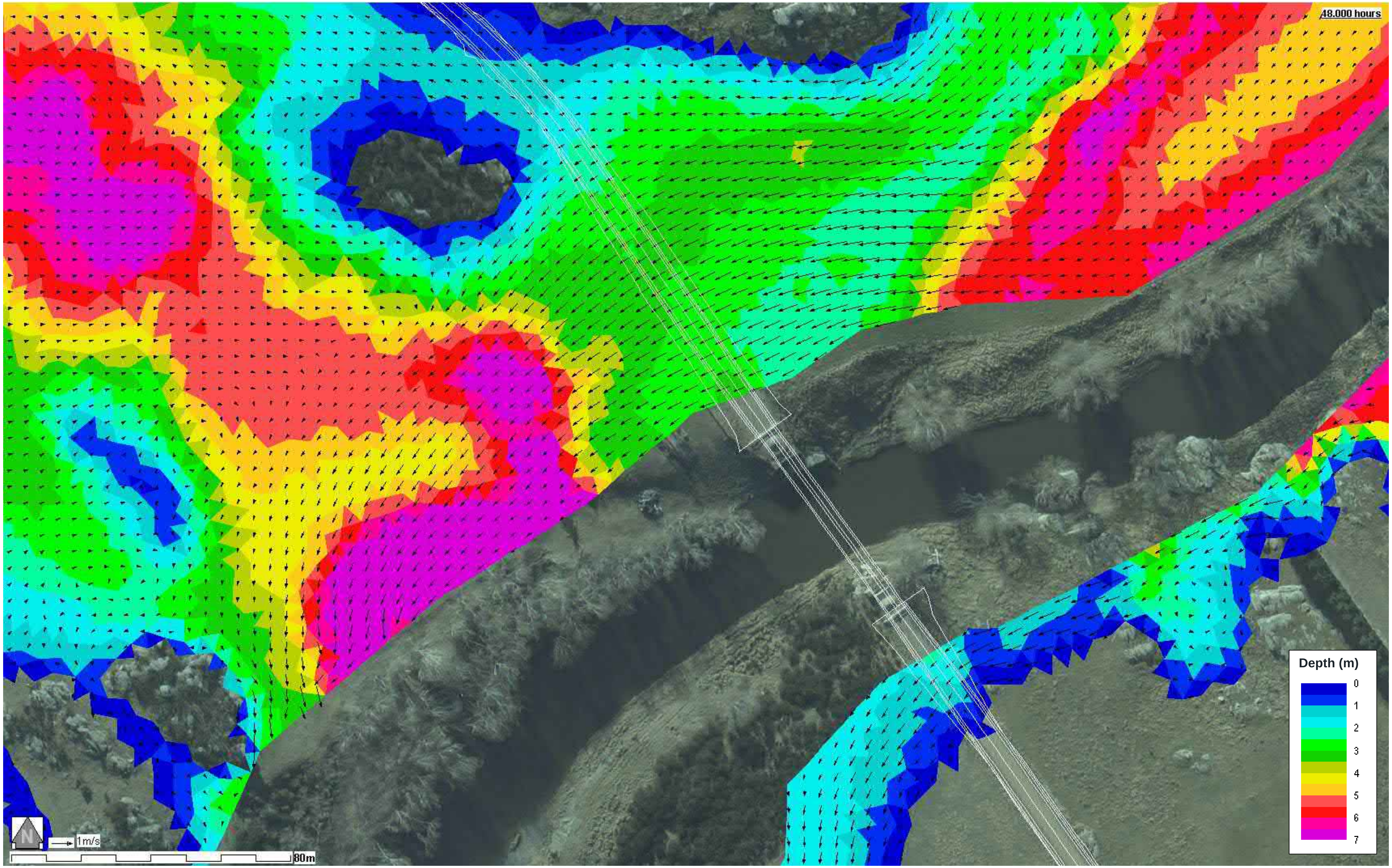


Figure B - 3 Predicted peak depth and velocity vectors near the proposed bridge (white lines) during a ULS (0.1% AEP, 2118 climate) event for the existing (no-bridge) scenario.

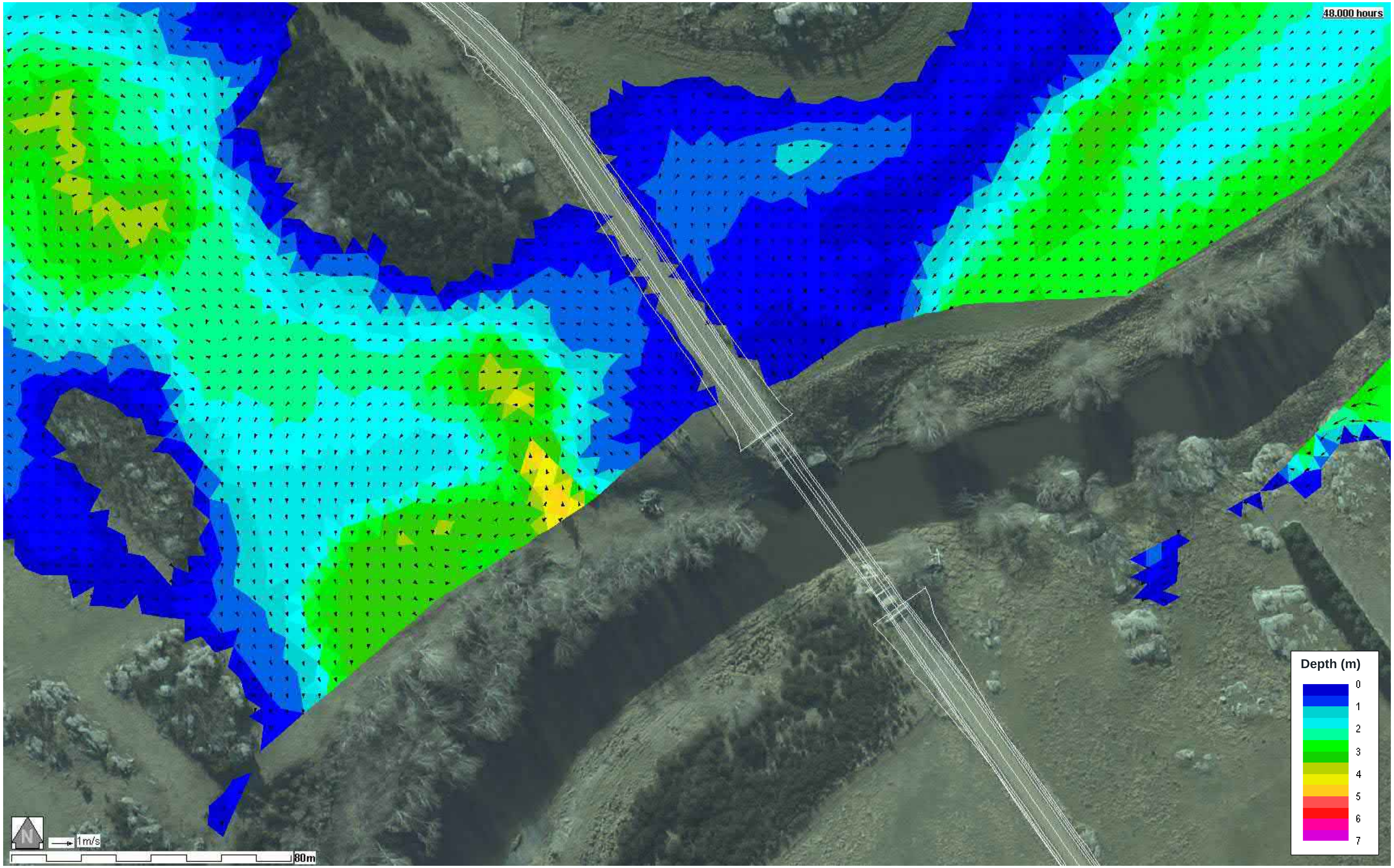


Figure B - 4 Predicted peak depth and velocity vectors near the proposed bridge (white lines) during an SLS (4% AEP, 2118 climate) event following construction of the bridge.

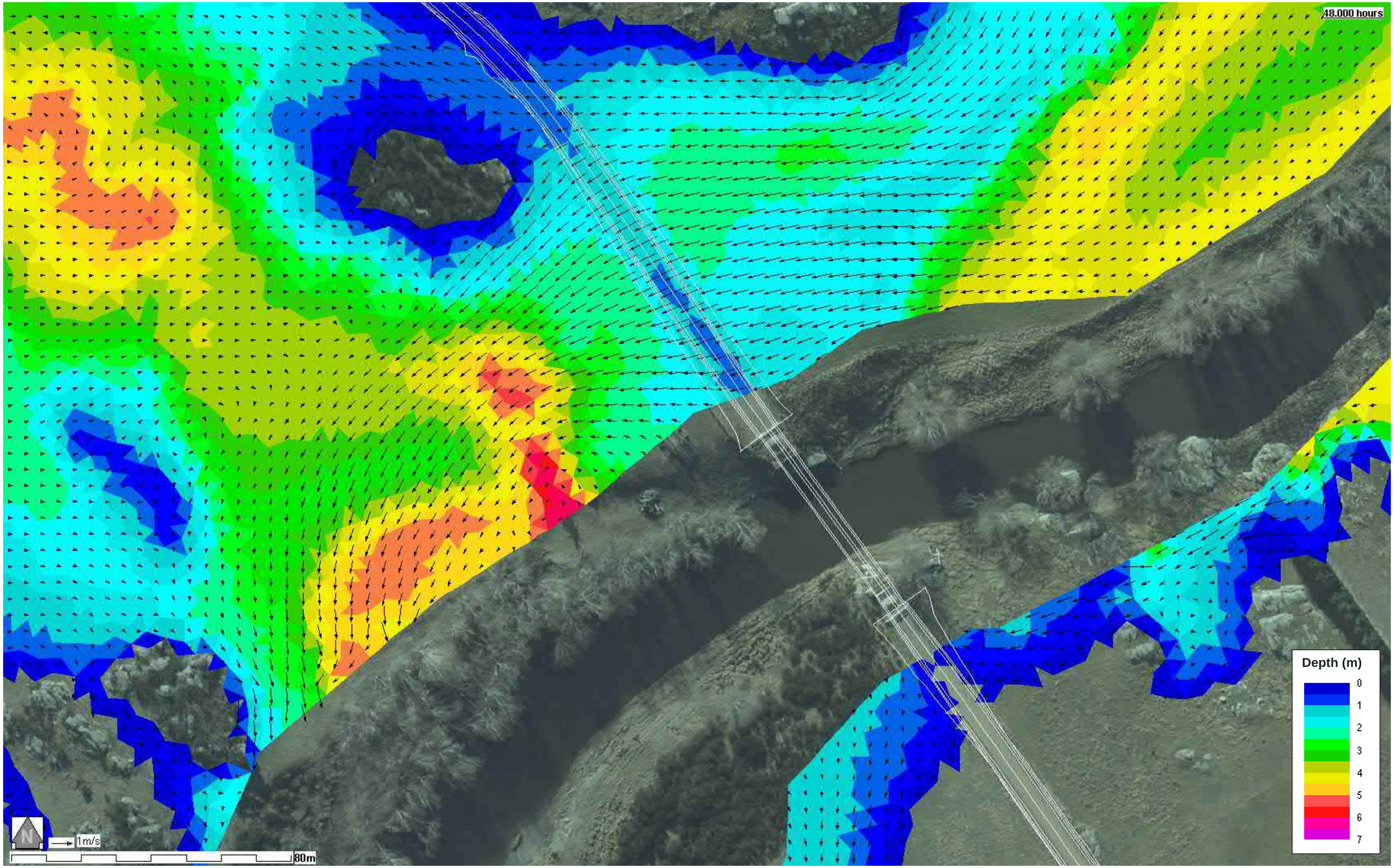


Figure B - 5 Predicted peak depth and velocity vectors near the proposed bridge (white lines) during a ULS (0.1% AEP, 2118 climate) event following construction of the bridge.

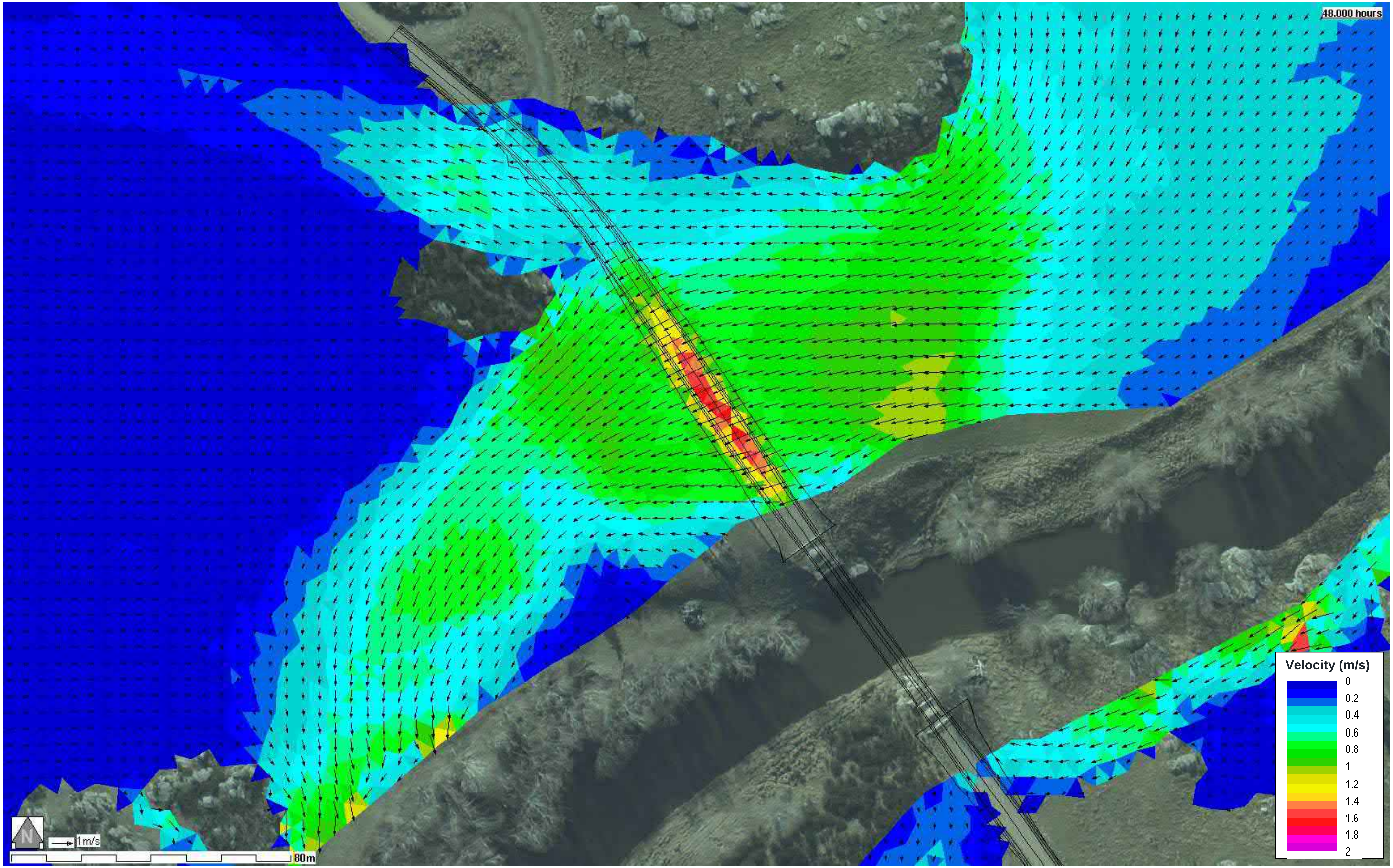


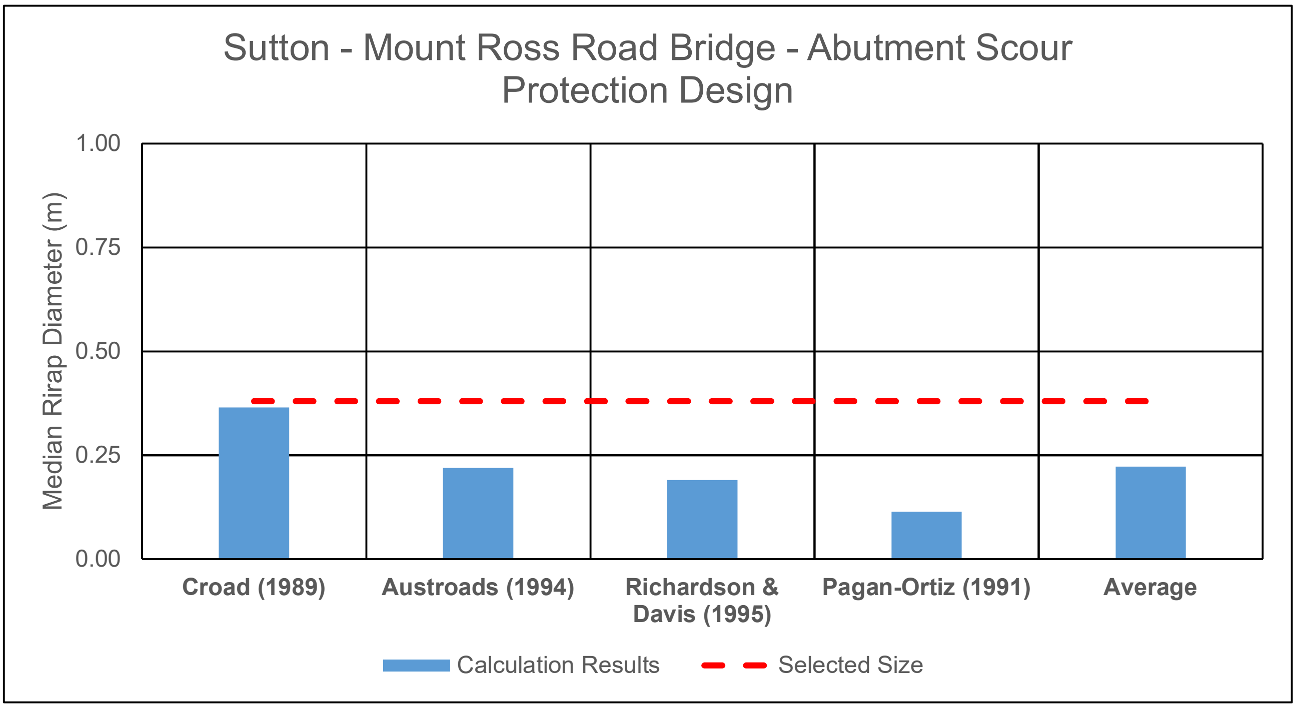
Figure B - 6 Predicted magnitude and direction of the peak velocity near the proposed bridge (black lines) during a ULS (0.1% AEP, 2118 climate) event following construction of the bridge.

Appendix C – Calculations

Project: Sutton-Mt Ross Rd   Date: 14/11/2018
Project No: 6-CD102.00/106GD

Created by Franciscus Maas

d_{r50}, Median Riprap Diameter (m)				
Croad (1989)	Austroads (1994)	Richardson & Davis (1995)	Pagan-Ortiz (1991)	Average
0.365	0.220	0.191	0.115	**0.222**



Project:	Sutton-Mt Ross Rd	Date:	14/11/2018
Project No:	6-CD102.00/106GD		
Created by	Franciscus Maas		
Method:	Croad (1989)		

Input data: SDF (1100m3/s)

g	9.81	m/s^2
S_s	2.65	Specific gravity of rock
$V_{approach}$	1.52	m/s
V_b	2.16	m/s
y	9.09	m Water Depth
α	26.6	Slope angle
θ	40	Angle of repose of riprap stone
K_{sl}	0.718	Embankment slope factor

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

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

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

Output data:

Saftey Factor	Fr	dr_{50}
1	0.229	0.365
1.2	0.229	0.438

Project: Sutton-Mt Ross Rd — Date 14/11/2018

Project No: 6-CD102.00/106GD

Created by Franciscus Maas

Method: Austroads (1994)

Input data: SDF (1100m3/s)

g	9.81	m/s^2	
S_s	2.65	Specific gravity of rock	
$V_{approach}$	1.52	m/s	
y	9.09	m	Water Depth

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

Output data:

Saftey factor	Fr	dr_{50}
1.5	0.161	0.220
2	0.161	0.293

Project:	Sutton-Mt Ross Rd	Date	14/11/2018
Project No:	6-CD102.00/106GD		
Created by	Franciscus Maas		
Method:	Richardson & Davis (1995)		

Input data: SDF (1100m3/s)

g	9.81	m/s^2	
S_s	2.65	Specific gravity of rock	
$V_{approach}$	1.52	m/s	
y	9.09	m	Water Depth
K_s	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:

Saftey factor	Fr	dr_{50}
1.5	0.161	0.191
2	0.161	0.254

Project:	Sutton-Mt Ross Rd	Date	14/11/2018
Project No:	6-CD102.00/106GD		
Created by	Franciscus Maas		
Method:	Pagan-Ortiz (1991)		

Input data: SDF (1100m3/s)

g	9.81	m/s^2	
S_s	2.65	Specific gravity of rock	
$V_{approach}$	1.52	m/s	
y	9.09	m	Water Depth

Spill-through abutment:

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

Output data:

Saftey factor	Fr	dr_{50}
1.5	0.161	0.115
2	0.161	0.153

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